Optimisation of Road Safety Treatment Strategies through Crash Modification Factors and Simulation

A Thesis Submitted by

Mohammad Nour Ibrahim Al-Marafi

B.Sc. Eng., M.Sc. Eng.

For the award of

Doctor of Philosophy

2019

Abstract

Road safety has become an intensively studied topic with an overarching aim of better understanding why road crashes occur and thus to reduce both frequency and severity. If it is known why road crashes occur, agencies should be able to better apply more effective and efficient road safety improvement strategies. The aim of the traffic engineer is to design and provide a safe travel environment to the road user. While road crashes cannot be completely prevented, a sound understanding of the causative factors helps to minimise crash rate. Crash occurrences can be viewed as a result of the interaction of numerous variables including road geometry, vehicle condition, and operational conditions such as speed and traffic volume.

The main objective of this research was to evaluate traffic and geometric road features and their influences on the safety performance of road intersections, roundabouts, and road segments by estimating suitable crash modification factors (CMFs). To accomplish the study objective, crash prediction models (CPMs) were developed using a generalised linear model (GLM) technique, i.e. Poisson or negative binomial (NB) distribution. The regional area of Toowoomba City, Australia was adopted as the case study. Traffic, geometric, and crash data on 106 road intersections for the years 2008-2015, as well as 49 roundabouts and 84 roadway segments for years the 2010-2015 were used for crash modelling and evaluation. The NB distribution was adopted in preference to Poisson distribution as the data showed over-dispersion. Several goodness-of-fit (GOF) tests were performed on the developed models to identify the better-fitting models. These models were then validated using both the estimation and validation datasets.

An accurate identification of hazardous road locations (HRLs) prevents wasted resources that may result if possible improvements at such locations are identified with less accuracy. The Empirical-Bayes (EB) approach was employed to identify the HRLs in the study area. This approach was adopted to provide more accurate safety estimation by accounting for the regression-to-the-mean bias usually associated with the road crash data. The HRLs were then ranked based on their potential for safety improvement (PSI) value, which is the difference between the expected and predicted road crashes at each location. The top 10 poorly performing locations for each of the road intersections, roundabouts, and road segments were identified for further investigation.

The CMFs identify any change in the safety performance resulting from implementing a particular treatment. In this study, CMFs were used to estimate the effect of the various proposed safety treatments at identified HRLs. The cross-sectional method (regression approach) was applied to estimate CMFs for individual safety treatment. This method has been considered recently and has not been extensively applied, however, it can be considered as a viable alternative method to estimate the CMFs in cases where observational before-and-after studies are not practical due to data restrictions.

In order to estimate the variation in the values of CMF with different sites characteristics, the crash modification functions (CMFunctions) were developed. Using CMFunctions, the safety effects of various traffic and geometric elements of different road facilities (i.e., intersections, roundabouts, and roadway segments) were investigated. The study also notes that while there has been substantial research in the broad area, very few studies have been undertaken to estimate CMFs for the combined effect of multiple safety treatments. However, the four most suitable techniques for estimating combined CMFs were reviewed and applied together to propose effective safety measures for the HRLs. Since there were variations in the estimation of combined CMFs using the four techniques, the average values were adopted as the best approach to estimate the effect of combined treatments. The results demonstrated that multiple treatments have higher safety effects (i.e., lower CMF) than single treatments. The results also indicated that the effect of treatments on road safety does not depend on the number of treatments that have been applied but rather depend on the quality and suitability of these treatments relative to the road's operating environment.

The traffic simulation software PTV VISSIM 9.0 was employed to assess the traffic operational performance before and after safety treatment implementation. The top 10 HRLs for each of the road facilities were simulated and evaluated under different scenarios in terms of level of service (LOS), traffic delay, travel time, and average speed. The results showed that there is no significant degradation of traffic operations expected at treated locations.

Finally, a benefit analysis was conducted to estimate the savings during the 10 years after applying the proposed treatments. The crash reduction factors and crash costs were utilised to estimate the crash cost reduction that was associated with single and combined treatments. Such estimation can support road authorities and practitioners to select the final treatment plans for the identified HRLs by undertaking benefit-cost analysis to assist the decision-making process.

Contributions of this research can be summarised as: (i) to develop CPMs for different types of road facilities, (ii) to develop CMFunctions to estimate the variation in the values of CMF with different sites characteristics, (iii) to propose a methodology to identify the most appropriate safety treatments (single and multiple treatments) using CMFs, costing and simulation packages. The research has also identified some important aspects for future research to extend the present work.

Certification of Thesis

This thesis is entirely the work of Mohammad Nour Al-Marafi except where otherwise acknowledged. The work is original and has not previously been submitted for any other award, except where acknowledged.

Dr Kathirgamalingam Somasundaraswaran

Principal Supervisor

Prof Ron Ayers

Associate Supervisor

Prof Frank Bullen

Associate Supervisor

Student and supervisors signatures of endorsement are held at the University.

Acknowledgements

First of all, I would like to thank God who without his blessings, I could not achieve what I achieved till now.

I would like to express my deepest thanks and appreciation to Dr Kathirgamalingam Somasundaraswaran, my principal supervisor, for his patience, invaluable guidance, encouragement, and support throughout this work. His leadership and experience provided the opportunity to achieve the goal of the research in a reasonable time. He even took time out of his busy schedule to help me overcome the challenges I faced during my study. Many thanks are also extended to my associate supervisors, Prof Ron Ayers and Prof Frank Bullen for their valuable time, suggestions, and advisory support towards the fulfilment of this work. It is a great honour for me to be their student. Enormous thanks to Dr Barbara Harmes for proofreading of this thesis.

I am thankful and a beneficiary of the Tafila Technical University (TTU) for providing the financial support required fulfilling this work. My sincere appreciation goes to the University of Southern Queensland (USQ) for enabling me to obtain the resources necessary throughout my academic study. I would like to extend my gratitude to the Toowoomba Regional Council (TRC), and to the Department of Transport and Main Roads (DTMR) in Queensland for their help in providing the required data to accomplish this work. I would also like to extend my gratitude to PTV Planung Transport Verkehr AG, Germany for their kindness in providing a licence to use VISSIM software.

A very special thank goes to my father Ibrahim Khalaf and my mother Enam Sobhi who without their prayers and encourages, I would not reach this stage. I owe you this achievement. I extend my sincere gratitude to my lovely wife Rogayah Bassam and my lovely daughter Sara Mohammad Nour for their patience, understanding, and moral support me a lot throughout my study. I am also sincerely grateful to my brothers, sisters, relatives and friends for their inspiration and pray.

Associated Publications

Journals

Al-Marafi, MN & Somasundaraswaran, K 2018, 'Review of crash prediction models and their applicability in black spot identification to improve road safety', *Indian Journal of Science and Technology*, vol. 11, no. 5.

Al-Marafi, MN, Somasundaraswaran, K & Bullen, F, 'Development of Crash Modification Factors for Intersections in Toowoomba City', *International Journal of Urban Sciences (Under review).*

Al-Marafi, MN, Somasundaraswaran, K & Ayers, R, 'Developing crash modification factors for roundabouts using a cross-sectional method', *Journal of traffic and transportation engineering (English edition) (Accepted).*

Conference Papers

Al-Marafi, MN & Somasundaraswaran, K 2018. Development of Crash Prediction Models for Road Segments. In: *2nd Conference of Asean Road Safety (CARS 2018),* 4-6 December 2018, Kuala Lumpur, Malaysia.

Table of Content

List of Figures

List of Tables

Appendix D

List of Abbreviations

CPM	Crash Prediction Model
SPF	Safety Performance Function
GLM	Generalised Linear Model
CMF	Crash Modification Factor
CMFunction	Crash Modification Function *(used to show the variation in the values of CMF)*
CRF	Crash Reduction Factor
AASHTO	American Association of State Highway & Transportation Officials
HSM	Highway Safety Manual
HCM	Highway Capacity Manual
GDP	Gross Domestic Product
NRSS	National Road Safety Strategy
PSI	Potential for Safety Improvement
EB	Empirical Bayes
FB	Full Bayes
SI	Sichel
BA	Before-After
CG	Comparison Group
YC	Yoked Comparison
NB	Negative Binomial
ZINB	Zero-Inflated Negative Binomial
AADT	Annual Average Daily Traffic
AIC	Akaike Information Criterion
GOF	Goodness-Of-Fit
df	degree of freedom
MSPE	Mean Squared Prediction Error
MAD	Mean Absolute Deviation
MSE	Mean Square Error
R^2_{FT}	Freeman Tukey R-squared
RTM	Regression-To-the-Mean
CURE	Cumulative Residual
HCL	High Crash Location
HRL	Hazardous Road Location
AUD	Australian Dollar
Std. Er.	Standard Error
TRC	Toowoomba Regional Council
LOS	Level of Service
PV	Present Value
B/C	Benefit-Cost ratio

Chapter 1
Introduction

1.1 Background

Road traffic crashes are major global social and economic issues, as shown by reports from many countries around the world. The World Health Organization (WHO 2015) indicated that worldwide road crashes claim nearly 1.24 million lives a year, while about 20 to 50 million people sustain road crash injuries. In addition, road crashes cost many countries about 3% of their gross domestic product. In Australia, there were 1295 fatalities, 32,300 serious injuries, and 224,104 minor injuries in 2016, costing the nation more than an estimated $33.1 billion (BITRE 2017; Litchfield 2017). This number of fatalities and injuries have a huge impact on the families affected, whose lives are often changed irrevocably. Road fatalities also impact on the societies in which the killed and injured people worked and lived through associated emotional and financial losses.

The prediction of road crashes is very complex, depending upon a large range of factors including the pattern of traffic movement, the existence of mixed vehicle types in the traffic stream and random human actions. Life and property losses focus the traffic engineer's thoughts on the need to provide a safe pattern of traffic movement to road users and to maximise their safety. For this reason, systematic studies of traffic crashes are regularly undertaken. Correct and consistent verification of the cause of crashes can help to identify preventive and corrective measures in terms of traffic control and road geometric and textural design at potential crash locations.

1.2 Research Problem

The overarching research problem is the reduction of road crashes to benefit society. Traffic safety agencies, in the past, have typically used measures of the rate of the number of crashes (as a function of traffic volume) or the absolute number of crashes at a location, to determine if the location had a traffic safety problem compared to other locations with similar conditions. However, these methods tend to be subjective, short sighted, and reflect an outdated view on road safety (Hauer 1995; Lord & Mannering

2010; Tegge et al. 2010). Other techniques have emerged to deal with the shortcomings related to such techniques. These newer techniques focus on predicting the relation between the traffic crash frequency and other variables that effect crash frequency, such as traffic volume and road geometry. Relationships can be developed using statistical models to provide a realistic and accurate prediction of crash frequency and thus help to identify suitable measures to reduce crashes.

In recent years, several studies have been conducted to investigate the impact of various road geometric design elements and traffic volumes on safety, using crash prediction models (CPMs) and/or crash modification factors (CMFs). The recognition of any change in geometric design features or traffic operation will increase or decrease crash frequency. CMFs, in particular, identify the change in road safety (crash frequency) resulting from implementing a particular treatment. This treatment may be in the form of design modification, change in traffic operations, or any countermeasures. Ideally, CMFs can be an important tool that assists road safety agencies to select the most appropriate treatments to achieve the highest return on investments.

Observational before-after (BA) studies are the most common method used to estimate CMFs. Generally, there are five approaches which can be employed to implement observational BA studies: (i) comparison group (CG) approach, (ii) naïve approach, (iii) full Bayes (FB) approach, (iv) empirical Bayes (EB) approach, and (v) yoked comparison (YC) approach. The observational BA studies include estimating safety performance based on safety data before and after a specific treatment is implemented on either one or several sites (Shahdah et al. 2014). The EB and FB approaches can be used to control regression-to-the-mean (RTM) bias associated with observational studies (Persaud & Lyon 2007; Gross et al. 2010). Although the observational BA studies are considered to be the preferred method for estimating CMFs, there are some practical limitations associated with this method. As examples, countermeasures or treatment implementation dates should be known to determine the before and after evaluation periods; sufficient years have to pass after treatments are implemented; and it is difficult to distinguish safety effects when more than one treatment has been implemented at a specific site (Hauer 1997; Persaud et al. 2010; Wood et al. 2015). In such cases, the cross-sectional method (regression approach) can be employed to estimate CMFs because of its simplified approach for obtaining data compared to observational BA studies. It is worth noting that the cross-sectional method does not take into account the effects of factors that are not included in the analysis, i.e. external causal factors (Gross et al. 2010; Hauer 2013). However, this method can be considered as a viable alternative method that can be adopted in cases where observational BA studies are not practical due to data restrictions.

1.3 Research Gap

The review of the available international literature revealed that the focus was only on developing CMFs and applying these factors to identify the appropriate treatments on the basis of the crash reduction percent achieved. At the time of writing, there has been no in-depth study that has incorporated traffic simulation models with CMFs to evaluate the impact of the proposed safety treatments on both traffic operation and road safety at the same time. Also, most of the previous studies have ignored the variation of CMF values among treated sites by estimating CMF as a fixed value. Ideally, it is not logical to assume a systematic safety effect for all treated sites with different characteristics. For instance, greater benefits of safety improvements may be obtained at the sites with higher traffic volumes. As a part of the cross-sectional method, a crash modification function (CMFunction) formula can be developed to estimate the variation in the values of CMF with different site characteristics, rather than using a single value. For estimating the combined safety effects of multiple treatments, HSM part D suggests multiplying the values of CMF for individual treatments. However, the HSM indicated that calculating combined CMF using a simple multiplication approach may result in overestimating or underestimating the expected crash frequencies, as this approach assumes that the road safety effect of each treatment is independent. In this research, several approaches are suggested to more reliable estimate the values of combined CMF.

1.4 Research Hypothesis

If the reasons for road crashes occur are known, then road agencies could be able to identify and implement road safety improvement projects more effectively and efficiently. The hypothesis for this thesis is: *"Could a better understanding of the main contributing factors in road crashes help in identifying and applying effective crash reduction measures at critical locations?"*

1.4.1 Underpinning Assumptions

The key assumptions made in the research are:

1. The generalised linear model (GLM) techniques including Poisson or negative binomial distributions can be fitted crash count data. Thus, the study makes the assumption that road crashes have either a Poisson or a negative binomial distribution.
2. The main assumption in developing CMFunction using the cross-sectional method is that CMFs for each explanatory variable follow exponential relationship.
3. The average delay and travel time for the sites used in the validation processes (simulation stage) not available for the full 24 hours of the day and the assumption has been made that the peak periods are acceptable to define the validation parameters.

1.5 Research Questions

While the research hypothesis provides the overarching "research question", it is useful to outline the underpinning research components that form the body of the thesis and are encapsulated within the various Chapters.

1. Can crash prediction models be used to identify high risk locations?
2. Can the cross-sectional method be used to develop CMFs for safety treatments?
3. Does applying multiple safety treatments improve safety outcomes?
4. Is it possible to utilise traffic simulation and cost-effectiveness to determine appropriate safety treatments?

1.6 Research Objective

To answer the hypothesis, the research proposes effective crash reduction measures for different roadway categories including intersections, roundabouts, and roadway segments using CMFs for both single and multiple safety treatments. In order to understand the main limitations associated with CMF development approaches and define the most appropriate approach, the research began with a comprehensive review of the available international literature. The contributing elements underpinning the hypothesis can be summarized as follows:

1. Develop specific Crash Prediction Models (CPMs) using an appropriate statistical modelling technique and assess the performance of the models using data from Toowoomba, Queensland, as a case study.
2. Identify some of the sites in the case study that have a higher than expected number of crashes for further investigation of safety improvements using an appropriate approach.
3. Develop specific Crash Modification Functions (CMFunctions) using a cross-sectional method. These functions were employed to estimate the values of CMFs for various road safety treatments at the identified sites.
4. Identify and quantify the multiple safety treatments that significantly affect road crash reduction through calculating combined CMFs.
5. Simulate the traffic operation to indicate any changes in its quality after hypothetical safety treatments on the identified sites (i.e., before-after evaluation).
6. Identify the best treatments for safety improvement using total crash reduction and total economic gain including the use of benefit-cost ratios.

The above objectives are strongly linked with the research questions as follows: objectives 1 and 2 addressed question 1, objective 3 addressed question 2, objective 4 addressed question 3, and objectives 5 and 6 addressed question 4.

1.7 Research Limitations

The limitations of the research are outlined through the following:

1. The data used in the modelling stage are for the severe-crash type without including the type of property damage only crashes. High traffic volumes (AADT) and high heavy vehicle percentages are not included.
2. The road intersections in the modelling process were analysed as a whole to investigate the effect of common risk factors, not in different groups such as signalised or un-signalised intersections and three-leg or four-leg intersections.
3. The roadway segmentation process undertaken is based on the method of homogeneous segments with respect to traffic volume and geometric characteristics.
4. The detailed expected treatment costs associated with each proposed treatment type are not available, as the expected cost of treatments varied according to the

particular location and annual maintenance cost. The total discounted benefits have been estimated for all proposed treatments.

1.8 Thesis Structure

This thesis has been presented in seven chapters. Chapter one provides a background of the study, the research gap, hypothesis, questions, and objectives as well as the limitations of this research. Chapter two reviews some of the important previous studies related to road crashes, crash prediction models (CPMs), black spot identification, crash modification factors (CMFs), and traffic simulation. The CMF development methods including various observational Before-After (BA) studies and cross-sectional method are presented, and related issues are discussed. Moreover, current techniques for combining individual CMFs are discussed.

Chapter three presents the data collection process and methodology that are adopted in the analysis stage. The data collection process comprises three parts: identifying the study area; data collection and preparation; and selecting the road facility. The methodology that was followed to achieve the study objectives comprises five parts: model development and validation; identifying high crash locations (HCLs) or black spots; estimating single and combined crash modification factors; traffic simulation; and economic analysis.

Chapter four proposes the most appropriate road safety measures for the top 10 hazardous intersections in the study area based on the values of combined CMF. This chapter considers all research questions and objectives with respect to the practical aspect of intersection analysis. The values of CMF for various safety measures were estimated using a cross-sectional method (regression approach). Then, four techniques were employed to calculate the values of combined CMF for proposed safety treatments. The proposed safety treatments were evaluated using simulation models and expected crash cost reductions.

Similarly, chapter five identifies the appropriate safety treatments for the roundabouts with high crash risk. The all research questions and objectives were also addressed in this chapter with respect to the practical aspect of roundabout analysis.

Chapter six provides details on how geometric and operational elements impact on road safety and also identifies the most appropriate treatments on hazardous roadway segments. In this chapter, the research questions and objectives were also addressed with respect to the practical aspect of roadway segment analysis.

Finally, chapter seven provides a summary and conclusion of the major findings, research application, and recommendations for future works.

Chapter 2

Literature Review

2.1 Introduction

Statistical modelling is widely used to develop crash prediction models (CPMs) relating crash occurrences on a road network to the geometric and traffic characteristics of the roads. These models have applications such as estimating the potential crash frequency on road networks, identifying the factors contributing to crashes and, evaluating the crash reduction benefits of implemented treatments. Several studies have been conducted to evaluate and understand the nature of road crashes (Pecchini et al. 2014; Polders et al. 2015; Kamla et al. 2016; Vayalamkuzhi & Amirthalingam 2016; Dong et al. 2017; Wang et al. 2018). This chapter reviews the contemporary international literature related to road crashes, crash prediction models, black spot identification, crash modification factors, and traffic simulation. The chapter covers seven sections which show in the following graphical layout.

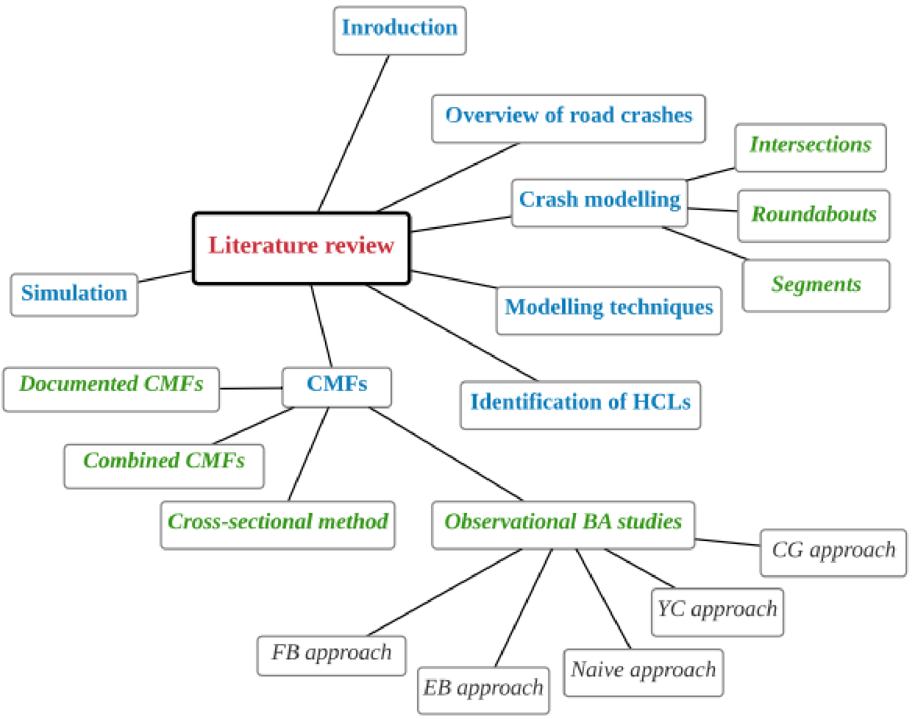

Figure 2.1 Chapter 2 outline and roadmap

2.2 Overview of Road Crashes

Road crashes on a worldwide basis result in hundreds of thousands of fatalities, millions of injuries and hundreds of billions of dollars in economic costs annually (Litman 2009; WHO 2015). Without significant efforts to improve road safety, particularly in developing countries, the number of fatalities due to road crashes has been predicted to increase by 75% between 1999 and 2020 (Jacobs & Aeron-Thomas 2000). In March 2010, the United Nations General Assembly adopted a proposal on improving global road safety. This proposal was presented by the Russian Federation and supported by over a hundred countries including Australia, with the aim of minimizing the number of road traffic fatalities between 2011 and 2020 (WHO 2013). Figure 2.2 illustrates the possibility of saving about five million lives as a result of this decision. The reduction of crashes and their consequences is viewed as being of major importance to all countries. Improved crash analysis and determination of suitable preventative measures is needed to reverse the upward trend in the number and severity of road crashes. The large number of road crashes is not only a social issue that costs many people their lives but is also an economic issue that costs societies large sums of money and adds undesirable economic burdens. WHO (2015) stated that road crashes can cost developing countries between 1% and 3% of their Gross Domestic Products (GDPs) per annum. For example, BITRE (2009) estimated that the social cost of road crashes in Australia was $17.85 billion in 2006 which was equal to about 1.7% of GDP.

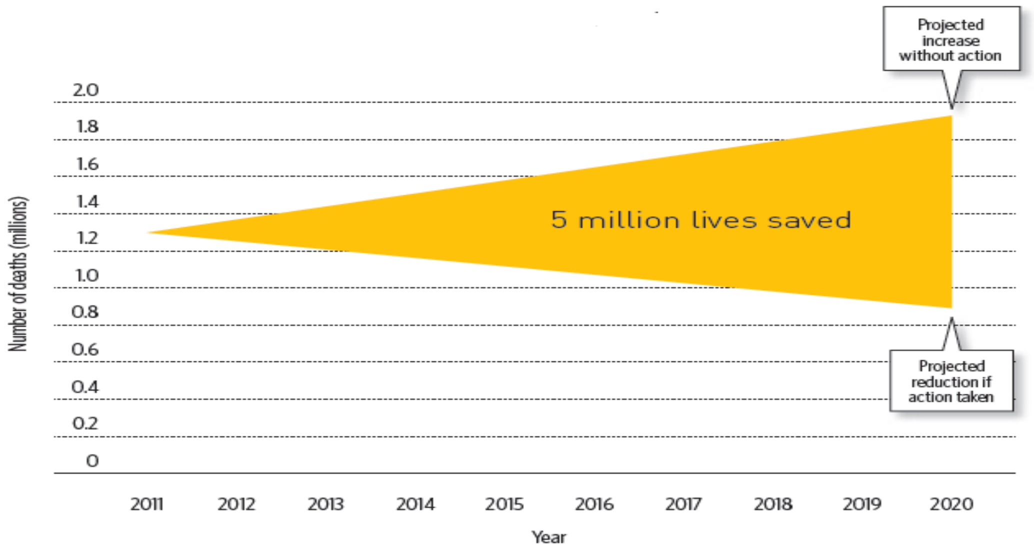

Figure 2.2 The expected number of road crashes fatalities between 2011 and 2020
Source: WHO (2013)

Ismail and Abdelmageed (2010) pointed out that road crashes cost low and middle-income countries more per year than the total aid received for development purposes. As a result of this socio-economic problem, countries continue to develop and apply more radical approaches to the road safety problem. For example, Sweden is one of the countries with the least number of road fatalities relative to its population, but to improve on this record the Swedish Parliament introduced the "Vision Zero" approach, which aims to make the roads free from fatalities and serious injuries by 2020 (Johansson 2009). In Australia, the National Road Safety Strategy (NRSS) introduced in 2011 a target to reduce road fatalities by 30% by 2020 as shown in Figure 2.3.

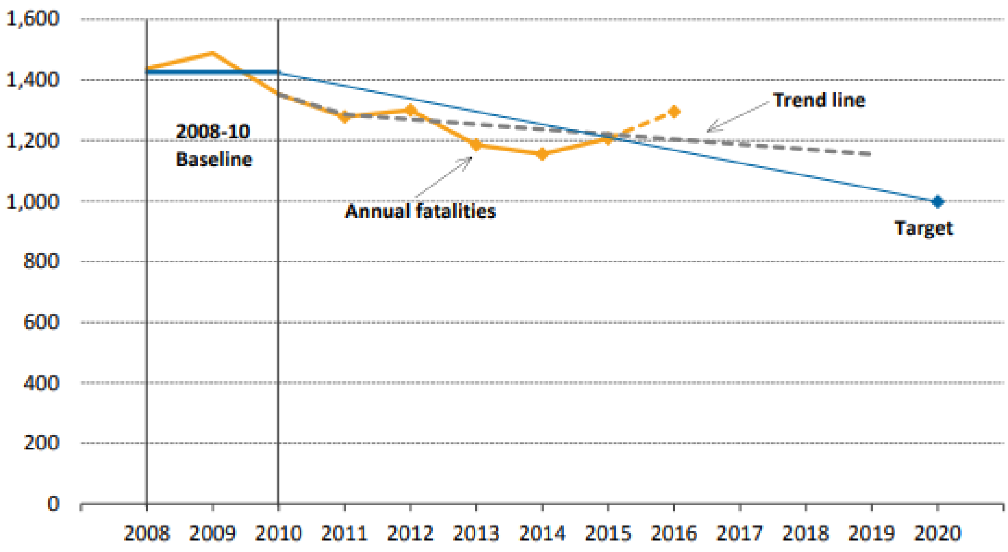

Figure 2.3 NRSS statistical progress towards fatality target between 2011 and 2020
Source: BITRE (2017)

2.3 Crash Prediction Modelling

Crash prediction models (CPMs) have been found to be as a useful tool by road engineers and planners. Substantial research has been conducted over the years on the development of CPMs for estimating the predicted number of crashes and safety impacts on various roadway types. A review of the use of CPMs for intersections, roundabouts, and roadway segments is provided in the following sub-sections.

2.3.1 Intersections

Many studies have been conducted over the years on the development of CPMs for the prediction of possible crashes at road intersections. Given that intersections are amongst the most hazardous sites on road networks (due to both geometric configuration and traffic concentration), they are one of the most important elements that must be considered if the road network is to accommodate a safe flow of traffic in all directions.

Chin and Quddus (2003) estimated the expected crash number at 52 signalized intersections in Malaysia between 1992 and 1999. They developed a random-effects model to evaluate the relationship between the frequency of road crashes and the geometry, traffic and control characteristics. The study concluded that three variables (the use of adaptive signal control, the presence of bus bays and the presence of an acceleration section) tended to lower crash frequency. In a study by Bauer and Harwood (2000) lognormal, Poisson and negative binomial regression analyses were used to develop statistical models to investigate the relationship between road crashes and highway geometry, traffic control, and traffic volume variables for at-grade intersections in California, USA. This study concluded that the negative binomial and lognormal distributions were more suitable for modeling road crashes than the normal distribution.

Negative binomial (NB) models were used by Wang and Nihan (2001) to estimate the frequency of angle crashes at signalised intersections in Tokyo, Japan. The study collected the data from 81 signalized intersections between 1992 and 1995. The analysis found several factors affecting crash risk, including number of through entering lanes, angle of entering and exiting approaches, intersection location, entering approach speed limit, and the presence of a pedestrian overpass at approaches. El-Basyouny and Sayed (2013) investigated the relationship between road crashes and conflict points at intersections. The negative binomial model was also used in this study to predict crash frequency. The model was applied to the data from 51 signalised intersections in British Columbia, Canada. The results showed a significant proportional relationship between crashes frequency and conflict points. In addition, the study found that the number of predicted conflict points increased with the traffic volume.

Both negative binomial and Poisson distribution models were also used by Sayed and Rodriguez (1999) to develop crash models for non-signalised intersections in British Columbia. The generalised linear model (GLM) approach was applied to overcome the limitations associated with conventional linear models when applied to crash analysis. In their study, the authors estimated the parameters of the crash models based on a methodology shown in the earlier work of Bonneson and McCoy (1993). However, in their study, four applications of crash prediction models were described, namely developing critical crash frequency curve, before-and-after evaluation, identification of high crash locations (HCLs), and ranking HCLs. The research helped prove the usefulness of crash prediction models in reliably evaluating the safety of intersections. Oh et al. (2004) established crash prediction models at signalised and stop controlled intersections in rural areas using negative binomial and Poisson techniques. To complete this study, geometric characteristics, traffic volume, and crash data were collected from 100 signalised intersections and 260 stop-control intersections. Several goodness-of-fit (GOF) measures were also computed to evaluate the suitability of the predicted models. Regardless of geometric characteristics and intersection type, the results found that traffic volume significantly affected the safety performance of the intersections.

In a study by Dissanayake and Roy (2014) a binary logistic regression model was used to identify the main factors that affected road crash severity. In this study, the data were collected between 2004 and 2008 in Kansas City. The study concluded that some of the significant variables that affect the probability of road crashes are asphalt type road surface, speed, alcohol involvement, driver age, medical condition of the driver, daylight, type of vehicles, and fixed roadside object types such as trees. The same method was used by Chen et al. (2012) to investigate the factors that significantly impact on intersection crashes involving injuries in Victoria, Australia between 2000 and 2009. The results showed seven factors significantly related to the severity of intersection crashes, including speed zone, driver gender and age, time of day, seat belt usage, traffic control type, and crash type. Park et al. (2016) investigated specific characteristics of road crashes at rural non-signalised intersections using ordered logistic regression models. The results revealed that contributory factors associated with road crashes at non-signalised intersections were traffic volume, poor sight distance, angle of intersection, traffic violation number at intersection, time of day, heavy vehicles proportion, and number of lanes on minor road.

Abdel-Aty and Keller (2005) studied various factors that affect crash severity at signalised intersections. The study used an ordinal probit model technique to analyse the crash data from the years 2000 and 2001. The results showed that the presence of a median island and increasing posted speed limit up to 65 mph on the minor road were associated with lower crash severity. The same method was used by Tay and Rifaat (2007) to determine the risk factors that affect the severity of road crashes at intersections in Singapore. The study revealed that road type, vehicle type, driver's characteristics, crash type, and time of day were significant determinants of crash severity at intersections.

In their study, Gomes et al. (2012) developed crash prediction models using Poisson gamma distribution models. The models were estimated using data collected for 50 four-legged and 44 three-legged intersections in Lisbon. The study found the negative impact on safety was associated with the presence of a right turn lane on the major approach and the presence of a median island on the minor approach. Harwood et al. (2003) used before and after evaluation to study the impacts of the right and left turn lanes on safety at intersections. Data for geometric design, traffic volume, traffic control, and road crashes were collected from 300 not improved intersections as well as 280 similar intersections that were improved. The study concluded that adding both right and left turn lanes was effective in optimizing safety at signalised and non-signalised intersections. Similarly, a prior study by Vogt (1999) concluded that the presence of a left turn lane for four-legged non-signalised intersections resulted in improving road safety. In contrast, Dong et al. (2017), found that the number of left turn lanes on major and minor approaches was associated with more crashes at signalised intersections. The study also indicated that lower posted speed limits were associated with lower number of road crashes.

The influence of traffic control type on intersections safety was analysed by Leong (1973) and Greibe (2003). Leong (1973) showed that the presence of signal control reduced the average of road crashes at four-legged non-signalised intersection. While, the effect of signal control was negligible at three-legged non-signalised intersections. Greibe (2003) examined the impact of signal control on road safety at intersections. The study found that signalised intersections in general were as safe as non-signalised intersections with the same traffic volume.

Studies undertaken by Leong (1973); David and Norman (1975); Hanna et al. (1976); O'Brien (1976); Park et al. (2016) have concluded that four-legged non-signalised intersections were associated with more road crashes compared to three-legged non-signalised intersections. Park et al. (2016) revealed that crash frequency at four-legged intersections was found to have 1.53 times more than at three-legged intersections.

Similarly, studies conducted by Bauer and Harwood (1996) and Harwood et al. (1995) showed that four-legged intersections experienced twice the number of road crashes compared to three-legged intersections.

Kumara and Chin (2005) analysed the factors affecting road safety at signalised intersections in Singapore. Poisson distribution models were employed to analyse nine years of crash, traffic volume, geometric characteristic, and traffic control data from 104 intersections. The results showed that traffic volume, number of signal phases, right turn slip lane, surveillance cameras, gradient, and median railings significantly affect the occurrence of road crashes at intersections. Chin and Quddus (2003) employed random effect negative binomial (NB) models to identify the contributory factors that affect intersection safety. Crash data from a total of 52 intersections in Singapore were used in the analysis, which collected data between the years 1992 and 1999. In this study, a total of 32 explanatory variables were considered for use, including geometric characteristics, regulatory control measures, and traffic volume. The results revealed 11 explanatory variables that significantly affected road safety at the intersections. Four variables were considered to be highly significant: total traffic volume, uncontrolled left turn lane, number of phases per cycle, and presence of a surveillance camera.

Kumara and Chin (2003) applied a zero-inflated negative binomial (ZINB) model technique to investigate the effect of geometric characteristics, traffic volume, and traffic control on likelihood of crash occupancy. This technique was used in the study to deal with the excess zero crashes that were recorded at the investigated sites. The crash data from 104 signalized 3-legged intersections in Singapore for a period of 9 years, from 1992 to 2000. The model indicated that right turn channelization, median railings, acceleration section on a left turn lane, and approach gradient of more than 5% tended to reduce crash frequency. On the other hand, total approach volume, uncontrolled left turn slip road, large number of signal phases, and short sight distance tended to increase crash frequency.

In summary, different CPMs have been developed to study the effects of different traffic and geometric variables on intersection-related crashes. The literature review shows that explanatory variables related to traffic volume, traffic control, and geometry elements have made a significant contribution to occurrences of intersection crashes. Statistical models such as Poisson and negative binomial (NB) have been widely used in developing intersection crash models.

2.3.2 Roundabouts

A number of studies have been conducted to investigate the effects of the geometric elements and traffic conditions on safety at roundabouts (De Brabander & Vereeck 2007; Daniels et al. 2011; Anjana & Anjaneyulu 2014; Kamla et al. 2016; Farag & Hashim 2017). In order to gain a better understanding of crash causes and contributing factors, the researchers have paid considerable attention to developing different analytical approaches.

Arndt and Troutbeck (1998) developed multiple linear regression models to investigate the effects of roundabout geometry variables on the number of road crashes in urban and rural areas of Queensland, Australia. A total of 492 crashes and 100 roundabouts on urban and rural roads were studied. Data for geometric design, traffic volume, traffic control, and crashes were collected between 1986 and 1990. Three models were proposed to fit varying crash types (single vehicle crashes, entering-circulating crashes, and approaching rear-end crashes). This study concluded with recommendations for the design and construction of roundabouts that would minimize the number of crashes.

A study performed by Farag and Hashim (2017) evaluated the safety performance of the roundabouts using a generalised linear model (GLM) approach, i.e. Poisson and negative binomial (NB) models. Two types of crash models were estimated separately: flow based crash models containing only exposure variables; and full crash models containing exposure variables as well as geometry and traffic variables. In the study, data were collected from 15 roundabouts in Oman over a period of three years. The results showed that the number of lanes at specific approach, entry angle, circulating width, and 85^{th} percentile speed significantly affected safety performance at roundabouts. In addition, increasing the number of lanes and installation of a right turn lane were found to be associated with lower crash frequency.

Sacchi et al. (2011) developed crash prediction models (CPMs) to assess roundabout safety performance in Italy. The NB distribution model was used to analyse data and then the cumulative residual plots method was employed to evaluate the model transferability. The results revealed that based on a comparison carried out using models from other countries (United Kingdom, United States of America, Canada, Sweden, and New Zealand) that Italian roundabouts tended to be less safe.

Kamla et al. (2016) investigated the traffic and geometric characteristics and their impacts on the frequency of crashes. A total of 70 roundabouts, including all recorded crashes was used in the study. The results indicated that the crash frequency tended to increase as the traffic volume and inscribed circle diameter increased. Retting (2006) and Rodegerdts et al. (2010) also concluded that a larger inscribed circle diameter leads drivers to increase their circulating speed and thus increases the risk of crashes at roundabouts. The influence of a splitter island (Figure 2.4) on roundabout safety was examined by Montella et al. (2012); Anjana and Anjaneyulu (2014); Austroads (2015). The studies concluded that the presence of splitter islands have positive impacts on safety as these can be used to control the entry speed.

Kim and Choi (2013) identified the major factors associated with road crashes at roundabouts in South Korea. The NB distribution models were applied to analyse the impact of contributory factors on road safety using data from 14 roundabouts. In this study, a total of eleven explanatory variables were examined. The results showed that four explanatory variables have positive impacts on roundabout safety: inscribed circle diameter, flare length, circulating lane width, and central island diameter. On the contrary, seven explanatory variables have negative impacts on roundabout safety: number of approaches, number of entering lanes, entry width, entry lane radius, flare width, circulating lane radius, and number of circulating lanes. Figure 2.4 illustrates the explanatory variables that were used. It is worth mentioning that this study has some limitations such as the use of a small sample size.

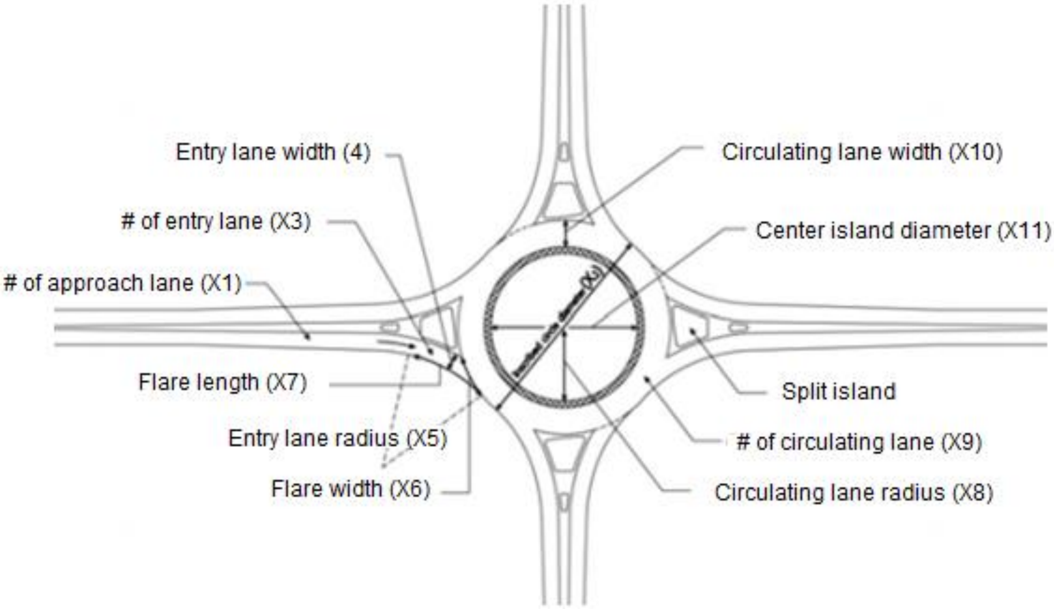

Figure 2.4 Geometric elements of roundabout designs
Source: Kim and Choi (2013)

Turner et al. (2009) used Poisson and NB regression models to examine the factors affecting road safety in New Zealand. Crash data were collected, using a five-year period extending between 2001 and 2005, from 104 roundabouts. The findings showed that multiple entry lanes were associated with greater crash frequency (66% more) than single entry lane roundabouts. Other studies confirmed this finding (Robinson et al. 2000; Mandavilli et al. 2009; Šenk & Ambros 2011). Šenk and Ambros (2011) developed a crash prediction model (CPM) using log-linear Poisson distribution to study the factors affecting road safety at 90 roundabouts in the Czech Republic. Data on roundabout elements and crash history were collected during a period between 2009 and 2010. The study investigated the effects of five explanatory variables on the roundabouts' safety performance including vehicle speed, number of lanes, traffic volume, driver behaviour, and weather conditions. Based on the model results, the study concluded that two lane roundabouts performed significantly worse than one lane roundabouts for the specific study conditions as outlined in that research. In addition, the explanatory variables such as driver behaviour and weather conditions had a slight negative impact on safety.

Daniels et al. (2011) investigated the relationship between traffic and geometric design elements and their corresponding safety impacts. The study involved developing Poisson and Gamma models to analyse crash data in Flanders, Belgium. The data used in this study were based on a previously composed dataset of 90 roundabouts (Daniels et al. 2010), which were extended to 148 roundabouts. The results of the investigation found that three-legged roundabouts were more dangerous compared to roundabouts with four or more legs. In addition, the overall crash frequency was not significantly affected by the central island diameter. Austroads (2015) stated that more than four-legs for multi-lane roundabouts should preferably be avoided as this could create increased conflicts for exiting traffic.

Shadpour (2012) developed CPMs based on the data collected from 48 roundabouts in Waterloo, Canada during the period between 2004 and 2010. The author investigated the impacts of traffic volume, number of legs, number of lanes at specific approach, duration of roundabout operation, and central island structure. The results revealed that when traffic volume grows by 7.3%, the frequency of road crashes will increase by 9.4%. The frequency of road crashes in four-legged roundabouts was found to be 44% higher than three-legged roundabouts. Two-lane roundabouts were found to have 54% higher road crashes than single-lane roundabouts. However, the central island structure and duration of roundabout operation were found not to be statistically significant.

Montella (2011) carried out the analysis and the site inspections carried out by a team of specialists who had a background related to road safety engineering to investigate the relationships between various contributory factors and roundabout crashes. A total of 62 different contributory factors were identified from 15 roundabouts located in Naples, Italy during the period 2003-2008. The study concluded that among all the contributory factors that were investigated, the geometric design factors were the most frequently occurring. In almost 60% of all recorded crashes, at least one geometric factor was found. The main geometric data used were as follows: inscribed circle diameter, circulating roadway width, radius of deflection, entry width, entry radius, entry angle, exit width, exit radius, and deviation angle. Figure 2.5 illustrate the main geometric factors used in this study. A recent study performed by Montella (2018) showed that the geometric design elements such as entry radius, radius of deflection, and deviation angle can be employed to control high speeds entering at roundabouts.

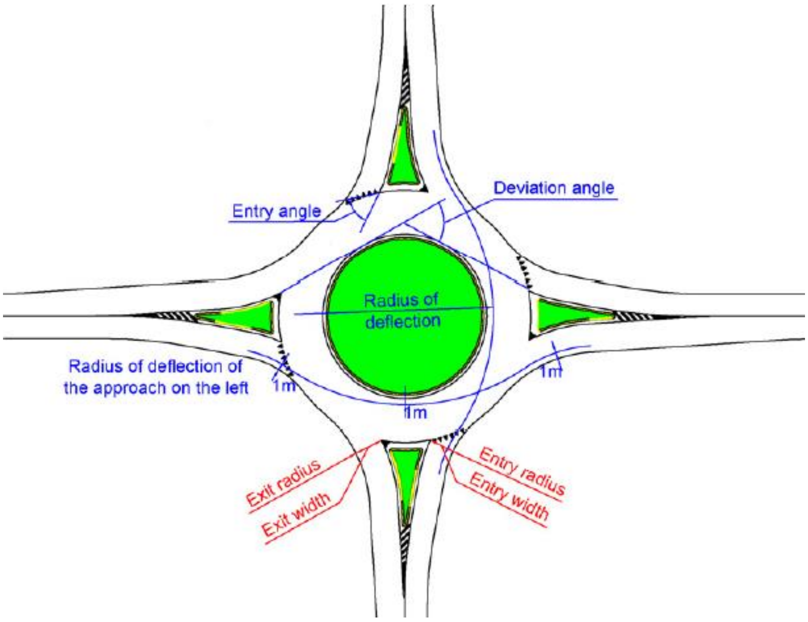

Figure 2.5 Main roundabout geometric design factors
Source: Montella (2011)

From the aforementioned studies, it can be noted that the effect of geometric elements on crashes sometimes may be inconsistent. This is attributed to the fact that the results of different studies are based on a variety of modelling techniques, different crash-severity levels, and different data sources and locations. For example, some studies (Rodegerdts et al. 2010; Kamla et al. 2016) found that the crash frequency tended to increase as the inscribed circle diameter increased while another study (Kim & Choi 2013) indicated the opposite effect of the inscribed circle diameter. However, the previous studies revealed that the explanatory variables related to traffic and geometric elements have a significant influence on roundabout-related crashes. There are several studies where a generalised linear model (GLM) approach, such as Poisson and negative binomial models, has been utilized as a means to examine road safety at roundabouts (Turner et al. 2009; Sacchi et al. 2011; Farag & Hashim 2017).

2.3.3 Roadway Segments

Several crash prediction models (CPMs) were developed to investigate the relationship between safety at roadway segments and influencing factors. In their analysis on roadway segments, Turner et al. (2012) modelled a relationship between road crashes and road geometry, traffic volumes, roadside hazards, road surfacing, cross-section and driveway density for two lane rural roads in New Zealand. The results indicated that CPMs provide a good method to help understand how safety is affected by these variables. The research also indicated that CPMs can be used to identify which actions are best to reduce the number of crashes.

Greibe (2003) developed CPMs for road segments (links) and urban intersections in Denmark using a Poisson distribution. To obtain the roadway segments, the data was collected from 142 km of urban roadway, divided into 314 homogeneous segments with average segment length of approximately 450 m. The impacts of the following variables on road safety were investigated: traffic volume, speed limit, length of roadway segment, one/two-way traffic, roadway width, number of lanes, and number of minor crossings/side roads. The study found that the explanatory variables which describe the speed limit, road environment, parking facilities, number of minor side roads and number of exits per km proved to be important and significant explanatory variables for estimating the number of crashes in roadway segments.

Cafiso et al. (2010) attempted to define CPMs for two-lane rural road sections based on a combination of geometry, exposure, context and consistency variables related to the road safety performance. The roads considered were two-lane local rural roads, with a five-year crash analysis period to compensate for the low traffic flow and crash frequencies expected on local roads. The models suggested were also based on the Generalized Linear Modelling approach (GLM), assuming a NB distribution error structure. Three of the examined models were considered appropriate, based on practical considerations, statistical significance, and goodness of fit indicators. The main explanatory variables included in the selected models: traffic volume and length of segment (exposure variables); driveway density and roadside hazard rating (context variables); curvature ratio and operating speed (geometric and operational variables); and standard deviation of the operating speed (consistency variables).

Abdel-Aty and Radwan (2000) employed the NB modelling technique to estimate crash frequency on rural roadway segments in Central Florida. Crash data was collected from 566 homogenous roadway segments over three years study period from 1992 to 1994. The results showed that high traffic volume, additional number of lanes, reduced lane width, reduced shoulder width, reduced median width, and speeding increased the probability of crash frequency. Moreover, among those explanatory variables, traffic volume, lane width, and number of lanes are the most critical factors to affect the safety. The influence of the number of roadway lanes on safety was also examined by Noland and Oh (2004) and Mussa and Chimba (2006). The authors concluded that additional lanes are associated with more crash risk. Mussa and Chimba (2006) employed a zero-inflated negative binomial model to investigate the impacts the number of lanes had on roadway safety. The results revealed that roadways with 6-lane or more had higher crash risk compared to 4-lane roadways based on the study conditions. On the contrary, Milton and Mannering (1998); Garber and Ehrhart (2000); Kononov et al. (2008) pointed out that the roadway safety improved as the number of lanes increased.

Mustakim and Fujita (2011) developed CPMs for rural roadways in Malaysia using data collected during an 8-year period between 2000 and 2007. Multiple non-linear regression models were applied to investigate the relationship between road safety and roadway traffic and geometric elements. The results indicated that the absence of traffic lights, the increase in speed and traffic volume (which results in a reduced time gap) are the major contributors in increasing the crash risk on rural roadway segments.

Ackaah and Salifu (2011) developed CPMs based on a NB error structure to study road crashes on rural highway segments in Ghana from 2005-2007. Data was collected from 76 segments with each segment ranging between 0.8 and 6.7 km. The study identified the main explanatory variables that significantly influenced the crash risk as traffic volume, length of roadway segment, intersection density (i.e., number of intersections per unit length of roadway segment), and type of terrain. On the other hand, horizontal and vertical curves, posted speed limit, roadway width, shoulder width, and road marking were not found to be statistically significant risk factors for road crashes. The results indicated that increased segment length, traffic density, and intersection density tended to increase the probability of crash risk. In addition, level terrains were found to be associated with more crashes when compared with mountainous and rolling terrains.

Dissanayake and Roy (2014) used a binary logistic regression model to identify the main factors that affected road crash severity. In this study, the data were collected between 2004 and 2008 in Kansas City, USA. The results concluded that some of the significant variables which affect the probability of road crashes are asphalt type road surface, speed, alcohol involvement, older driver, medical condition of the driver, daylight, type of vehicles, and fixed object types such as trees. The same method was used by Lee and Mannering (1999) to investigate the relationships between roadway geometric characteristics and crash frequency. The study found that the crash probability can be significantly reduced by increased median, lane, and shoulder widths. In other work, Hadi et al. (1995) developed several CPMs for both urban and rural roadway segments in Florida between 1988 and 1991. Poisson and NB models were used in this study. The findings showed that, depending on the highway type, increasing lane width, inside shoulder width, outside shoulder width, and median width are effective in increasing road safety as shown in Figure 2.6.

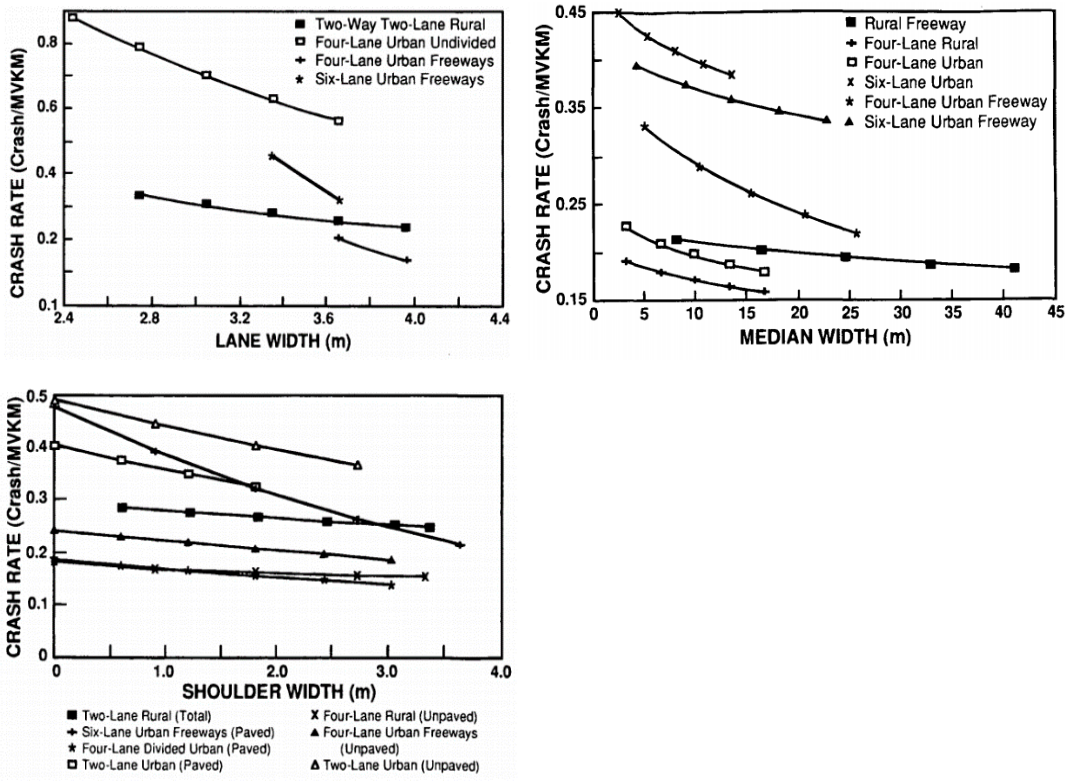

Figure 2.6 Effect of lane, shoulder, and median widths on roadway segment crashes
Source: Hadi et al. (1995)

A zero-inflated-Poisson model was applied by Qin et al. (2004) to develop CPMs for various crash types including: single-vehicle collision, multi-vehicle collision (same direction), multi-vehicle collision (opposite direction), and multi-vehicle collision (intersecting). Data on crash history and roadway characteristics were collected on the study roadway segments in Michigan State during the four year period between 1994 and 1997. In this study, the average length of roadway segments was approximately 1 km. Crash models, based on crash types, were developed as a function of traffic volume (AADT), length of roadway segment, speed limit, lane width, and shoulder width. The findings indicated that the relationship between crash frequency and traffic volume is non-linear and varies by crash type.

Vayalamkuzhi and Amirthalingam (2016) analysed the impact of roadway geometric characteristics on road safety in India using both Poisson and NB models. The study was performed on a four-lane divided urban roadways for a 4-year period, from 2009 to 2012. The results indicated that operating speed, median strip opening, minor road access point, and horizontal curvatures are significant in influencing the total crash frequency.

Cafiso et al. (2018) investigated the influence of roadway segmentation techniques on the performance of CPMs, in terms of goodness-of-fit (GOF) and the independent variables that could be modelled. Four different segmentation techniques were examined: (1) homogeneous segments with respect to traffic volume and curvature (suggested by Highway Safety Manual (AASHTO 2010)), (2) segments with constant length, (3) segments containing two curves and two tangents, and (4) segments with constant geometric and traffic variables within each segment. The study revealed that the developed models using segmentation techniques (2) and (3) showed the best results. Miaou and Lum (1993) stated that including short roadway segments less than 0.08 km could lead to bias in the estimated models, especially when using linear models. Similarly, Ogle et al. (2011) concluded that short roadway segments (less than 0.16 km) lead to uncertain results in road crash analysis.

Strathman et al. (2001) developed a statistical model to investigate the safety performance of urban roadway segments in Oregon State. The roadway segmentation process for this study used homogeneous roadway segments with respect to traffic volume, traffic control, and geometric characteristics which resulted in variable lengths being adopted. A number of roadway design elements were found to be statistically significant in various models, including the vertical grade, number of lanes, median type, surface type, lane width, shoulder width, curve characteristics, and turning lanes.

Overall, the previous studies on the safety of roadway segments focused on modelling the relationship between crash frequency and traffic and roadway geometric elements. The studies found that explanatory variables such as roadway segment length and traffic volume (AADT) are the most often used in crash modelling. Moreover, the studies show that several regression techniques were used by researchers to model crashes. The most commonly used techniques include Poisson and negative binomial. The following sub-section provides detailed insights into the various statistical modelling techniques used in previous studies.

2.4 Modelling Techniques

Several regression techniques have historically been used for crash prediction models. These include random effects, multiple logistic, multiple linear, Poisson distribution, negative binomial (or Poisson gamma), zero-inflated Poisson distribution, and zero-inflated negative binomial models. These techniques are now reviewed in order to derive the most appropriate for assessing the safety of road networks.

Random effect

The random effect technique assists in controlling the variations in crash frequencies among different locations, assuming that road crash data is hierarchical in nature. The hierarchy in road crash data is proposed as follows: the lowest level of the hierarchy represent the crashes themselves, while the type of location on the road network at which the crash occurred represents the higher level hierarchy. In this type of model, the main assumption is that association may exist among crashes occurring at the same location, so these crashes may share unobserved or unrecorded characteristics related to the location. These unobserved characteristics might include low pavement friction, poor pavement condition, or poor reflectivity of road signs (Chin & Quddus 2003; Kim et al. 2007). The results from this technique may not be transferable to other data sets because the results are observation specific (Lord & Mannering 2010).

Multiple logistic regression

The multiple logistic regression technique is used to analyse the relationship between a set of explanatory variables and a binary crash outcome (Agresti 2002; Yan et al. 2005; Nambuusi et al. 2008; Dissanayake & Roy 2014). For example, this technique can be applied when the crash severity representation is in a binary outcome form such as a fatal or non-fatal crash. This technique is also suitable to investigate the effect of a specific variable while controlling other variables.

Multiple linear regression

There are many studies in which crash outcomes are continuous (e.g., number of total crashes). In such cases, multiple linear regression analysis which describes relationships between continuous outcomes and explanatory variables are more credible (Arndt & Troutbeck 1998; Kutner et al. 2005). Although multiple linear regression models are used widely in road crash studies, they have limitations in describing adequately the random, non-negative, discrete, and typically sporadic events, which are all characteristics of road crashes (Chin & Quddus 2003; Kim et al. 2005; Montella et al. 2008; Ackaah & Salifu 2011; Vayalamkuzhi & Amirthalingam 2016; Claros et al. 2017; Farag & Hashim 2017).

Poisson distribution

Since crash occurrences are unavoidable, discrete and more likely random events, the family of Poisson regression techniques appears to be more appropriate than multiple linear regression models. However, Abdel-Aty and Radwan (2000) stated that Poisson models have some limitations. One of these limitations is that the mean must equal the variance of the crash number (dependent variable). In most crash data, the variance of the crash number exceeds the mean and, in such a case, the data would be over-dispersed.

Negative binomial

To solve the limitation of over-dispersion in Poisson regression technique, some authors (Chin & Quddus 2003; Lord & Mannering 2010; Gargoum & El-Basyouny 2016; Moghaddam et al. 2017) recommend using other methods. An alternative is the use of negative binomial regression which does not require the equal mean and variance assumption. Basically there is a need to employ techniques which can sufficiently describe discrete, random, and non-negative crash events and such techniques will include Poisson regression and negative binomial regression (Poisson when the data is not over-dispersed and negative binomial when it is).

Zero-inflated

The zero-inflated or zero-altered probability model has been applied to deal with the excess zeros (i.e., no crashes) that commonly arise in road crash data (Miaou 1994; Kumara & Chin 2003; Qin et al. 2004; Mussa & Chimba 2006; Washington et al. 2010). This type of model assumes either the negative binomial or Poisson distribution of the outcome data based on the presence of over-dispersion or not. Miaou (1994) studied the statistical performance of negative binomial, Poisson distribution, and zero-inflated Poisson models in investigating the relationship between truck crashes and the geometric design of roadway segments. The Miaou concluded that the Poisson distribution model is a suitable model for developing the relationship when the variance and mean of the crash frequencies are approximately equal. If the over-dispersion is found to be high, the negative binomial model and zero-inflated Poisson model were found to be more appropriate for use. On the whole, the zero-inflated Poisson model seems a justified model when crash data exhibit a high frequency of zero-crash results. Despite zero-inflated models being widely applied by the researchers (Shankar et al. 1997; Lee & Mannering 2002; Kumara & Chin 2003; Hu et al. 2011; Kibar et al. 2018) to investigate the safety performance of situations where the observed crash data is characterized by a high zero density, other researchers such as (Lord, Manar, et al. 2005; Lord et al. 2007; Dong et al. 2017) have criticized this type of application in roadway safety investigations. Lord et al. (2007) stated that since the zero-crash state has a long-term mean equal to zero, the zero-inflated models cannot correctly reflect the crash data generating process.

Table 2.1 shows a summary of regression models used in previous studies for analysing crash data. The review carried out and detailed in Table 2.1 suggests that the best models for the proposed research are the negative binomial and Poisson distribution.

Table 2.1 Characteristics of models used for analysing crash-frequency data

Model Type	Studies used or discussed this type	Advantages	Disadvantages
Random Effects	Chin and Quddus (2003); Nambuusi et al. (2008); Lord and Mannering (2010)	Handle spatial correlation[1]	Results from this technique may not be transferable to other data sets because the results are observation specific.
Multiple Logistic	Kim et al. (2005); Kutner et al. (2005); Montella et al. (2008); Chen et al. (2012); Dissanayake and Roy (2014)	Suitable to study the effect of one variable while controlling for other variables[2]	Applied to analyze binary crash outcomes (an event happened or not)
Multiple Linear	Arndt and Troutbeck (1998); Chin and Quddus (2003); Kim et al. (2005); Mustakim and Fujita (2011)	Easy to estimate crash number	Unable to describe adequately the random, non-negative, discrete, and typically sporadic events.
Poisson Distribution	Abdel-Aty and Radwan (2000); Bauer and Harwood (2000); Chin and Quddus (2003); Greibe (2003); Lord and Mannering (2010)	Handle with unavoidable discrete and more likely random events	Cannot handle over- and under-dispersion (the mean must equal the variance of crash number).
Negative Binomial (NB)	Abdel-Aty and Radwan (2000); Bauer and Harwood (2000); Usman et al. (2010); Ackaah and Salifu (2011)	Does not require the equal mean and variance assumption, able to describe adequately the random, non-negative, discrete, and typically sporadic events.	Cannot handle with small sample sizes.
Zero-inflated Poisson and NB	Miaou (1994); Lord, Washington, et al. (2005); Lord et al. (2007); Basu and Saha (2017); Dong et al. (2017)	Handle datasets that have excess zero-crash frequencies.	Zero-inflated NB can be negatively affected by a low sample-mean and small sample-size bias.

[1] Crashes occurring at the same location may share unobserved or unrecorded characteristics related to the location

[2] In logistic regression the coefficients derived from the model (e.g., β_1) indicate the change in the expected log odds relative to a one unit change in X_1, holding all other predictors constant

2.5 Identification of High Crash Locations

Identification of high frequency crash locations, variously known as black spots, high-risk locations, hazardous road locations (HRLs), hotspots, or crash-prone situations, is normally considered as the first step in a road crash reduction process. Elvik (2008b) defined black spots as any locations that have a higher predicted number of road crashes than normal when compared to other similar locations. In general, the identification of black spots is divided into two main approaches based on the type of crash data used in the identification process. The first approach depends on historical crash data. In this approach, the black spot is defined as the location which has a higher than average crash number, crash frequency (crash per year or crash per kilometre) or crash rate (crash per vehicle). The second approach is a model-based definition which depends on analysing each site location by applying statistical models to identify black spots (AASHTO 2010). According to Hauer and Kononov et al. (2002) the identification of hazardous locations signifies a list of spots being prioritised for further research and engineering investigation which can distinguish road crash patterns, effective variables, and potential countermeasures. In those processes, cost-effective remedial projects are often selected to obtain the optimal outcomes from limited resources.

Šenk et al. (2012) investigated the possibility of using crash models for the identification of black spots. The geometric and traffic characteristics of secondary rural roads in South Moravia were used in this study. The GLM was employed to determine the predicted number of crashes for individual types of road segments. A critical road link (segment) is defined as a link where the recorded number of crashes significantly exceeds the expected number of crashes on roads with similar traffic and geometric characteristics. The results indicated the possibility of using this method as an effective tool for road safety management. Miranda-Moreno et al. (2005) investigated the performance of three statistical models: Poisson lognormal, heterogeneous negative binomial, and traditional negative binomial model for ranking locations for road safety improvement. The authors compared these models for the identification of black spots based on the performance and practical implications. This study concluded that the choice of model assumptions and ranking criteria can lead to different lists of black spots. In other work, Mustakim and Fujita (2011) used the crash data from rural roadways from the year 2004 to 2007, to rank the black spots in Malaysia based on a crash point weightage formula as follows:

$$CPW = X_1(0.6) + X_2(0.3) + X_3(0.8) + X_4(0.2) \qquad (2.1)$$

Where: X_1 is the number of fatal, X_2 is the number of serious injury, X_3 is the number of slight injuries, and X_4 is the number of damage only. This study applied the multiple linear regression method for developing a model which relates crash point weightage to rank the black spot locations.

Sjölinder and Ek (2001) used crash frequency to identify the black spots where a road section is considered to be a black spot, from the crash frequency point of view, and a location is considered a black spot if Aj > Ac, where:

$$A_c = F_{ave} + K_a \sqrt{\frac{F_{ave}}{L_j}} - \frac{0.5}{L_j} \qquad (2.2)$$

A_c is a critical value for crash frequency, A_j is a number of crashes on segment *j* during a certain time period, L_j is a length of segment *j*, F_{ave} is the average crash frequency for all segments, and K_a is a constant that is selected for the significance test.

Elvik (2007) stated that the best method to determine black spots is the expected crash frequency, not the recorded crashes. At the same time, the combination of the recorded crash number and the model estimate for that site is the best method to estimate the expected crash frequency. A suitable technique to do this is to apply the empirical Bayes (EB) approach. Zou et al. (2013) examined the ability to use the Sichel (SI) model in calculating empirical Bayes (EB) estimates. In order to accomplish the objective of their study, the SI model and NB model were developed using the road crash data collected at 4-lane undivided rural highways in Texas. Results found that the selection of a crash prediction model (i.e., the NB or SI model) will affect the value of the weighting adjustment factor used for calculating the EB outputs, and the determination of black spots by using the EB method can be different when the SI model is used. According to separate studies done by Hauer and Harwood et al. (2002); Elvik (2007) by calculating the weighted combination of the recorded and predicted crashes number, the EB approach is able to provide an expected crash frequency for a specific roadway segment or intersection. Using the EB approach, the expected crashes for an entity can be estimated as follows:

$$\text{Estimate of the expected crashes for an entity} = weight \times \text{predicted crashes on the entity} + (1 - weight) \times \text{observed crashes on the entity} \qquad (2.3)$$

The value of *weight* varies from 0.0 to 1.0 and is obtained as follows:

$$weight = 1/(1 + K \times \text{ predicted crashes on the entity}) \qquad (2.4)$$

Where *K* represents the over-dispersion parameter of a crash prediction model (CPM). This parameter shows the amount of systematic variation in the crash frequencies which is not explained by the model. When the predicted model explains all systematic variation in the crash frequencies, the over-dispersion parameter will have a zero value (Elvik et al. 2017). In such case, the value of *weight* will be equal to 1.0.

Many researchers (Persaud et al. 1999; Saccomanno et al. 2001; Cheng & Washington 2005; Elvik 2008a; Montella 2010; Da Costa et al. 2015; Ghadi & Török 2017) evaluated the different black spot identification methods. The results showed the preference of the EB method over other methods. For instance, Cheng and Washington (2005) evaluated the three black spot identification methods of confidence interval, simple ranking and empirical Bayes (EB). In the confidence interval method, location j is considered as an unsafe location if the observed crash frequency N_j exceeds the crash frequency of a comparison (similar) location Z, with level of confidence σ which is typically 90%, 95%, or 99%. In the study, the characteristics of observed crash data have been employed to create simulated data distributions at hypothetical locations. The results showed that the EB approach significantly outperformed other methods. Similarly, Montella (2010) compared the performance of seven methods used in black spots identification. The following methods were compared: crash frequency, crash frequency of equivalent property damage only crashes, proportion method (based on crash type), crash rate, potential for improvement, EB estimate of total crash frequency, and EB estimate of severe crash frequency. To accomplish this comparison, five years (2001-2005) of crash data were collected in Italy. In the analysis period, a total of 2245 crashes including 728 severe crashes (fatal plus injury) were recorded. The study found that EB approach using total crash frequency performed better than the other methods. In addition, the EB approach was found to be the most reliable and consistent method for identifying priority improvement locations.

It is worth mentioning that the EB approach is employed to control regression-to-the-mean (RTM) bias by estimating a weighted average of the observed and predicted crashes (Hauer & Harwood et al. 2002; Persaud & Lyon 2007; Tegge et al. 2010; Abdel-Aty et al. 2014; Elvik et al. 2017). According to Persaud and Lyon (2007), the RTM phenomenon occurs due to the tendency of sites (e.g., roadway segments) that have a high crash frequency in a particular year to regress to a lower crash frequency the following year. In other words, consider a site with a high crash frequency or rate during a particular year. The random nature of crashes occurring indicates that it is likely that the crash frequency will decrease next year to follow the long-term mean value, even without treatment and without a change in traffic conditions. Elvik et al.

(2017) stated that the EB approach enables researchers to control RTM bias, long-term trends, and exogenous changes in traffic volume. In summary, the EB approach can be accepted as the most reliable and consistent approach to perform the proposed research for identifying priority improvement locations.

2.6 Crash Modification Factors

Crash Modification Factor (CMF), also known as Crash Reduction Factor (CRF), provides a simple and quick arithmetic method to estimate crash reductions. This factor is used for evaluating the road safety impacts of several types of engineering improvements. Typically, this factor is calculated using before-and-after comparisons. The relationship between the CMFs and CRFs is defined as $CMF = 1 - CRF/100$ and $CMF = N_w/N_{w/o}$, where N_w is the expected crash frequencies with the improvement and $N_{w/o}$ is expected crash frequencies without the improvement (AASHTO 2010). CMFs are used with a road safety prediction model to estimate the expected crash frequencies for a specific site and/or to estimate the effect of a change in conditions on road safety. Bonneson and Lord (2005) indicated that CMFs usually range in value from 0.5 to 2.0, with a value of 1.0 indicating no effect on safety by the change in geometric design and traffic control feature. CMFs less than 1.0 indicate that the treatments reduced the predicted number of crashes and CMFs greater than 1.0 indicate that the treatments increased the predicted number of crashes.

The USA's Highway Safety Manual (HSM) Volume 3 Part D (AASHTO 2010) and other studies (Bonneson & Pratt 2009; Bahar 2010; Gross et al. 2010; Li et al. 2010; Persaud et al. 2010; Wang et al. 2017; Galgamuwa & Dissanayake 2018) used the observational Before-After (BA) studies and/or cross-sectional method for estimating safety effectiveness and developing the CMFs of specific roadway treatments.

2.6.1 Observational Before-After Studies

Observational Before-After (BA) studies involve estimating either the number of crashes or some other risk measure before and after a given treatment is implemented on either one or several sites (Gross et al. 2010). The CMFs in the HSM were estimated using observational BA studies that account for the regression-to-the-mean (RTM) bias. Generally, there are five approaches that can be employed to implement observational BA studies; (1) Comparison Group (CG) approach, (2) Yoked

Comparison (YC) approach, (3) Naïve (simple) approach, (4) Empirical Bayes (EB) approach, and (5) Full Bayes (FB) approach (Hauer 1997; Harwood et al. 2003; Shen 2007; Lan et al. 2009; Persaud et al. 2010; Abdel-Aty et al. 2014; Park, Abdel-Aty & Lee et al. 2015; Elvik et al. 2017; Wang et al. 2017). Each of these approaches will be discussed in detail.

2.6.1.1 Comparison Group Approach

The CG approach employs a comparison group of non-treated sites to compensate for the external causative factors that may affect the change in the crash frequencies (Shen 2007; Mbatta 2011; Park 2015). In this approach, data of road crashes at the comparison group are incorporated to estimate the change in road crashes that would occur at the treated sites if the safety treatment had not been implemented. Mountain et al. (1992) reported that the accuracy of the CG approach increases as the similarity between treated sites and comparison sites increases. The CG approach is based on two basic assumptions (Shen 2007):

1. The factors that affect safety have changed in the same way from the before period to the after period (where treatment had been applied) on both treated sites and comparison sites; and
2. The changes in the various factors affect the safety of treated sites and comparison sites in the same manner.

Using this approach, the expected crash frequencies in the after period for the treated sites without performing of safety improvement, N_a, can be estimated as follows (Hauer 1997):

$$N_a = N_b \times R_c \tag{2.5}$$

Where, N_b is the recorded crash frequencies in the before period for the treated group and R_c is the ratio of after-to-before recorded crash frequencies at the comparison sites. The CMF can thus be estimated at a particular site as the ratio between the expected crash frequencies after the improvement was performed using Equation 2.5 and the recorded crash frequencies before the improvement was performed. Pendleton (1991) stated that the sample size of the comparison sites should be at least five times larger than the treated sites. Likewise, Hauer (1997) stated that the crash frequencies in the comparison sites should be large compared with the crash frequencies in the treated sites. Furthermore, the length of before-and-after periods for the treated sites and comparison sites should be the same. Figure 2.7 illustrates the conceptual outline employed by the CG approach. It should be noted that the CG approach does not take into account the naturally expected reduction in crash frequencies in the after period for treated sites with high crash rates (Hauer 1997; Park 2015). Thus, this approach does not account for the RTM bias that is associated with crash data.

Figure 2.7 Conceptual outline of the CG approach

2.6.1.2 Yoked Comparison Approach

The Yoked Comparison (YC) approach is a special case of the CG approach where a single treatment site is matched to each comparison site (i.e., one-to-one matching) on the basis of similar traffic and geometric conditions. Figure 2.8 illustrates the conceptual outline employed by the YC approach. According to Gross et al. (2010), the strengths and weaknesses of the YC approach are similar to those of the CG approach with a couple of exceptions. The main benefit of the YC approach, in relation to the CG approach, is that it does not require as much data. This is also, a weakness of the YC approach as it limits the amount of data for evaluating safety benefits. It should also be noted that this approach cannot deal with RTM bias.

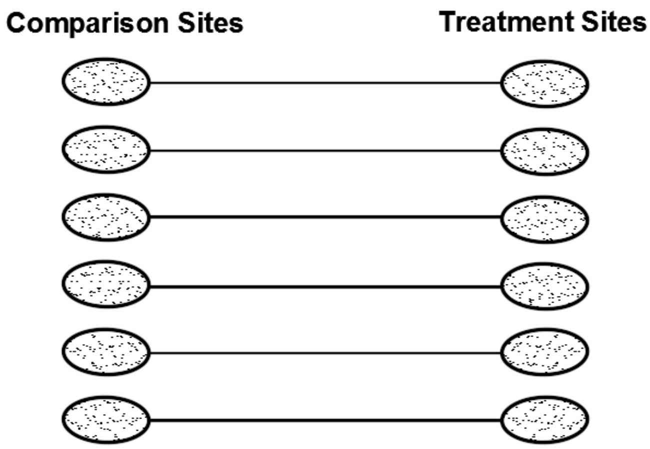

Figure 2.8 Conceptual outline of the YC approach

Harwood et al. (2003) evaluated the safety effectiveness of right-turn lane and left-turn lane improvements using Empirical Bayes (EB), Yoked Comparison (YC), and Comparison Group (CG) approaches. The authors recommended using YC and CG approaches only if the results of the EB approach are not statistically significant. This is because the YC and CG approaches cannot account for the RTM effect. In addition, the study showed that the CG approach results were more accurate than the YC approach results as the CG approach employs more than one comparison site for each treated site.

2.6.1.3 Naïve Approach

The main assumption of the naïve (simple) approach is that the crash frequencies before the treatment implementation will be expected (Abdel-Aty et al. 2014). In this approach, the expected crashes are calculated by using the ratio of road crashes to the number of years before treatment and converting that ratio to the expected after crashes using only the number of years after treatment (Persaud & Lyon 2007; Liu et al. 2011; Isebrands & Hallmark 2012). According to Gross et al. (2010) and Abdel-Aty et al. (2014) the naïve approach tends to over-estimate the effect of the treatment due to the RTM problem. In other work, Lan et al. (2009) found that the naïve approach incorrectly predicted a total reduction in crashes after a hypothetical treatment was performed without any effect. The reason that this is incorrect is due to RTM bias which is not accounted for in this approach.

2.6.1.4 Empirical Bayes Approach

The Empirical Bayes (EB) approach was introduced by Hauer (1997) and Hauer and Harwood et al. (2002) to estimate road safety. This approach increases the accuracy of estimation to address the main limitation of the CG and Naïve approaches by accounting for the RTM effect (Shen & Gan 2003; Saccomanno et al. 2007; Khan et al. 2015). In addition, the EB approach is better than the CG approach because it accounts for the effects of traffic volumes and time trends on crash occurrence and safety (Persaud & Lyon 2007). According to Ko et al. (2013) the EB approach estimates the safety at treated sites based on comparison with reference sites (intersections or roadways) with similar features and crash history. Figure 2.9 illustrates the conceptual outline employed by the EB approach. The expected crash frequencies at a treated site can be estimated using Equation 2.3 based on the reference sites. Moreover, as mentioned earlier, the EB approach can be also be used to identify black spot locations.

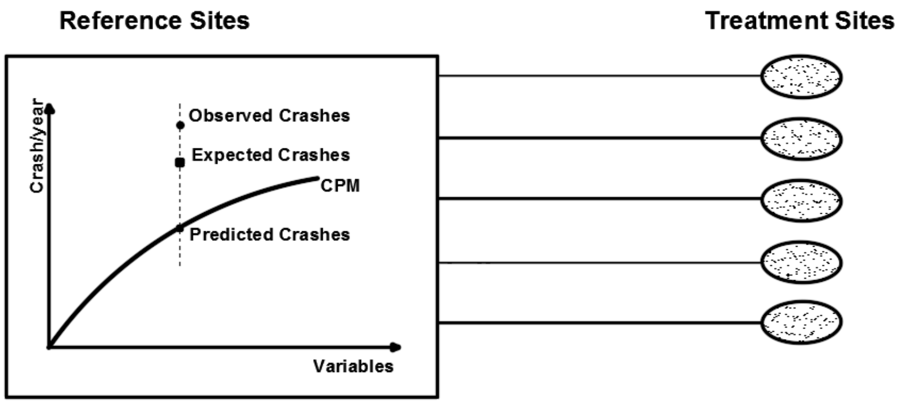

Figure 2.9 Conceptual outline of the EB approach

Persaud and Lyon (2007) compared CG and EB approaches in estimating safety benefits at treated sites had treatment not been implemented. Data of crash frequencies were collected from 1669 stop control intersections during 6-year (1994-1999) in California. The dataset was divided into two groups. The first group included the crashes which occurred between 1994 and 1996 and the second group included crashes between 1997 and 1999. The expected crash frequencies for the after period (1997-1999) were estimated using both CG and EB methodologies and then compared with actual crashes in the after period. The results showed that the CG approach systematically overestimated the crash frequencies for sites, whereas the EB approach appeared to be unbiased in that it sometimes under-estimated and sometimes over-estimated the crash frequencies for the sites. Figure 2.10 shows the superiority of the EB approach based on cumulative residuals. In the same study, a comparison between naïve and EB approaches was also performed. To perform this comparison, data were incorporated from previous studies such as Persaud et al. (1984); Hauer and Persaud (1987); Persaud et al. (1997); Persaud et al. (2001); Persaud et al. (2004); Lyon et al. (2005); Persaud et al. (2005). The expected after crashes without treatment were estimated using the naïve and EB methodologies that were described earlier. The results showed substantial differences between the naïve and EB estimated in terms of actual reduction.

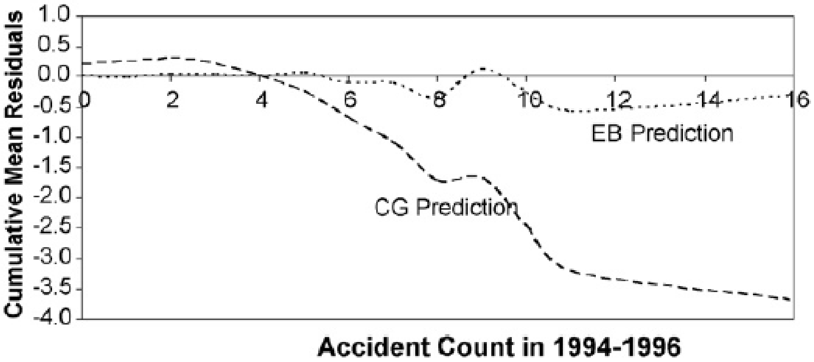

Figure 2.10 Cumulative residuals based on the crash frequencies during 1994-1996
Source: Persaud and Lyon (2007)

2.6.1.5 Full Bayes Approach

The Full Bayes (FB) approach is similar to the Empirical Bayes (EB) in the use of non-treated reference sites to make inferences and to account for possible influences unrelated to the treatment. Lan et al. (2009) stated that the main difference between the FB and EB approaches is that the predicted crash frequencies without treatment were obtained by the CPM that was estimated using data from both before period of treated sites and reference sites. On the other hand, for the EB approach, the CPM was estimated using only data from reference sites.

More recently, researchers have introduced the use of the FB approach to evaluate the impact of safety treatments (Lan et al. 2009; El-Basyouny & Sayed 2010; Persaud et al. 2010; Sacchi & Sayed 2015). This approach has shown several advantages over other approaches, including the ability to account for all uncertainties in the data used, requiring less data, providing more flexibility in selecting crash frequency distributions, providing more detailed causal inferences, and the ability to consider the effect of one site's proximity to other sites (i.e., spatial correlation) in the model formulation. Sacchi and Sayed (2015) compared the results of naïve, EB, and FB approaches in estimating the treatment effectiveness. Two types of the hypothetical treatment sites selection were adopted to perform the analysis: random selection to reduce the selection bias effect; and non-random selection by selecting sites with abnormal crash frequency (black spots). For sites selected randomly, the results revealed that all approaches provide reasonable results. In addition, the results revealed that the FB approach showed better performance than the naïve and EB approaches on the basis of non-random sites selection. It is worth noting that the complexity of the FB approach makes the EB approach more attractive for researchers to use (Persaud et al. 2010; Khan et al. 2015).

2.6.2 Cross-Sectional Method

There are some limitations associated with observational BA studies. For example, treatment date should be known to determine the evaluation periods and several years have to elapse after implementing any treatment to collect a reasonable amount data. It is also difficult to distinguish safety effects when implementing more than one treatment at a site. In such cases, the cross-sectional method can be employed to estimate CMFs because of its simplified approach for obtaining data compared to observational BA studies. According to Gross (2006), the cross-section method is conducted in the case where an observational BA study is impractical. AASHTO (2010) also indicated that the cross-sectional method might be appropriate when implementing a treatment on a roadway where crash data is missing or cannot be obtained. This method is used when comparing the road safety performance of a site with certain specific features to another site without these features (Li et al. 2010).

As a part of the cross-sectional method, the crash modification function (CMFunction) method has been employed recently to derive CMFs at a specific site. The CMFunction method uses the coefficients of prediction models (Lord & Bonneson 2007; Gross et al. 2010; Park et al. 2014; Sacchi et al. 2014; Lee et al. 2015; Park, Abdel-Aty & Lee et al. 2015; Wood et al. 2015) to estimate the safety benefits after improvements. Wood et al. (2015) compared the CMFs obtained from observational BA studies (using the EB approach) and the cross-sectional method (using the regression approach). The study revealed that the cross-sectional method appears to yield results consistent with the EB approach results. Therefore using the cross-sectional method will yield a reasonable result where data for after treatments are not available. Likewise, Sacchi et al. (2014) and Park, Abdel-Aty & Lee et al. (2015) proposed using CMFunctions based on a cross-sectional approach to identify the relationship between safety effects and roadway characteristics.

Sacchi et al. (2014) indicated that estimation of CMF as a single value may not be adequate to represent how safety treatment affects crash frequency over time. Therefore, the authors developed CMFunctions which incorporate the variation in safety effectiveness of treatment over time. Elvik (2009) developed a framework to evaluate CMFunction for the same treatment type on the basis of meta-analysis for several studies. Elvik estimated CMFunction for installation of a bypass road and conversion of a signalised intersection to a roundabout on the basis of population changes. The author found that CMF values increased with the population for both treatments. However, the author recommended using a fairly large sample size to develop more accurate CMFunctions.

In summary, Table 2.2 provides a listing of methods used to estimate CMF along with their advantages and disadvantages.

Table 2.2 Summary of methods used for estimating crash modification factors

Method Type	Advantages	Disadvantages	Note
Comparison Group (CG)	Control the effects of external causal factors.	Does not account for RTM bias; difficulty to find an adequate number of similar sites without treatment.	Produces more accurate estimates than a naive comparison method.
Yoked Comparison (YC)	Simplicity of applying, no need for a large number of reference sites.	Does not account for RTM bias; limits the amount of data for evaluating safety benefits; difficulty dealing with zero crash frequency.	A single treatment site is matched to each comparison site.
Naïve Comparison	Simplicity of applying.	Does not account for RTM bias; over-estimate the effect of the treatment; not control the effects of external causal factors.	The crash frequencies before the treatment implementation would be expected.
Empirical Bayes (EB)	Mitigating the RTM bias; no need for a large number of reference sites.	Difficult to collect a reasonable data	Produces more accurate estimates than a CG and naive comparison method.
Full Bayes(FB)	Mitigating the RTM bias; ability to account all uncertainties in the data used; no need for a large number of reference sites; capable of accounting for the temporal and spatial variations.	Complexity of applying; difficult to collect a reasonable data.	Can be used as complex alternative to the EB approach.
Cross-Sectional	Mitigating the RTM bias, accounts the variation in safety effectiveness of treatment over time.	It does not take into account the effects of elements that are not included in the analysis; sufficient sample size is especially required when large explanatory variables are included in the developed model.	The accuracy is affected by how closely a developed model expresses the relationship between explanatory variables and crash frequency.

2.6.3 Documented CMFs based on Treatment Types

Several types of treatment can be identified and quantified using different methods to propose the best treatments for road safety improvement (Zegeer & Deacon 1987; Strathman et al. 2001; Lord & Bonneson 2007; Bonneson & Pratt 2009; Li et al. 2010; Park & Abdel-Aty 2016; Wu & Lord 2016; Wang et al. 2017; Choi et al. 2018; Galgamuwa & Dissanayake 2018). However, the studies showed that while some of the treatments had a positive impact, other treatments had a negative impact on road safety.

Strathman et al. (2001) studied the statistical relationship between the number of crashes and roadway geometric features by developing CMFs for freeway segments in Oregon, USA using the cross-sectional method. The study found that the number of lanes, surface type, curve characteristics, median type, vertical grade, turning lanes, lane width and shoulder width were statistically associated with crash occurrences. For example, the study showed that for each 0.3m (1.0 foot) of right shoulder width added to a freeway segment, the crash number decreased by 4.0 percent. Similarly, Bonneson and Pratt (2009) employed the cross-sectional method to investigate the relationship between different geometric design components and their corresponding safety effects in Texas State. The results showed that reduction of shoulder width from 3.0m (10 feet) to 2.4m (8 feet) was associated with 3.0 percent more crash frequencies. Moreover, when the median width reduced from 19.2m (64 feet) to 14.4m (48 feet), the crash frequencies were increased by 4.1 percent. Likewise, several researchers (Hadi et al. 1995; Miaou 1996; Bauer et al. 2004) have studied the effects of treatments on road safety especially the effect of widening the shoulder.

Harwood et al. (2003) evaluated the road safety effects of adding right and left turn lanes at urban intersections using observational BA studies. Data were collected from 280 intersections including geometric design, traffic volume, traffic control, and traffic crash data. The results revealed a 33 percent reduction in the number of crashes when adding a left turn lane on a major road approach at 3-legged intersections and 27 percent at 4-legged intersections. In addition, a reduction by 5 percent was found when adding a right-turn lane on a major approach at a stop controlled intersection. In other work, Hauer (1988) concluded that adding a left turn lane at intersections, and combining this treatment with the installation of kerbs, will reduce road crashes by 60, 65, and 70 percent in rural, suburban, and urban intersections, respectively. It was also concluded that adding road marking on this lane will reduce road crashes by 15, 30, and 50 percent in urban, suburban, and rural intersections, respectively.

Wu and Lord (2016) estimated the CMFs for lane and shoulder widths using a regression approach in the cross-sectional method. A total of 1492 roadway segments were identified and included in the analysis. The results showed that the CMF for lane and shoulder width was 0.73 and 0.77, respectively. Similarly, Lord and Bonneson

(2007) used the cross-sectional method to estimate the values of CMF for frontage rural roads in central Texas. Data on traffic volume, geometric elements, and crash history were collected from 141 roadway segments during a 5-year period between 1997 and 2001. The results showed that crash frequencies were reduced after increased lane and shoulder widths of roadway segments. Moreover, the edge road marking existence has a significant influence on the safety of rural two-way frontage roads. The same type of study was conducted by Li et al. (2010) to estimate the value of CMFs for frontage rural roads. This study concluded there is a non-linear relationship between road crash risk and changes in roadway geometric design characteristics (e.g., lane and shoulder widths). In terms of relating crash frequency to lane and shoulder widths, the work by Zegeer and Deacon (1987) is probably the most relevant, mainly because of the scope and the use of multivariate analysis such as observational BA studies. Large data were used to develop and calibrate crash models to estimate the effects of lane width on road safety. This work proposed guidance on the selection of road lane and shoulder widths to improve road safety. Recently, Galgamuwa and Dissanayake (2018) investigated the safety effectiveness after adding 0.6m (2 feet) paved shoulders on 2-lane rural undivided roadway segments in Kansas City using the cross-sectional method. The results showed that presence of 0.6m (2 feet) to paved shoulders was associated with a 12 to 18 percent reduction in all crashes and 6 to 16 percent reduction in both fatal and injury crashes.

Hauer and Bonneson (2006) employed the CG approach and cross-sectional method with an exponential model to identify the impact of the changes in posted speed limits on the road safety performance for urban roads. The study concluded that changing the posted speed from 112 to 101, 96 to 87, 80 to 72, 64 to 58, and 48 to 43 kilometres per hour reduced road crashes by 16, 16, 17, 18, and 19 percent respectively. Likewise, Kloeden et al. (2007) studied the effect on road safety after reducing the urban posted speed limit in South Australia from 60 to 50 kilometres per hour. Data were collected before and after the new posted speed limit was introduced in 52 randomly selected sites over a 4-year period between 2002 and 2005. The study showed that the mean posted speeds reduced by about 3.8 kilometres per hour on roads where the speed limit was reduced and there was a 23 percent reduction in crash frequency.

The observational BA study with the EB approach was used by Bauer et al. (2004) to study the safety performance after treatment was implemented on existing urban freeways. The treatments included adding an additional lane on an urban freeway by modifying a part of the shoulder to a travel lane or by narrowing travel lanes. Data on crashes were collected between 1991 and 2000, included 2-year data (1991-1992) before the study period and 7-year data (1993-2000) after the study period. The results indicated that increasing the number of lanes from four to five lanes resulted in increases of about 10 percent in crash frequency. In addition, increasing the number of lanes from five to six lanes resulted in slight increases in crash frequency. Likewise, using the EB approach, Sun et al. (2013) investigated a treatment on two different segments of urban undivided four-lane roadways in Louisiana. Statistical analysis three years before and three years after of crash data was used, excluding the project implementation period. This treatment included changing a four-lane roadway to a five-lane roadway by re-striping lane markings without increasing roadway width. The authors estimated expected CMFs on both roadway segments (0.45 and 0.43). This result clearly demonstrates it as an appropriate solution under constrained conditions.

Both observational BA studies with EB approach and cross-sectional method with CMFunction were used by Park, Abdel-Aty & Wang et al. (2015) to evaluate the safety performance after adding one through lane in each direction of urban roadway segments in Florida. A total of 138 treated roadway segments were identified and also 177 untreated roadway segments were identified as reference sites. The crash data were divided into two group: the three years (2003-2005) before period; and the four years (2009-2012) after period. The results showed that the conversion of roadways from four-lane to six-lane was predicted to achieve a 15 percent crash reduction.

Yanmaz-Tuzel and Ozbay (2010) estimated the crash reduction of different safety improvements that applied using observational BA with FB approach to urban roadways in New Jersey. The treatment types include installation of median barriers, increase in lane width, improvement of vertical and horizontal alignment, and installation of guard rails. The results found that the crash reduction for each treatment type was 14.3, 28.1, 23.1 and 28.6 percent, respectively. In other work, Meuleners et al. (2008) employed the observational BA with CG approach using all reported crashes at treated intersections for the period between 2000 and 2002 in Western Australia. The study identified certain treatment types that were successful in crash reduction such as installing the traffic signal (21.2% crash reduction), traffic island on approach (18.7% crash reduction), and left-turn slip (11.1% crash reduction). Moreover, and according to Thomas and Smith (2001); Gan et al. (2005); (Harkey et al. 2008), the percent of crash reduction after installing traffic signals at urban 4-legged intersections was 27, 22, and 23 percent, respectively for the three groups of researchers.

Elvik et al. (2009) used a meta-analysis of researches related to the installation of median and lane marking on urban roadway segments. The study concluded that the adding of median and lane marking resulted in a CMF value of 0.61 and 0.82 respectively. These outcomes may be related to the fact that crossing traffic can be reduced by adding median and lane marking. In general, and according to Lord and Bonneson (2006), CMFs can be used in roadway design processes and not just in the development of road countermeasures to treat existing roadways (e.g., intersections and segments). This can be achieved through the use of crash models to estimate a base value of the expected crash frequencies of the suggested facility and to then apply CMFs to evaluate the different alternative designs to identify the best design which provides the safest road.

2.6.4 CMFs for Multiple Treatments

There are a number of techniques proposed to estimate the value of combined CMFs for multiple treatments. Each of these techniques was discussed in detail.

HSM technique

The first of these techniques was adopted by the USA's HSM (AASHTO 2010) and this technique assumes that the road safety effect of each treatment is independent when CMFs for individual treatments are multiplied to estimate combined CMFs (Park et al. 2014; Wu & Lord 2016). Moreover, and according to Gross and Hamidi (2011), this assumption of independence gives a simple computational technique but lacks a consistent theoretical justification. For instance, adding a single lane and increasing shoulder width are treatments which both address crash frequency, and the implementation of one of these two treatments may have an influence on the safety effectiveness of the other.

Turner technique

The second technique was proposed by Turner (2011), where a specific weighted factor of 2/3 (two-thirds) is applied when estimating combined CMFs for two or more treatments. Turner developed this weighted factor after analysing different techniques to estimate combined CMFs for multiple safety treatments using data exclusively from

New Zealand. Comparison results of different techniques with CMFs for actual treatment combinations showed that all techniques have over-estimated the actual crash reductions. Therefore, based on this discovery, Turner suggested this factor. However, it is important to note that the validity of this technique for other regions needs verification.

Systematic reduction of subsequent CMFs technique

The third technique was proposed by the US State of Alabama (NCHRP 2008), which assumed that the safety effects of the less effective safety treatment are systematically reduced. This means that the full effect of the most effective safety treatment among all treatments is used and had an added benefit of additional treatments, i.e. less effective treatments (see Table 2.3). Moreover, this technique recognizes that additional safety treatments are likely to add an additional benefit, but not the full benefit due to the potential interrelationships between treatments.

Applying only the most effective CMF technique

The fourth technique applies only the most effective safety treatment, which is the lowest CMF among all treatments. This technique was proposed based on the survey performed by NCHRP (2008). The disadvantage of this technique is in underestimating the combined effect of safety treatments if the additional safety treatments provided additional benefit (Gross & Hamidi 2011; Park et al. 2014).

Bahar technique

The fifth technique was proposed by Bahar (2010), where a weighted average of CMF values for the same treatment from various studies was identified using meta-analysis. It is important to note that this technique was not developed to estimate the combined impact of different treatments. Instead, it was developed to combine CMF values estimates for the same treatment. However, and according to Gross and Hamidi (2011), this technique can be applied to combine CMF values for different treatments.

Table 2.3 summarizes the main existing techniques for combining individual CMFs. It is worth mentioning that there are very few studies have investigated the combined impacts of multiple treatments. In a study by Pitale et al. (2009), the CMF values for individual and combined treatments were estimated using before-after evaluation. The study found that the safety impacts of paving of aggregate shoulders, installing shoulder rumble strips, and widening paved shoulders from 0.6 to 1.2m (2 to 4 feet) on rural two-lane roadway segments are 16%, 15%, and 7% reductions in crash frequencies, respectively. The study also found that a 37% reduction in crash frequencies resulted from multiple (combined) treatments, consisting of paving shoulders + installing shoulder rumble strips. In other work, Bauer and Harwood (2013) investigated the safety impact of the combination of percent grade (vertical alignment) and horizontal curvature on rural two-lane highways in Washington State. CPMs of five types of vertical and horizontal alignment combinations for severe crashes and property damage only crashes were developed using crash history from 2003 to 2008. In this study, CMFs representing safety performance were estimated as the ratio of the predicted crashes for a given grade and horizontal curve combination to the predicted crashes for the level tangent (grade<1%) that defined a base condition.

Park et al. (2014) examined the existing combining techniques, and the results showed that the technique adopted by HSM and the fourth technique (most effective safety treatment technique) were close to the actual values of CMF. Similarly, Park and Abdel-Aty (2017) evaluated the performance of several existing techniques and developed an alternative technique based on exploratory analysis. The values of CMF were estimated for various roadway types in Florida using observational BA studies (with EB and CG approach) and cross-sectional method. In this study, the data on roadway treatments (single and combined) were obtained from previous studies (Park et al. 2014; Park & Abdel-Aty 2015). The types of treatment included, widening shoulder width, installing of shoulder rumble strips, and a combination of both treatments. The results of the comparison of the combined techniques have identified the third technique (systematic reduction on the second treatment) as the best combined technique. Gross and Hamidi (2011) used the result from two earlier studies by Hanley et al. (2000) and Pitale et al. (2009) to examine the techniques that were used to estimate combined CMFs. The study used two individual treatments (widening shoulders and installing shoulder rumble strips) to achieve the objective. The results showed that the combined CMFs that were estimated using the technique adopted by the HSM and the technique introduced by the State of Alabama were close to actual CMFs.

In summary, the conclusion from the previous studies shows that the values of CMF are likely to vary according to study area even for the same treatment type. Thus, combining the values of CMF obtained from different study areas and comparing the results with actual values of CMF for multiple treatments do not clearly identify the best technique of combining multiple treatments.

Table 2.3 Summary of existing techniques for combining individual CMFs

Number	**Techniques**	**Description**
1	$\mathrm{CMF}_{\mathrm{combined},i} = \mathrm{CMF}_{i1} \times \mathrm{CMF}_{i2} \times \ldots \times \mathrm{CMF}_{ij} \times \ldots \times \mathrm{CMF}_{in}$ $\mathrm{CMF}_{\mathrm{combined},i}$: combined CMF at the i^{th} site. CMF_{in}: CMF associated with treatment *j (j* = 1, 2, …, *n)* at i^{th} site.	Proposed by USA's HSM (AASHTO 2010) and is assume independence of treatments.
2	$\mathrm{CMF}_{\mathrm{combined},i} = 1 - [\frac{2}{3}(1 - (\mathrm{CMF}_{i1} \times \mathrm{CMF}_{i2} \times \ldots \times \mathrm{CMF}_{ij} \times \ldots \times \mathrm{CMF}_{in}))]$ $\mathrm{CMF}_{\mathrm{combined},i}$: combined CMF at the i^{th} site. CMF_{in}: CMF associated with treatment *j (j* = 1, 2, …, *n)* at i^{th} site.	Proposed by Turner (2011) and is based on multiply weighted factor.
3	$\mathrm{CMF}_{\mathrm{combined},i} = \mathrm{CMF}_{i1} - \frac{1-\mathrm{CMF}_{i2}}{2} - \cdots - \frac{1-\mathrm{CMF}_{ij}}{j} - \cdots - \frac{1-\mathrm{CMF}_{in}}{n}$ $\mathrm{CMF}_{\mathrm{combined},i}$: combined CMF at the i^{th} site. CMF_{in}: CMF associated with treatment *j (j* = 1, 2, …, *n)* at i^{th} site.	Proposed by US state of Alabama (NCHRP 2008) and is assume safety impacts of second treatment is systematically reduced.
4	Only the lowest value of CMF is applied (i.e., the most effective safety treatment).	Apply only the most effective CMF.
5	$\mathrm{CMF}_{\mathrm{combined}} = \frac{\sum_{r=1}^{n} CMF_{unbiased,r}/S_r^2}{\sum_{r=1}^{n} 1/S_r^2}$ $S = \sqrt{\frac{1}{\sum_{m=1}^{n} 1/S_m^2}}$ $\mathrm{CMF}_{\mathrm{combined}}$: combined unbiased CMF value. $\mathrm{CMF}_{\mathrm{unbiased}}$: unbiased CMF value from study *r*. n: number of CMF to be combined. S: standard error for the combined CMF.	Proposed by Bahar (2010) and is based on Meta-analysis (weighted average of multiple CMF values).

2.7 Traffic Simulation

Traffic simulation models are the most useful tools to evaluate possible traffic operations under different conditions. There are significant numbers of traffic simulation software packages available for different purposes. According to Tian et al. (2002); Trueblood and Dale (2003); Choa et al. (2004); FDOT (2014); Mahmud et al.

(2016); Xiang et al. (2016) traffic simulation programs such as VISSIM, CORSIM, Synchro/SimTraffic, SIDRA, Highway Capacity Software (HCS) 2000, LOSPLAN, and PARAMICS were the most common software packages for simulating traffic systems.

Tian et al. (2002) studied the variations in the outputs (performance measures) among three traffic simulation programs: VISSIM, SimiTraffic, and CORSIM. The study found that SimiTraffic produced the highest variation in both capacity and delay, whereas CORSIM produced the lowest variations. The highest variations usually arise when traffic demand reaches the capacity condition. However, the variations in the performance measures can be reduced by either conducting more simulation runs or using a longer simulation period. The authors also noted that multiple simulation runs are required to obtain an accurate estimation of the real-world conditions. Barrios et al. (2001) compared a number of traffic simulation programs: VISSIM, PARAMICS, SimTraffic, and CORSIM, based on their graphical presentation capabilities. The study revealed that VISSIM was favoured over others due to its three-dimensional capabilities. Similarly, Choa et al. (2004) investigated the ability of CORSIM, VISSIM, and PARAMICS to simulate a freeway interchange. The authors concluded that VISSIM and PARAMICS reflect real-world conditions more accurately. In addition, both VISSIM and PARAMICS have more input parameters which require more set-up time compared to CORSIM.

In a study by Xiang et al. (2016), the effect of the installation of median U-turn intersection as alternative treatment to reduce traffic conflicts and congestions at intersection areas was investigated. The VISSIM simulation package was employed to model and evaluate the operational features of the direct-left-turn and median U-turn intersections. In this study, data from six intersections in China were used to calibrate the model. Three performance measures including, number of stops, capacity, and delay were evaluated and compared under different scenarios (i.e., direct-left-turn and median U-turn) for the same intersections. The authors found that the operations at intersection areas were significantly improved by introducing the median U-turn rather than direct-left-turn. The VISSIM package was also employed by Trueblood and Dale (2003), to analyse traffic operation at roundabouts. The study concluded that using VISSIM to simulate roundabouts can provide a reasonable estimation of how an improved roundabout may operate. This is due to the excellent graphical capabilities of this simulation package and its ability to model roundabouts using many different scenarios.

Mandavilli et al. (2008) utilized the SIDRA simulation package to investigate the environmental effect of modern roundabouts in minimizing vehicular emissions. In this study, six non-signalised intersections with different traffic volumes were converted to modern roundabouts. Four performance measures were selected to investigate the environmental effects of the roundabouts including, CO, CO_2, HC, and NO_x emissions. The study showed that the roundabouts performed better than non-signalised intersections. In other work, Sisiopiku and Oh (2001) compared the performance of roundabouts with 4-legged intersections under signal control, yield control, and stop control with different traffic volumes using the SIDRA simulation package. In terms of capacity and delay, roundabouts showed a better performance over other intersection types with two-lane approaches and high traffic volume.

Heng and Perugu (2009) employed simulation models to identify prospective alternative routes at congestion areas in Ohio. Three routes were evaluated in the study area to identify the best alternative route. In that study, the VISSIM simulation package was used to simulate the existing conditions of the road network. While HCS 2000 and Synchro simulation packages were used to evaluate the performance at intersections based on the level of service, queue length, capacity, and delay time.

In general, different simulation packages use different input parameters and have different degree of accuracy and complexity. A brief summary of the most popular simulation packages for traffic evaluation is provided in Table 2.4. As a result of this summary, the VISSIM software package is demonstrated to provide a high degree of accuracy and has the ability to analyse all road facility types; thus, it can be accepted as suitable for the proposed research. Although there are some limitations associated with VISSIM software such as (i) required in-depth knowledge of the program and its features due to its complexity; (ii) any minor inconsistence between the simulated and real conditions can produce major error in the outputs, therefore, the network and traffic coding process should be created with care; and (iii) high cost of software.

Table 2.4 Summary of the main simulation software packages

Software	**Developed by**	**Main Performance Measures**	**Facility**	**Degree Of Accuracy and Complexity**
VISSIM	Planung Transport Verkehr (PTV), a German company	LOS [1], density, speed, travel time , and queue length	Intersections, roundabout, and roadway segments	High
Synchro/ SimTraffic	Trafficware, a United States company	LOS, density, speed, travel time , and queue length, V/C [2]	Intersections and roundabouts	Moderate to high
SIDRA	Australian Road Research Board, Australia	V/C, LOS , and delay	Intersections and roundabouts	Moderate
CORSIM	Federal Highway Administration (FHWA), United States	LOS, density, speed, travel time , and queue length	Intersections and roadway segments	High
LOSPLAN	Florida Department of Transportation (FDOT), United States	LOS	Roadway segments	Low to moderate
HCS	Microcomputers in Transportation (McTrans), United States	LOS, travel time, density, speed, V/C	Intersections and roundabouts	Moderate
PARAMICS	Quadstone Limited, a British company	LOS, speed, queue length	Intersections, roundabout, and roadway segments	Moderate

[1] Level of service (LOS) is a qualitative measure used to relate the quality of traffic operation

[2] Volume Capacity Ratio (V/C) is a measure that reflects the quality of travel of a facility

2.8 Summary

The chapter provides a comprehensive review of the available international literature of crash prediction models (CPMs) and their applications in safety estimation. The main purposes of the literature review were to understand the existing situation of the research area, to recognize the outstanding issues to be solved, and to refine the objectives and create the research framework for the current research. Through a review of the literature, the main findings are summarized below.

Different CPMs have been developed to investigate the impacts of various geometric and traffic variables on crash frequencies. However, the statistical techniques such as Poisson and Negative Binomial (NB) regression models have been widely used as suitable techniques for developing road crash models. This is due to the ability of these techniques to analyse data while preventing the possibility of having a negative integer crash value during the analysis period. Moreover, these techniques can adequately deal with the random, discrete, and typically sporadic events, which are all characteristics of road crashes. At the same time, the selection of explanatory variables in most of the reviewed models has shown that the variables were included in the CPMs without an appropriate variable selection procedure (e.g., Pearson correlation matrix). This means that the selection of the variables is done on a subjective basis (i.e., based on the availability of data) which might lead to biased results. So, the use of a variable selection procedure is useful to minimize such bias and to avoid misleading results.

Various approaches to identify the black spot locations have been developed. The integration of expected crash frequency into the method of analysis has been highlighted by researchers for precise investigations. The Empirical Bayes (EB) approach can provide an expected crash frequency for a specific location by calculating the weighted combination of the recorded and predicted crash frequencies. In addition, the EB approach has been introduced by researchers as a means of solving the RTM problem. However, this approach identifies high crash locations (black spots) based on their Potential for Safety Improvement (PSI), calculated as the difference between predicted and expected crashes at the location.

Crash Modification Factors (CMFs) or Crash Reduction Factors (CRFs) can provide a simple and quick arithmetic method for estimating crash reductions after particular treatments. Observational Before-After (BA) studies and the cross-sectional method are the two existing methods for estimating safety effectiveness and calculating the CMFs of specific roadway treatments. Several studies have estimated CMFs using observational BA studies that account for the RTM bias. Five approaches can be employed to implement observational BA studies and these are: (1) Comparison Group (CG) approach, (2) Yoked Comparison (YC) approach, (3) Naïve (simple) approach, (4) Empirical Bayes (EB) approach, and (5) Full Bayes (FB) approach. However, practical limitations associated with these methods such as countermeasures or treatment implementation dates should be known to determine the before and after evaluation periods, sufficient years have to pass after treatments are implemented, and it is difficult to distinguish safety effects when more than one treatment has been implemented at a specific site. As a result, the cross-sectional method has been widely used in recent years to estimate CMFs. In this method, the CMF value is estimated for a specific site based on its characteristics before implementation of the treatment by using the coefficients of the prediction models. According to previous studies, the results from the cross-sectional method seem to be consistent with the observational BA study results.

Several studies concluded that CMF values are likely to vary according to the study area, even for the same treatment type. Thus, combining the values of CMF obtained from different study areas and comparing the results with actual values of CMF for multiple treatments do not precisely identify the safety effect of combining multiple treatments. Many researchers have pointed out that very few studies have been carried in order to estimate CMFs for the combined effect of several safety treatments, especially within the same study area. Moreover, Gross and Hamidi (2011) and Park and Abdel-Aty (2017) stated that the Highway Safety Manual (HSM) part D and other related studies (e.g. CMF Clearinghouse) provide basic directive on the CMFs application and limited directive on the application of combined CMFs.

Most previous studies estimate CMF as a single value by ignoring the variation of CMF values among different sites characteristics. In most cases, it is not realistic to assume a uniform safety impact for all treated sites with different characteristics (Gross et al. 2010; Sacchi et al. 2014). Recently, a few studies estimated CMF values through developing a CMFunctions to overcome this limitation. A CMFunction allows the value of CMF to change based on site characteristics.

In the previous studies, the focus was only on developing CMFs and applying these factors to identify the appropriate treatments on the basis of the crash reduction percent achieved. To date, and to the best of my knowledge, there is no study has incorporated traffic simulation models with CMFs to evaluate the effect of the proposed safety treatments on both traffic operation and crash reduction achieved. Moreover, very few studies have employed cost evaluation to identify the expected cost savings after applying each type of treatment proposed.

Chapter 3
Data Collection and Methodology

3.1 Introduction

As outlined in Chapter 1, the overarching objective of this study is to determine crash modification factors (CMFs) for single and combined road treatments on intersections, roundabouts, and roadway segments. The initial phase of the research was to collect traffic data, geometric characteristics, and crash data for the selected sites. The data collection stage is very important as good data helps to ensure more efficient and reliable results at the analysis stage. In general, this study focused on the data required for estimating CMFs using the cross-sectional method. Data collection and the preparation process for analysis stage are discussed in section 3.2. The methodology adopted in this study to analyse the prepared data is discussed in section 3.3.

The flow chart for the research methodology to fulfil the objectives of the study initially stated in the introductory chapter, Chapter 1, is illustrated in Figure 3.1. The flow chart covers four main stages. The first stage (Stage 1 in Figure 3.1) reviews existing models to define the most appropriate method of analysis (see Chapter 2). In the second stage (Stage 2), the study area was selected and data collected and prepared for each road type. In the third stage (Stage 3), the crash models were developed and validated for each road type to identify black spot locations. In the last stage (Stage 4), the crash modification functions (CMFunctions) were estimated using the prediction models. The appropriate treatments were identified based on crash reduction, impact on traffic operation, and an economic appraisal of treatments. A full description of these stages is discussed in the following sections.

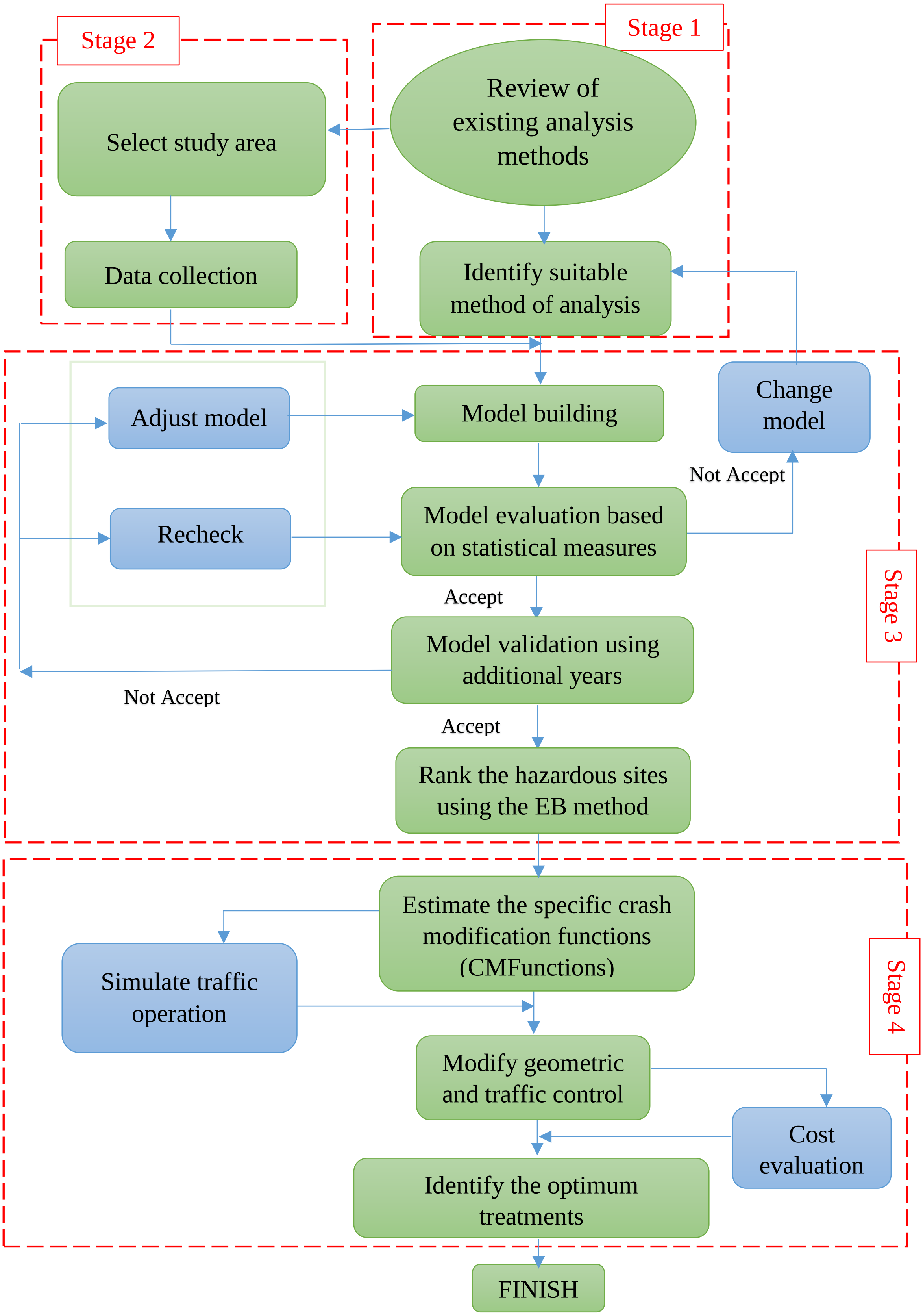

Figure 3.1 Chart flow for the research methodology

3.2 Data Collection Process

3.2.1 Identifying the Study Area

Road travel in Australia plays a dominant role where, because of demographics, approximately 90 % of passenger travel occurs by road (ABS 2012). Since the start of record-keeping in 1925, there have been more than 187,000 deaths on the roads in Australia (DIRD 2016). According to the Australian Bureau of Statistics, road crashes were the tenth leading cause of death in Australia in 2002 and road crashes contributed 22 % of deaths caused by ‘external causes’ (i.e. crashes, poisonings and violence). Over the same time period, three to four people have died and about 93 people have been seriously injured every day due to road crashes in Australia (BITRE 2015). As mentioned previously, the case study is Toowoomba City, which has an area of about 117 square kilometres and is located 130 kilometres west of Brisbane, the capital of Queensland as shown in Figure 3.2. Toowoomba is Australia’s second largest inland city with a population estimate for 2015 of 163,232, a growth of about 1.3% on a population estimate in 2014 (ABS 2015). Queensland Government statistics have revealed that per head of population, road crashes cause more deaths in Toowoomba (one death per 11,000 people) than in Brisbane, Ipswich, Logan, and the Gold Coast.

Figure 3.2 Toowoomba city location for Queensland State

3.2.2 Data Collection and Preparation

The road network in the study area was divided into intersections, roundabouts, and roadway segments. Three types of data were collected and used in the analysis; road crashes, geometric characteristics, and traffic data. Road crash data for the road network in Toowoomba city was provided by the Department of Transportation and Main Roads (DTMR), Queensland in Excel spreadsheet format. Crash data consisted of information about the crash: day, time, location, severity level, traffic control type, and speed limit. Property damage only crashes that occurred after 31 December 2010 was not recorded by DTMR and was not available. Fatal crashes were the lowest recorded crash type in the study area during the time frame used in this research. Approximately 2% of the road crashes are fatal crashes. Due to the low frequency of fatal crashes, the study has been adapted to include severe crashes (i.e., fatal plus serious injury) to accomplish the analysis stage.

The HSM (AASHTO 2010) recommends that using a study period of three to five years would be sufficient, as a period shorter than three years is more likely to have high variance due to the randomness of road crashes. In contrast, a study period of longer than five years is more likely to have bias due to physical changes in road features. In this study, data for the period from 2008-2015 was used for intersections analysis, which was divided into six years of data (2008-2013) for model prediction and two years of data (2014-2015) for model validation. Data for the period from 2010-2015 was used for roundabouts and roadway segments, which was divided into three years of data (2010-2012) for model prediction and three years of data (2013-2014) for model validation. The difference in the study period was because the number of road crashes at both roundabouts and road segments was lower compared to the number of road crashes at intersections. Thus, the number of road crashes was predicted for three years instead of one year as for in intersections.

Road geometric data was collected from site visits, historical design records, and Google Earth Pro. In addition, traffic volume data for the road networks was obtained from Toowoomba Regional Council (TRC) and DTMR, Queensland. The data were obtained in Annual Average Daily Traffic (AADT) format.

3.2.3 Site Selection

As stated by Corben and Wai (1990), the use of either high or low crash frequency locations for the data collection process could lead to concerns about the sample being biased towards high or low crash frequency approaches. Therefore, a random selection approach was adopted to minimise bias. The sites were identified based on the geographic location, to represent the Northern, Southern, Eastern and Western regions in Toowoomba. According to HSM (AASHTO 2010), the minimum sample size required for each facility type is 30 to 50 sites. Thus, a sample of 106 intersections, resulting in 1,108 severe crashes was included and considered suitable for use. The dataset included 62 signalised intersections with 813 crashes and 44 un-signalised intersections with 295 crashes. For roundabouts, a sample of 49 roundabouts, resulting in 126 severe crashes was used.

A roadway segment was defined for the study as a homogeneous segment with respect to road geometry, traffic control, and traffic volume and this resulted in varying lengths for the roadway segments. The presence of a main intersection, or change in the road characteristics, resulted in the start of a new roadway segment. Based on this definition, a sample of 89 roadway segments were considered, with a total length of 44.7 km. The total number of fatal and injury crashes in the sample segments was 315 crashes during the study period (2010-1015). It should be noted that in order to determine if there were any significant changes to the geometric design for the selected sites over all the study period (2008-2015), a visual inspection was undertaken by comparing 2008 imagery with 2015 imagery using Google Earth Pro.

3.3 Methodology

3.3.1 Model Development

This section describes the statistical models considered for modelling road crashes in the study area. A Pearson correlation matrix for all candidate independent variables was developed to examine a strong correlations between variables as discussed in a later sub-section. In addition, several performance measures were used to evaluate the goodness-of-fit (GOF) of the models and to validate the models over additional years.

3.3.1.1 Crash Prediction Modelling

As discussed in Chapter 2, there are many options for estimating the model parameters, such as Poisson distribution, negative binomial distribution, random effects, multiple linear regression, and multiple logistic regression models. Due to the characteristics and the nature of the crash data (discrete number, non-negative integer, and randomly distributed in nature), the techniques considered best to analysis data are stochastic regression models such as Poisson and negative binomial techniques (Abdel-Aty & Radwan 2000; Chin & Quddus 2003; Cafiso et al. 2010; Lord & Mannering 2010; Ackaah & Salifu 2011; El-Basyouny & Sayed 2013; Gargoum & El-Basyouny 2016; Elvik et al. 2017; Farag & Hashim 2017; Moghaddam et al. 2017). The Generalised Linear Model (GLM), which is the Poisson and negative binomial (NB) with a log-link function, was adopted for this study.

Poisson regression model

Poisson regression is a distribution that predicts the probability of a certain number of rare events occurring during a given time period (Caliendo et al. 2007). This model assumes that the mean and variance are equal or approximately equal. To analyse the road crashes at the i^{th} site (e.g., intersection, roundabout, or roadway segment), let Y_i represent the crashes number occurring on i^{th} site during a certain period and y_i represent observed number of crashes at the i^{th} site during the same time period where, y_i = 0, 1, 2, ... and i = 1, 2, 3,... . If it is assumed that, the crash numbers follow a Poisson distribution (i.e. mean equal variance) with variance μ_i, the probability of a number of crashes y_i occurring at a given time period can be expressed as follows:

$$P_{(Y_i=y_i)} = \frac{\mu_i^{y_i} \ exp^{(-\mu_i)}}{y_i!} \tag{3.1}$$

Negative binomial regression model

When the mean and the variance of the model data are not equal, the Poisson distribution becomes unsuitable for analysing the data. This problem can be resolved by the use of negative binominal (NB) regression instead of Poisson regression. The NB regression describes the occurrence of random and rare events. This model can be used in the case of means smaller than the variance ($\mu + \mu^2 /k$). Generally, the NB model uses the following distribution form shown below.

$$P(Y_i = y_i) = \frac{\Gamma\,(y_i + k^{-1})}{\Gamma\,(k^{-1}) y_i!} \ \left(\frac{1}{1+k\,\mu_i}\right)^{k^{-1}} \left(\frac{k\,\mu_i}{1+k\,\mu_i}\right)^{y_i} \tag{3.2}$$

Where, k is the dispersion parameter and Γ is the gamma function.

The general form of the prediction model by using Poisson or NB regression is as follows:

$$N_{pre.i} = e^{\beta_0 + \sum_{j=1}^{n} \beta_j X_{ij}} \tag{3.3}$$

Where, $N_{pre.i}$ is the predicted crashes number per time period (T) at i^{th} site; β_0, and β_j are model parameters; X_{ij} is explanatory variable j at i^{th} site. In this study, based on the HSM and related studies, the expression in Equation 3.3 above has been rewritten as follows:

For intersection and roundabout models;

$$N_{pre.i} = Q_{major,i}^{\alpha 1} \cdot Q_{minor,i}^{\alpha 2} \cdot e^{\beta_0 + \sum_{j=1}^{n} \beta_j X_{ij}} \tag{3.4}$$

For roadway segment models;

$$N_{pre.i} = SL_i^{\alpha 1} \cdot Q_i^{\alpha 2} \cdot e^{\beta_0 + \sum_{j=1}^{n} \beta_j X_{ij}} \tag{3.5}$$

Where, $Q_{major,i}$ and $Q_{minor,i}$ are the AADT on major and minor approach at i^{th} site, respectively; Q_i is the AADT on roadway segment at i^{th} site; SL_i is the length of roadway segment at i^{th} site; X_{ij} is the explanatory variable j at i^{th} site; and α_1, α_2, β_0, and β_j are the model parameters. Equations 3.4 and 3.5 were obtained by using natural logarithm for the variables AADT and length of roadway segment, to reflect the nonlinear relationship between these variables and crash frequency (Wong et al. 2007; Abdel-Aty & Haleem 2011; Park et al. 2014). IBM SPSS statistics version 23 (IBM Corp 2015) was the software utilized to estimate the model parameters.

3.3.1.2 Correlation Matrix

In this section, the Pearson correlation matrix for all candidate independent variables was developed using the IBM SPSS (IBM Corp 2015) statistics. A Pearson correlation matrix was used to measure the strength of linear dependence between the individual independent variables. The value of the Pearson correlation coefficient is usually between +1 and - 1. A zero value refers to no correlation between the two given variables and 1.00 value refers to a strong correlation or relationship between the two given variables. A positive value indicates a direct relationship between the variables and a negative value indicates a reverse relationship between the variables. The purpose of this matrix was to investigate whether some independent variables were strongly correlated. A strong correlation between independent variables in regressions could strongly affect the other coefficients in the same prediction model (Abdel-Aty & Radwan 2000; Washington et al. 2010; Turner et al. 2012). The inference is that adding more than one independent variable does not add to the quality of the model and having two in the same model may render the model non-significant. The strength of the relationship is classified by Navidi (2008) as presented in Table 3.1. In this study, the correlation value (Pearson correlation) between independent variables in prediction models was accepted between -0.49 and +0.49 at moderate strength.

Table 3.1 Classification of Correlation Strength

Strength of Relationship	**Value of Correlation**
Non or Very weak	0.0 to ± 0.09
Weak	± 0.1 to ± 0.29
Moderate	± 0.3 to ± 0.49
Strong	± 0.5 to ± 1.00

3.3.1.3 Measuring Goodness-of-Fit

Various performance measures were used to test the model assumption and to verify the goodness-of-fit (GOF) of different models, including the deviance, the Pearson chi-square (x^2), Akaike's information criterion (AIC), Bayesian information criterion (BIC), residuals plot, and cumulative residual (CURE) plot.

a) Deviance and Pearson chi-square (x^2) were adopted to verify if the dataset followed a NB distribution or Poisson distribution. Generally, if the value of the deviance divided by the degree of freedom (*df*) and the value of the Pearson Chi-square (x^2) divided by the degree of freedom (*df*) is between 0.8 and 1.2, this indicates that the model assumption (i.e., NB distribution or Poisson distribution) is appropriate to fit the data (Bauer & Harwood 2000; Ackaah & Salifu 2011; Abdul Manan et al. 2013). Both deviance and Pearson chi-square (x^2) are calculated as follows (Pearson 1934):

$$Deviance = 2\sum_{i=1}^{n}(y_i\, log\frac{y_i}{\acute{y}_i} - y_i + \acute{y}_i) \quad (3.6)$$

$$X^2 = \sum_{i=1}^{n}\frac{(y_i - \acute{y}_i)^2}{\acute{y}_i} \quad (3.7)$$

Where, $\acute{y}_i$ is the predicted crash number at i^{th} site; and y_i is the observed crash number at i^{th} site.

b) Akaike's information criterion (AIC) test was used to measure the GOF of each model, relative to each of the other models. In other worlds, this test can be used to identify the best fitting model from several candidates. The AIC test was defined by Akaike (1974) as shown below.

$$AIC = -2\log L + 2P \tag{3.8}$$

Where, log L is the maximum log-likelihood of the Model; P is the number of independent variables in the model excluding the constant.

c) Bayesian Information Criterion (BIC) test was used to measure the GOF of each model, relative to each of the other models. The BIC is similar to AIC test, but takes into account the sample size. BIC test was defined by Schwarz (1978) as shown below.

$$BIC = -2\log L + \ln(n)\,S \tag{3.9}$$

Where, n is the number of data points (sample size) and S is the number of independent variables in the model including the constant. In general, the smaller the AIC and BIC values, the more preferred the model would be (Cafiso et al. 2010; Abdul Manan et al. 2013; Young & Park 2013).

d) Residuals plot method is a graphical measure used to compare different models (Washington et al. 2005; Haleem et al. 2010; Wang et al. 2013). Using this performance measure, the residual values (defined as the difference between the observed and predicted crash number at each site) were plotted against the natural logarithm of AADT variable as one of the main common independent variables used in the analysis. The indication that the model fits the data well is when the residual values fluctuate around the zero value, and the residual values are not widely spread.

e) The cumulative residual (CURE) plot was proposed by Hauer and Bamfo (1997) to evaluate how well the developed model fits the data. The CUREs (defined as the sum of the differences between the observed and predicted values) are plotted in increasing order for an independent variable, usually plotted against AADT. In CURE plot, the closer the curve randomly fluctuates around the horizontal axis (zero-residual line) and lies between the two standard deviation curves (+2σ and -2σ), the better the developed model fits the data. The CURE curve above zero line indicates that a model under-estimates the crash count, whereas, CURE curve below zero line indicates that a model over-estimates the crash count. Also, large vertical drifts upward or downward in the curve represent large residual values.

3.3.1.4 Model Validation

The validation of the crash prediction models (CPMs) against sequential additional years of crash data for the study area were used to evaluate the models' ability to predict crash numbers. Generally, researchers (Washington et al. 2005; Bissonette & Cramer 2008; Washington et al. 2010; Mehta & Lou 2013; Young & Park 2013) have recommended using multiple measures to examine a particular model's validity because no single test has a 100% reliable answer. For this study, four measures were applied for validating CPMs, which are the mean squared prediction error (MSPE), the mean square error (MSE), the mean absolute deviation (MAD), and the Freeman Tukey R-squared (R^2_{FT}) measure. These measures were used to validate the developed models based on the observed number of crashes in the validation dataset (i.e., using additional years) and predicted number of crashes.

a) Mean squared prediction error (MSPE) mesure is used to determine the variance of the difference between observed crashes and predicted crashes results. In addition, it is typically employed to evaluate error associated with a validation dataset. The MSPE value is calculated as follows (Washington et al. 2005):

$$MSPE = \frac{1}{n}\sum_{i=1}^{n}(\acute{y}_i - y_i)^2 \quad (3.10)$$

Where:

$\acute{y}_i$ –is the predicted crashes number at i^{th} site;
y_i –is the observed crashes number at i^{th} site; and
n –is the sample size of database.

b) Mean square error (MSE) measure is typically used to evaluate error associated with an estimation dataset. Ideally, MSPE and MSE results can be used to reveal whether the models are over-fitted (MSPE>MSE) or under-fitted (MSPE<MSE) (Bissonette & Cramer 2008). The MSE value is calculated as follows (Washington et al. 2005):

$$MSE = \frac{1}{n-\mathrm{p}}\sum_{i=1}^{n}(\acute{y}_i - y_i)^2 \quad (3.11)$$

Where, p is the number of model parameters.

c) Mean absolute deviation (MAD) value provides a measure of the average magnitude of the prediction variability using both estimation and validation dataset. The MSE value is calculated as follows (Washington et al. 2005):

$$MAD = \frac{1}{n}\sum_{i=1}^{n}|\acute{y}_i - y_i| \quad (3.12)$$

In general, a smaller value (closer to zero) of MSPE, MAD, or MSE refers to a lower prediction error.

d) Freeman-Tukey R-Squared coefficient (R^2_{FT}) value also provides a measure of the average magnitude of the prediction variability. Larger R^2_{FT} value refers to a better fit. The R^2_{FT} value is calculated as follows (Freeman & Tukey 1950; Hamidi et al. 2010):

$$R^2_{FT} = \frac{\sum_{i=1}^{n}(f_i - f')^2 - \sum_{i=1}^{n}\hat{e}_i^2}{\sum_{i=1}^{n}(f_i - f')^2} \quad (3.13)$$

$$f_i = \sqrt{y} + \sqrt{y_i + 1} \quad (3.14)$$

$$\hat{e}_i = f_i - \sqrt{4 \times \acute{y}_i + 1} \quad (3.15)$$

Where:

f_i – is the Freeman-Tukey transform of y_i (is the variance stabilising transformation of variable y_i with mean $\acute{y}_i$);

f' – is the sample mean of f_i; and

$\hat{e}_i$ – is the Freeman-Tukey deviate at i^{th} site (is estimated by corresponding residual).

3.3.2 Identifying High Crash Locations

As mentioned earlier, CPMs are the only part of the total safety evaluation process for this study. The Empirical Bayes (EB) adjustment method was employed in this study to increase the accuracy of safety estimation by accounting for the regression to the mean (RTM) bias usually associated with the road crash data. RTM is the tendency of crash data to regress back to the mean (Tegge et al. 2010). The EB method has been introduced by researchers as a means to solve the RTM problem. The expected crash frequency and weighting adjustment factor for each site in the study area were calculated using the EB adjustment method. The general function for this method is defined as follows (AASHTO 2010; Srinivasan & Carter 2011):

$$N_{exp.i} = \omega_i \times N_{pre.i} + (1 - \omega_i) \times N_{obs.i} \quad (3.16)$$

For intersections and roundabouts, ω_i value can be calculated as follows:

$$\omega_i = \frac{1}{1+K\times\sum_{t=1}^{T} N_{pre.i}} \tag{3.17}$$

For roadway segments, ω_i value can also be calculated as follows:

$$\omega_i = \frac{1}{1+\frac{K}{SL_i}\times\sum_{t=1}^{T} N_{pre.i}} \tag{3.18}$$

Where:

$N_{exp.i}$ –is the expected crash frequency at i^{th} site;

ω_i –is the weighting adjustment to model prediction at i^{th} site;

$N_{pre.i}$ –is the predicted crash frequency in a period time T at i^{th} site (Equations 3.3-3.5);

$N_{obs.i}$ –is the observed crash frequency at i^{th} site;

K –is the over dispersion parameter of a prediction model; and

SL_i –is the length (km) of roadway segment.

The research identified high crash locations (black spots) based on their potential for safety improvement (PSI), calculated as the difference between predicted and expected crashes at a particular site as shown in Figure 3.3. The PSI values were calculated for all sites to identify and rank sites in the study area. Ideally, a positive value of PSI shows that the potential for safety improvements exists.

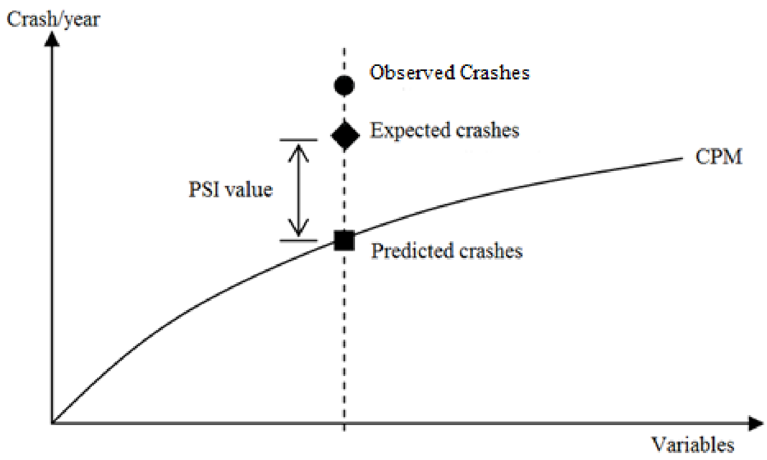

Figure 3.3 PSI computation using EB adjustment method

3.3.3 Crash Modification Factors

3.3.3.1 Crash Modification Function

Crash modification factor (CMF) is a value representing the change in road safety after modifying the geometric design or operation of the facility. As a part of the cross-sectional method, crash modification function (CMFunction) has been employed to estimate safety effectiveness and measure the CMFs of specific roadway treatments. This method was applied based on the parameter of the explanatory variable associated with the proposed treatment type. The value of CMF was estimated for a particular treatment type as follows (Lord & Bonneson 2007; Abdel-Aty et al. 2014):

$$CMFi = \frac{predicted\ average\ crash\ frequency\ with\ treatment}{predicted\ average\ crash\ frequency\ without\ treatment} \tag{3.19}$$

The expression in Equation (3.19) can also be written as shown in Equation (3.20).

$$CMF_i = e^{\beta_i \times [X_i - X_{ib}]} \tag{3.20}$$

Where, X_i is the observed value for the variable i; X_{ib} is the base condition for the variable i; and β_j is the model parameters for the variable i. When the value of CMF equals 1.0 there is no effect on safety. A CMF above 1.0 indicates that treatment results in a higher number of crashes. In contrast, CMF below 1.0 indicates that treatment results in a lower number of crashes. The standard error (Std. Er) of the CMF for each treatment type was also calculated as follows (Bahar 2010):

$$Std.Er_i = \frac{(e^{\beta_i\ [X_i - X_{ib}] + Std.Er_{\beta i}} - e^{\beta_i\ [X_i - X_{ib}] - Std.Er_{\beta i}})}{2} \tag{3.21}$$

Where, *Std. Er*$_i$ is the standard error of the CMF$_i$ and *Std. Er*$_{\beta i}$ is the standard error of the model parameter β_j. It should be noted that a standard error that equals 0.1 or less indicates that a CMF is more accurate (Abdel-Aty et al. 2014).

The base condition values in this study were adopted from previous studies and from the mean values of the dataset used. However, the base condition for individual sites may take different values to acomodate the site conditions, therefore, they need to be adjusted to accommodate the actual site condition. By definition, the base condition can be defined as the condition associated with CMF value 1.0.

3.3.3.2 Estimating Combined CMFs

The next stage undertaken was to analyse the CMFs for combined treatments using different techniques. The CMFs for combined treatments are estimated using the following four existing techniques: the HSM technique (technique 1); the Turner technique (technique 2); the systematic reduction of subsequent CMFs technique (technique 3); and applying only the most effective CMF technique (technique 4).

The first technique was adopted by the USA's HSM (AASHTO 2010) and this technique assumes that the road safety effect of each treatment is independent when CMFs for individual treatments are multiplied to estimate combined CMFs (Park et al. 2014; Wu & Lord 2016). For this technique, Equation (3.22) was used to estimate combined CMF at the i^{th} site.

$$\mathrm{CMF}_{\mathrm{combined},i} = \mathrm{CMF}_{i1} \times \mathrm{CMF}_{i2} \times \ldots \times \mathrm{CMF}_{ij} \times \ldots \times \mathrm{CMF}_{in} \tag{3.22}$$

Where, CMF_{in} is the crash modification factor associated with treatment j (j = 1, 2,…, n) at i^{th} site.

The second technique was proposed by Turner (2011), where a specific weighted factor of 2/3 (two-thirds) is applied to the multiplication of the CMFs for individual treatments. The combined CMF is estimated using Turner's technique as in Equation (3.23).

$$\mathrm{CMF}_{\mathrm{combined},i} = 1 - [\tfrac{2}{3}(1 - (\mathrm{CMF}_{i1} \times \mathrm{CMF}_{i2} \times \ldots \times \mathrm{CMF}_{ij} \times \ldots \times \mathrm{CMF}_{in}))] \tag{3.23}$$

The third technique was proposed by the US State of Alabama (NCHRP 2008), which assumed that the safety effects of the less effective safety treatment are systematically reduced. This means that the full effect of the most effective safety treatment among all treatments is used and had an added benefit of additional treatments (i.e., less effective treatments) as detailed in Equation (3.24).

$$\mathrm{CMF}_{\mathrm{combined},i} = \mathrm{CMF}_{i1} - \frac{1-\mathrm{CMF}_{i2}}{2} - \cdots - \frac{1-\mathrm{CMF}_{ij}}{j} - \cdots - \frac{1-\mathrm{CMF}_{in}}{n} \tag{3.24}$$

The fourth technique applies to only the most effective safety treatment, which is the lowest CMF value. However, the main disadvantage of this technique is that it may underestimate the combined effect of safety treatments if the additional safety treatments provided additional benefit (Gross & Hamidi 2011).

Finally, the average values from these four techniques (adjustment approaches) was adopted in the analysis to calculate CMFs for multiple treatments.

3.3.4 Evaluating Effectiveness by Simulation

Simulation is a powerful technique to examine the effect of changes in system parameters where the influence of such changes cannot be determined analytically. In the past, simulation models have been extensively used to generate a range of possible scenarios from which traffic operational performance can be estimated. In addition, these models help to compare the before and after scenarios. In this study, traffic simulation models were employed to simulate the traffic operations in order to determine the effect of suggested safety treatments on traffic conditions (e.g., delay, level of service, travel time, etc.). The microscopic traffic simulation software VISSIM 9.0 (PTV 2016) has been utilized in this research. Figure 3.4 shows the three main steps to evaluate traffic conditions before-after.

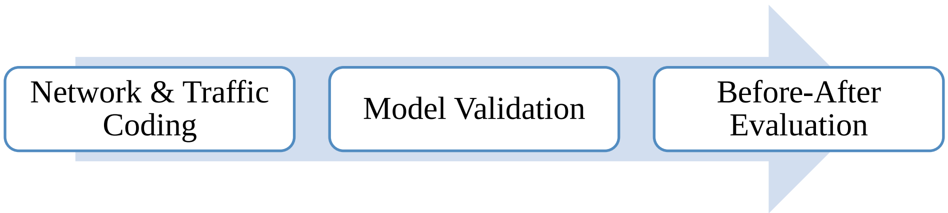

Figure 3.4 Before-after evaluation process using VISSIM

The following three steps describe in detail the evaluation of traffic operations before-after treatment implementation at the study area:

(a) In the first step, the traffic simulation models were constructed for the road network (i.e., intersections, roundabouts, and roadway segments) using the existing road conditions. In this step, three categories of data were required to generate the basic VISSIM input files including, supply, demand, and control data. The supply data included traffic and geometric characteristics of the road network, for instance, number of lanes, lane width, shoulder width, median island, and grade. This data type was obtained using Toowoomba Regional Council (TRC) data, Google Earth Pro, and site inspection. The second type of the data was demand data, which included traffic counts for road networks within the study area. The demand data was obtained from TRC and from the Department of Transport and Main Roads (DTMR), Queensland. The last type of data was control data including speed limit, traffic control type, and signal timing at intersections. The control data was obtained from the jurisdiction road authorities and site observation. Generally, VISSIM uses the notion of links and nodes to describe a road network. A link refers to a segment of road or highway between two nodes, and node usually refers to an intersection of two or more links. The road network should be laid out using the traffic and geometric characteristics (e.g., number of lanes) as well as the measured distances (e.g., width of lanes).

(b) In the second step, the models were validated to ensure that each model provided realistic simulations for existing conditions. This step was carried out before making any change in the base conditions of the road features. The validation stage included the comparison between the real and simulated values of delay time, level of service (LOS), travel time, and average speed at a particular site. For intersections and roundabouts, the delay time and LOS were used to evaluate the results, whereas, the roadway segments were evaluated using travel time and average speed. Table 3.2 shows LOS criteria for signalised and non-signalised intersections, as described in the Highway Capacity Manual (HCM2010) Volume 3. These criteria were also adopted by Austroads.

Table 3.2 LOS Criteria for intersections.

LOS	**Average Delay (sec/veh)**	
	Signalised Intersections	**Non-signalised Intersections [a]**
A	≤10	≤10
B	>10-20	>10-15
C	>20-35	>15-25
D	>35-55	>25-35
E	>55-80	>35-50
F	>80	>50

[a] Non-signalised intersection included all-way stop and roundabout control.
Source: HCM2010 (Transportation Research Board 2010).

In order to further confirm the simulation results, the average of 10 simulation runs for each site was adopted with random seed values. The simulation time for each run was a total of 3600 seconds with an interval period of 600 seconds. A relative error of 10% or less was considered to be acceptable and the following equation was used to calculate the relative error (Leng et al. 2008).

$$Relative\ error = \frac{simulation\ value - observed\ value}{observed\ value} \times 100\% \qquad (3.25)$$

(c) In the last step, the road features were modified and analysed according to the proposed treatments to evaluate the traffic operations before and after the proposed improvements.

3.3.5 Benefit Analysis

The crash reduction factors (CRFs) (i.e., $\mathrm{CRF} = 100 - \mathrm{CMF}\%$) for the proposed treatments were calculated to identify the potential crash reduction number after treatments were implemented. This step helped to distinguish between several proposed treatments to identify the best treatments for safety improvement and to study the ability to apply these treatments, considering the cost benefit. The total cost benefit of safety improvement projects can be determined by using the total costs gained from the expected number of crash reductions. BITRE (2009) estimated the average cost of road crashes based on the crash outcome in Queensland, Australia. The cost of road crashes per each fatality and injury in 2006 was found to be $2,664,622 and $266,016 (AUD), respectively. In the present study, the difference between the crash cost before and after treatments was calculated to define the cost saved based on the average cost of crashes estimated by BITRE (2009). These costs have also been adjusted to reflect the cost in 2017 instead of 2006 using an inflation rate of 2.5%. The inflation rate value was obtained from the average of Australian inflation rates between 2006 and 2017 as shown in Table 3.3. The formula that is used to estimate the crash costs in 2017, based on the crash costs in 2006 is as follows:

$$\mathrm{Cost}_{\,2017} = \mathrm{Cost}_{\,2006} \times (\,i + 1)^{n} \tag{3.26}$$

Where, *i* is the inflation rate; *n* is the difference between base year (i.e., 2006) and selected year, i.e. 2017.

In this study, the present value (PV) refers to the total discounted benefits for each site based on 10-year treatments life. Likewise, for PV calculation, the values of benefit discount rate typically range between 4.0% and 10.0%. The benefit discount rate reflects the time value of money. It is worth mentioning that the discount rate is inappropriate for evaluating human risk (Litman 2009), thus the benefit discounted rate was conservatively adopted in this study at a lower value i.e., 4.0%. The present values were calculated for each site using the following formula.

$$\text{Present value (PV)}_{benefit} = \sum_{n=1}^{N} \frac{C}{(1+r)^{n}} \tag{3.27}$$

Where, *C* is the net annual benefit; *r* is the discount rate; and *N* is the number of years of benefit (depending on the treatment life). Net annual benefit is the difference between crash costs before and after the implementation of treatments. The present value results were then used to quantify the benefit (i.e., crash cost reduction) of implementing each safety treatment at any particular site. Ideally, the present value can also be of assistance to the projects that presumably take priority.

Table 3.3 Percentages of Australia's inflation rate from 2006 to 2017

Year	**Percentages of inflation rate (2006 to 2017) [a]**				
	March	June	September	December	**Average**
2017	2.10	1.90	1.80	1.90	1.93
2016	1.30	1.00	1.30	1.50	1.28
2015	1.30	1.50	1.50	1.70	1.50
2014	2.90	3.00	2.30	1.70	2.48
2013	2.50	2.40	2.20	2.70	2.45
2012	1.60	1.20	2.00	2.20	1.75
2011	3.30	3.50	3.40	3.00	3.30
2010	2.90	3.10	2.90	2.80	2.93
2009	2.40	1.40	1.20	2.10	1.78
2008	4.30	4.40	5.00	3.70	4.35
2007	2.50	2.10	1.80	2.90	2.33
2006	2.90	4.00	4.00	3.30	3.55
Average					**2.50**

[a] Source: Australian Bureau of Statistics, Consumer Price Index

3.4 Summary

This Chapter has described the data collection process and methodology adopted. The data collection process comprised three elements: identifying the study area; data collection and preparation; and selecting the road facility. The data collected for all selected sites included road crash data, traffic volume data, traffic control data, and road geometry data. Three types of road facilities were used to perform the analysis: road intersections, roundabouts, and roadway segments.

The methodology that followed to achieve the study objectives comprised five parts: model development; identifying high crash locations; crash modification factors (single and combined); traffic simulation; and cost benefit analysis. The GLM with log-link function was proposed for crash modelling. Then, the EB adjustment method was employed for identifying high crash locations by calculating the weighted average of recorded and predicted crashes of a particular location. Thereafter, a cross-sectional method was used to estimate the CMFs as it has many advantages over other methods, such as simplicity in data collection. Finally, the proposed treatments at identified locations were evaluated using traffic simulation (VISSIM) and economic analysis.

Chapter 4
Intersection Safety Analysis

4.1 Introduction

As shown earlier in the literature review, numerous road safety studies have confirmed that intersections are among the most hazardous sites on road networks. In particular, intersections are inherently risky in cities because of their concentration per kilometre of the roadway. Intersections are recognised as a key consideration in the road network to accommodate the flow of safe traffic in all directions. Statistics indicate that 43.5% of all road crashes (fatalities and hospitalised injuries) in the state of Queensland during the period 2008-2015 occurred at intersections. In Toowoomba City, it was reported that 50.4% of all road crashes (fatalities and hospitalised injuries) took place at intersections during the same period (Queensland Government 2016).

This Chapter deals with investigating and predicting crash frequency at intersections using the Negative Binomial (NB) and/or Poisson statistical models. These models developed for local conditions were used to identify the geometric and traffic factors that would contribute to crashes at those intersections. The Empirical Bayes (EB) method was then used to identify local hazardous (black spot) intersections. These locally developed models were then used to estimate CMFs at the hazardous intersections to determine how each treatment could affect road safety. Combined CMFs for multiple treatments were also estimated using the techniques of the Highway Safety Manual (HSM), Turner, Alabama, and the most effective CMF (lower value) technique. Finally, traffic simulation models and benefit-cost analyses were employed to evaluate the expected outcomes after applying the safety improvements resulting from the research.

4.2 Data Preparation

The crash data obtained from the Department of Transport and Main Roads, Queensland included all roads and intersections in Queensland and it was necessary to separate out the intersection crashes for Toowoomba City to select sites for the study. As stated by Corben and Wai (1990), the use of either high or low crash frequency locations for the data collection process could lead to concerns about the sample being biased towards high or low crash frequency approaches. Therefore, random selection approach was employed to avoid any bias.

A sample of 106 intersections, which had resulted in 1,108 fatal and serious injury crashes, were randomly selected for the study. The dataset included 62 signalised intersections with 813 crashes and 44 un-signalised intersections with 295 crashes. The intersections were separated based on their geographic location in Toowoomba using quadrants of the city.

The study area was divided into four quadrants using James Street and Ruthven Street, which provided a uniform distribution for data selection based on the geographic location as shown in Figure 4.1. The intersections were identified using their location in the North-East (NE), North-West (NW), South East (SE) and South-West (SW) quadrants together with a reference number (e.g., NE5: James Street with Hume Street).

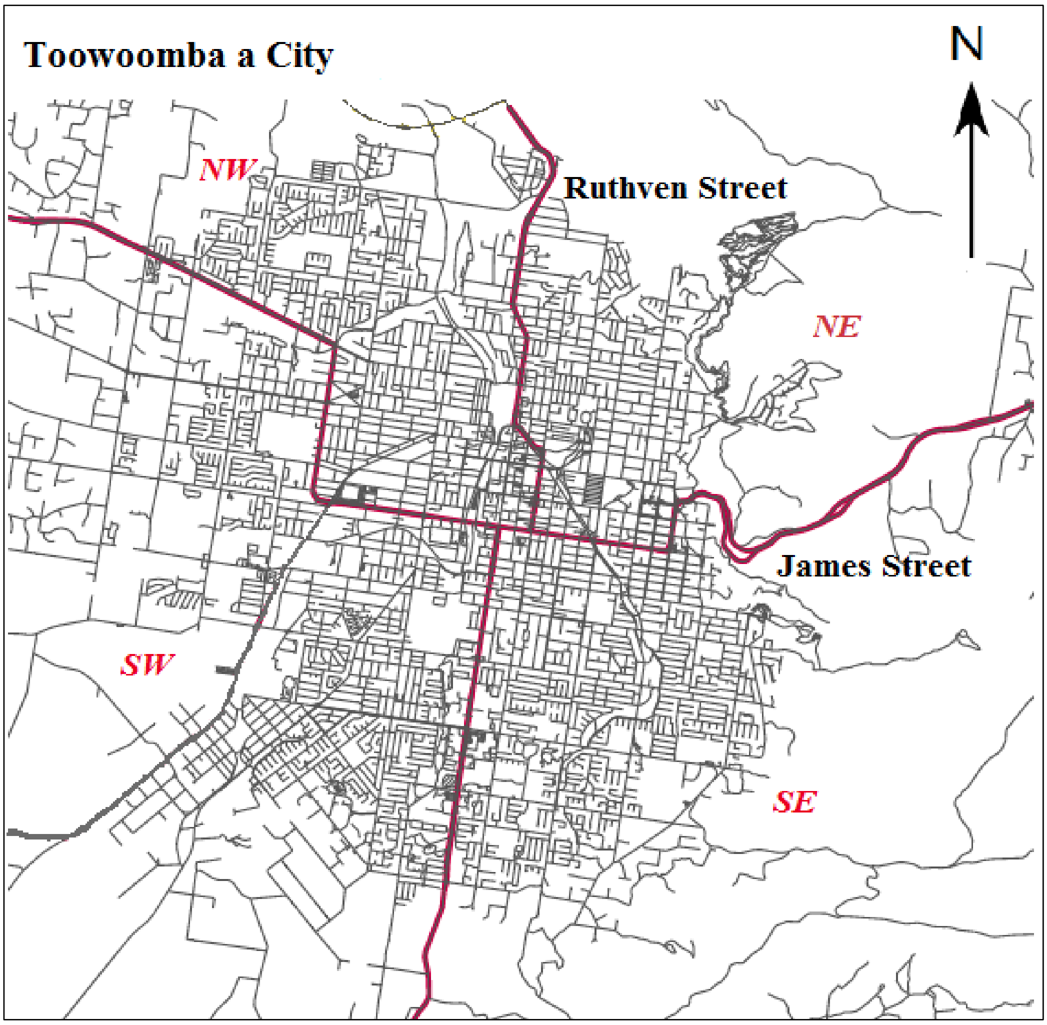

Figure 4.1 Toowoomba Road Network
Source: Toowoomba Regional Council, 2017

The intersections were identified using their location in one of the NE, NW, SE, or SW quadrants, with a number to identify the particular intersection. An example is given below, and full details of all intersections are given in Appendix A.

Intersection ID	Road Name
I_NE21	Hume Street and Chalk Street
I_NW21	Anzac Avenue and Herries Street
I_SE21	South Street and Ramsay Street
I_SW21	Drayton Road and South Street

Intersection crashes were defined as the number of crashes that occurred at the intersection area and within twenty meters measured upstream from the stop line as shown in the Figure 4.2.

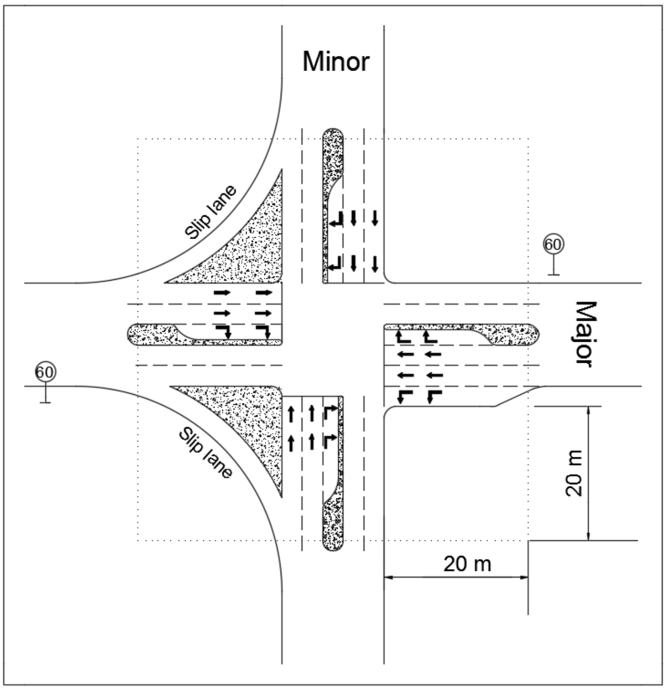

Figure 4.2 Intersection area boundary used in this study to count road crashes

For the scope of this analysis, the six years' (2008-2013) crash data were used for modelling purposes. The subsequent two years (2014-2015) crash data were then used for model validation. In order to propose effective crash reduction measures, it was important to understand the main factors that contribute to the occurrence of crashes. Seventeen variables were identified in this research as the most common factors associated with intersection road crashes and a detailed description of these variables is given below:

1. Number of Legs: This variable is the number of intersection legs, i.e. 3, 4, or 5 legs.
2. Number of through lanes entering: This variable is the total number of through lanes entering for major approaches and in the same way for minor approaches.
3. Number of through lanes exiting: This variable is the total number of through lanes exiting for major approaches and in the same way for minor approaches.
4. Number of right turn lanes: This variable is the number of exclusive right turn lanes for major approaches and in the same way for minor approaches.
5. Number of left turn lanes: This variable is the number of exclusive left turn lanes for major approaches and in the same way for minor approaches.

6. Number of Slip lanes: This variable is the number of slip lanes on the major or minor approaches used to allow the vehicles to turn without entering the intersection.
7. Control Type: This variable is the type of traffic control at the intersection, i.e. Signalized or Un-signalized intersection.
8. Traffic Volume: This variable is the Annual Average Daily Traffic (AADT) on the major approach and on the minor approach.
9. Median Island: This variable is the presence or otherwise of a raised median island at major approach and in the same way for minor approach.
10. Speed Limit: This variable is the speed limit in kilometres per hour on the major approach.

4.3 Developing Crash Prediction Models for Intersections

Using SPSS software version 23 (IBM Corp 2015), the Generalised Linear Model (GLM), i.e. Poisson and NB with log link analysis, was performed for this study as described in Chapter 3. Firstly, the NB distribution was used and tested using the value of Pearson Chi-square (x^2) divided by the degree of freedom (*df*) and the value of variance divided by the degree of freedom (*df*) (Abdul Manan et al. 2013). These values indicate whether the NB distribution assumption is acceptable or not. In the case of the assumption not being accepted, the Poisson distribution would be used.

4.3.1 Identifying Possible Models using a Correlation Matrix

To determine which explanatory variables should be considered for model development, a Pearson correlation matrix was used. Table 4.1 provides the correlation values between the 17 variables. Notation for each variable is provided in Table 4.2.

Table 4.1 Pearson Correlation Matrix for Intersections

Variable		Lg_i	LN_{i1}	LN_{i2}	LE_{i1}	LE_{i2}	TC	LT_1	LT_2	RT_1	RT_2	Q_{major}	Q_{minor}	SL_1	SL_2	MI_1	MI_2	V_i
Lg_i	**Pearson Correlation** [a]	1																
	Sig. (2-tailed)																	
LN_{i1}	**Pearson Correlation** [a]	.232	1															
	Sig. (2-tailed)	.017																
LN_{i2}	**Pearson Correlation** [a]	.816	.354	1														
	Sig. (2-tailed)	.000	.000															
LE_{i1}	**Pearson Correlation** [a]	.104	.742	.166	1													
	Sig. (2-tailed)	.291	.000	.090														
LE_{i2}	**Pearson Correlation** [a]	.719	.292	.794	.287	1												
	Sig. (2-tailed)	.000	.002	.000	.003													
TC	**Pearson Correlation** [a]	.589	.464	.682	.215	.574	1											
	Sig. (2-tailed)	.000	.000	.000	.027	.000												
LT_1	**Pearson Correlation** [a]	.163	.053	.219	.132	.254	.255	1										
	Sig. (2-tailed)	.095	.592	.024	.177	.009	.008											
LT_2	**Pearson Correlation** [a]	-.309	.158	-.413	.181	-.267	-.044	.051	1									
	Sig. (2-tailed)	.001	.105	.000	.063	.006	.658	.601										
RT_1	**Pearson Correlation** [a]	.291	.326	.368	.464	.469	.484	.292	.084	1								
	Sig. (2-tailed)	.002	.001	.000	.000	.000	.000	.002	.392									
RT_2	**Pearson Correlation** [a]	.116	.163	.100	.280	.454	.387	.265	.203	.520	1							
	Sig. (2-tailed)	.236	.095	.306	.004	.000	.000	.006	.036	.000								
Q_{major}	**Pearson Correlation** [a]	.075	.516	.238	.672	.296	.177	.197	-.039	.456	.223	1						
	Sig. (2-tailed)	.445	.000	.014	.000	.002	.069	.043	.694	.000	.022							
Q_{minor}	**Pearson Correlation**	.381	.215	.451	.233	.545	.511	.172	.014	.417	.425	.286	1					
	Sig. (2-tailed)	.000	.027	.000	.016	.000	.000	.077	.890	.000	.000	.003						
SL_1	**Pearson Correlation** [a]	.236	.256	.342	.303	.333	.339	-.144	-.036	.322	.205	.268	.430	1				
	Sig. (2-tailed)	.015	.008	.000	.002	.000	.000	.140	.714	.001	.035	.006	.000					
SL_2	**Pearson Correlation** [a]	.211	.196	.253	.241	.424	.298	-.107	-.051	.355	.398	.161	.363	.638	1			
	Sig. (2-tailed)	.030	.045	.009	.013	.000	.002	.275	.601	.000	.000	.099	.000	.000				
MI_1	**Pearson Correlation** [a]	.128	.265	.162	.481	.342	.282	.186	.234	.661	.434	.330	.267	.391	.310	1		
	Sig. (2-tailed)	.191	.006	.097	.000	.000	.003	.056	.016	.000	.000	.001	.006	.000	.001			
MI_2	**Pearson Correlation** [a]	.174	.169	.101	.308	.368	.104	-.035	.110	.270	.399	.195	.134	.315	.484	.468	1	
	Sig. (2-tailed)	.075	.082	.302	.001	.000	.288	.721	.260	.005	.000	.046	.172	.001	.000	.000		
V_i	**Pearson Correlation** [a]	.037	.302	.170	.209	.059	-.008	-.233	-.144	-.071	-.234	.168	.120	.119	.088	-.119	.046	1
	Sig. (2-tailed)	.705	.002	.082	.032	.551	.938	.016	.140	.471	.016	.085	.220	.223	.367	.223	.639	

[a] Listwise N=106.

The Pearson correlation between independent variables in prediction models is accepted when values are between -0.49 and +0.49 (moderate correlation) and the variable parameter is considered to be statistically significant at a 0.1 significance level (using 90% confidence). Based on these criteria, four road safety models were identified for use as shown in Table 4.2.

Table 4.2 Variables included in the selected intersection models

Variable	**SPSS labelling**	**Model I**	**Model II**	**Model III**	**Model IV**
Number of legs	Lg_i	✓			
Number of through lanes entering on major approaches	LN_{i1}				✓
Number of through lanes entering on minor approaches	LN_{i2}			✓	
Number of through lanes exiting on major approaches	LE_{i1}		✓	✓	
Number of through lanes exiting on minor approaches	LE_{i2}		✓		
Traffic control type	TC				✓
Number of left turn lane on major approaches	LT_1	✓	✓		✓
Number of left turn lane on minor approaches	LT_2			✓	
Number of right turn lane on major approaches	RT_1	✓			
Number of right turn lane on minor approaches	RT_2			✓	✓
AADT on major approach	Q_{major}	✓			
AADT on minor approach	Q_{minor}	✓		✓	
Number of slip lane on major approach	SL_1		✓		
Number of slip lane on minor approach	SL_2	✓		✓	✓
Presence of median island on major approach	MI_1		✓	✓	✓
Presence of median island on minor approach	MI_2	✓			✓
Speed limit on major approach	V_i	✓			

A statistical summary of all candidate independent variables considered in the analysis and the manner in which they are defined in the dataset is shown in Table 4.3. As shown, among the seventeen variables, there are three manners to present the independent variables: count, continues, and categorical (or dummy) variable. It should be noted that the intersection data were analysed as one group rather than separating the data into two groups, i.e., signalised or un-signalised intersections. This is because one of the strategies would involve changing the traffic control at the intersections, and it was considered preferable to use the data as one group (Chen et al. 2012; Gomes et al. 2012). The dataset was used to estimate the model parameters as described in next section 4.3.2.

Table 4.3 Statistical summary of intersection dataset

Variable	N	Min.	Max.	Mean	Std. Deviation	SPSS labelling	Variable Type
No. of legs	106	3	4	3.67	0.473	Lg_i	Count
No. of through lanes-entering							
Major-approach	106	2	5	3.46	0.886	LN_{i1}	Count
Minor-approach	106	0	4	2.40	1.478	LN_{i2}	Count
No. of through lanes-exiting							
Major-approach	106	2	5	3.20	0.960	LE_{i1}	Count
Minor-approach	106	1	4	2.03	0.980	LE_{i2}	Count
Traffic control type	106	0	1	0.58	0.495	TC	Categorical
No. of left turn lanes							
Major-approach	106	0	2	0.12	0.407	LT_1	Count
Minor-approach	106	0	2	0.16	0.417	LT_2	Count
No. of right turn lane							
Major-approach	106	0	2	0.75	0.906	RT_1	Count
Minor-approach	106	0	2	0.47	0.783	RT_2	Count
AADT [a]							
Major-approach	106	4,500 (8.41)	21,784 (9.99)	12,546 (9.36)	4,630 (0.399)	Q_{major}	Continuous
Minor-approach	106	1,600 (7.38)	14,837 (9.60)	5,769 (8.51)	3,199 (0.550)	Q_{minor}	Continuous
No. of slip lanes							
Major-approach	106	0	2	0.29	0.617	SL_1	Count
Minor-approach	106	0	2	0.19	0.537	SL_2	Count
Presence of median island							
Major-approach	106	0	1	0.46	0.501	MI_1	Categorical
Minor-approach	106	0	1	0.28	0.453	MI_2	Categorical
Speed limit (km/h)$_{Major}$	106	40	60	59.06	3.787	V_i	Continuous

[a] AADT = Annual Average Daily Traffic.

4.3.2 Modelling and Measuring Goodness-of-Fit

The CPMs were developed using a generalised linear modelling (GLM) approach. Two types of GLM were identified for use in this study: negative binomial (NB) and Poisson distributions. As mentioned previously, these two types are appropriate for analysing crash data (Lord and Mannering 2010, Abdul Manan *et al.* 2013). In order to find which of these two models was suitable for estimating safety outcomes, the study adopted the over-dispersion assumption. This assumption was discussed in Chapter 3. Initially, the distributions of crash counts were assumed to follow a negative binomial distribution that deals with over-dispersion within the datasets. Table 4.4 shows the parameter estimates, statistical significance of the intercept and predictor variables, and dispersion (K) estimates for each model. The intercept shows the estimated number of road crashes when all variables are kept at zero. In Model I, II, III, and IV the dispersion coefficients are estimated to be 0.210, 0.102, 0.330, and

0.271, respectively. As described early in Chapter 3, when the dispersion (K) value is positive and greater than zero i.e. K> 0.0, over-dispersion is indicated and the negative binomial model appropriate.

Table 4.4 Negative Binomial parameter estimates for selected models

Variable	**Model I**		**Model II**		**Model III**		**Model IV**	
	β	***P*-Value** [b]	**β**	***P*-Value** [b]	**β**	***P*-Value** [b]	**β**	***P*-Value** [b]
Intercept	-9.251	.000	-1.536	.000	-4.094	.013	-1.300	0.006
No. of Legs (Lg_i)	.622	.000	-	-	-	-	-	-
No. of through lanes Entering								
Major-approach (LN_{i1})	-	-	-	-	-	-	.398	.000
Minor-approach (LN_{i2})	-	-	-	-	.116	.028	-	-
No. of through lanes Exiting								
Major-approach (LE_{i1})	-	-	.448	.000	.146	.006	-	-
Minor-approach (LE_{i2})	-	-	.166	.002	-	-	-	-
Traffic control [c] (TC)	-	-	-	-	-	-	-.136	.588
No. of left turn lane								
Major-approach (LT_1)	.056	.091	.298	.041	-	-	.472	.031
Minor-approach (LT_2)	-	-	-	-	-.075	.000	-	-
No. of right turn lane								
Major-approach (RT_1)	-.034	.005	-	-	-	-	-	-
Minor-approach (RT_2)	-	-	-	-	-.067	.473	.231	.124
Ln(AADT)								
Major-approach (Q_{major})	.283	.144	-	-	-	-	-	-
Minor-approach (Q_{major})	.281	.098	-	-	.430	.023	-	-
No. of Slip lanes								
Major-approach (SL_1)	-	-	-.068	.707	-	-	-	-
Minor-approach (SL_2)	.316	.000	-	-	.247	.000	.021	.000
Median island [d]								
Major-approach (MI_1)	-	-	-.560	.004	-.154	.270	-.597	.013
Minor-approach (MI_2)	-.329	.016	-	-	-	-	.392	.149
Speed Limit (km/hr)$_{Major}$ (V_i)	.038	.000	-	-	-		-	-
Dispersion (K)	.210 [a]		.102 [a]		.330 [a]		.271 [a]	

[a] Computed based on the Pearson Chi-square
[b] significance at 0.1 level
[c] Traffic control =1 if Signalized; =0 if Un-signalized
[d] Median island = 1 if present; = 0 if not present

Table 4.5 provides the four models selected as suitable models based on statistical significance, goodness-of-fit, and Pearson correlation value.

Table 4.5 Summary of the selected models to estimate intersection crashes

Model No.	Model Form
I	$N_{pre.i} = Q_{major}^{.283} \cdot Q_{minor}^{.281} \cdot e^{(-9.251 + .622\,Lg_i + .056\,LT_1 - .034\,RT_1 + .316\,SL_2 - .329\,MI_2 + .038\,V_i)}$
II	$N_{pre.i} = e^{(-1.536 + .448\,LN_{i1} + .116\,LE_{i2} + .298\,LT_1 - .068\,SL_1 - .560\,MI_1)}$
III	$N_{pre.i} = Q_{minor}^{.430} \cdot e^{(-4.094 + .116\,LN_{i2} + .146\,LE_{i1} - .075\,LT_2 - .067\,RT_2 + .247\,SL_2 - .154\,MI_1)}$
IV	$N_{pre.i} = e^{(-1.300 + .398\,LN_{i1} + .136\,TC + .472\,LT_1 + .231\,RT_2 + .021\,SL_2 - .597\,MI_1 + .392\,MI_2)}$

$N_{pre,i}$ = predicted crashes number at i^{th} intersection

In addition, a goodness-fit-test (discussed in Chapter 3) using deviance, Pearson chi-square (x^2), degree of freedom (*df*), Akaike's Information Criterion (AIC), Bayesian Information Criterion (BIC), Residual values, and Cumulative residual (CURE) values was used to test the model assumption and to indicate how well the data fitted the model. The values of *Deviance/ Degree of freedom* and *Pearson chi-square / Degree of freedom* should range between 0.8 - 1.2 to consider the negative binomial model appropriate and the model would fit the data well (Bauer & Harwood 2000; Maina 2009; Abdul Manan et al. 2013). Table 4.6 shows that the values of *Deviance/ Degree of freedom* and *Pearson chi-square / Degree of freedom* for all developed models are within permissible range. These results show that the Negative Binomial (NB) distribution assumption is acceptable for each of the four models.

Table 4.6 Goodness of fit tests for negative binomial models (Intersection)

Model	Parameter	Value	*df* [a]	Value/*df*
I	Deviance	81.126	96	0.845
	Pearson Chi-Square	79.470		0.825
	Akaike's Info. Criterion (AIC)	254.166		.
	Bayesian Info. Criterion (BIC)	280.801		.
II	Deviance	103.509	100	1.035
	Pearson Chi-Square	94.263		0.943
	Akaike's Info. Criterion (AIC)	287.110		.
	Bayesian Info. Criterion (BIC)	303.090		.
III	Deviance	91.564	99	0.925
	Pearson Chi-Square	80.063		0.809
	Akaike's Info. Criterion (AIC)	294.754		.
	Bayesian Info. Criterion (BIC)	313.398		.
IV	Deviance	92.836	98	0.947
	Pearson Chi-Square	79.329		0.809
	Akaike's Info. Criterion (AIC)	295.419		.
	Bayesian Info. Criterion (BIC)	316.727		.

[a] *df* = degree of freedom

In Model I, all the predictor variables are significant (at 0.1) except for Annual Average Daily Traffic (AADT) on major approach. In the same way, in Model II all the predictor variables are significant except for the number of slip lanes on a major approach. Predictor variables in Model III are significant except for the number of right turn lanes on minor approaches and the presence of a median island on major approaches. Model IV is significant except for traffic control type, number of right turn lanes on minor approaches, and presence of median island on minor approaches.

Using the values of AIC and BIC from Table 4.6, the models were ranked starting with the best model as follows: Model I, Model II, Model III, and Model IV. The smaller the AIC and BIC values, the more preferred the model (Cafiso et al. 2010; Abdul Manan et al. 2013; Young & Park 2013).

The residual is the difference between the actual and predicted number of road crashes and this value could be used to identify the appropriate model that best fits the data. The quality of fit was also investigated using the residual values and cumulative residual values. Figure 4.3 illustrates the plot of the residual versus Log-AADT on the major approaches. When the residuals value fluctuates around the zero value and the residual are not widely spread, this indicates that the model fits the data well. From Figure 4.3, it is observed that the Model I is more appropriate than other models because it has the smallest spread among all models, where the residuals for Model I range from -1.41 to 3.75. Furthermore, the average spread of the residuals for the Model I was 0.57, while for Model II, Model III, and Model IV it was 0.75, 0.59, and 0.76, respectively.

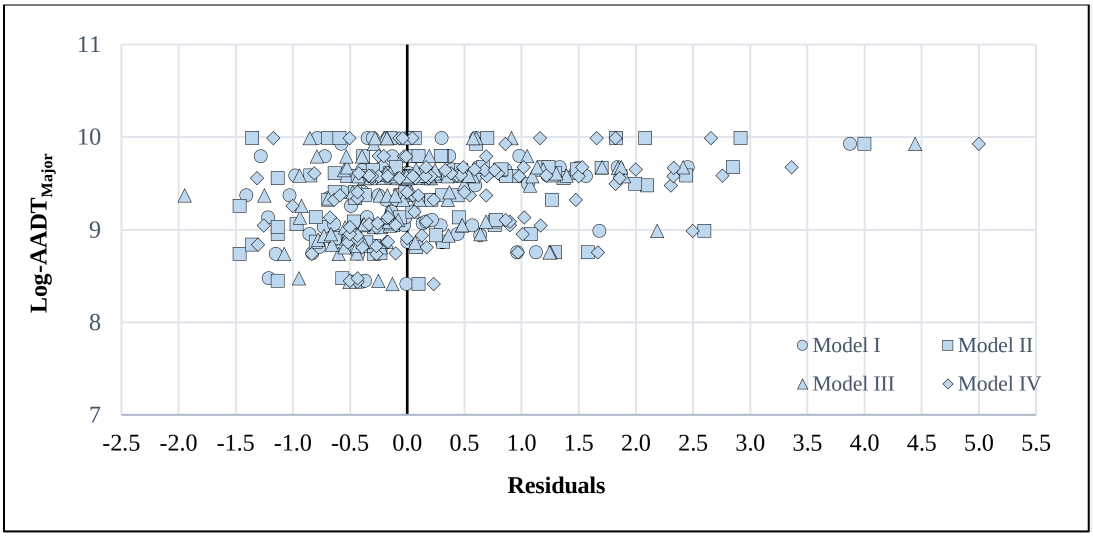

Figure 4.3 Plot of the Residuals with Log-AADT on the major approach

In addition, to better assess the quality of crash prediction models, it is useful to develop the cumulative residual (CURE) plots (Young & Park 2013; Hauer 2015). These plots reveal how well the predicted models fit the data with respect to each explanatory variable separately. In this analysis, the AADT on the major approaches has been adopted as a representative explanatory variable. In general, when the model fits the data well, the CUREs should fluctuate randomly around the zero residual line and be located within the standard deviation boundaries (±2σ). Figure 4.4 shows the CURE plots for all developed models. It can be noticed that all developed models fluctuate around the zero line and within ±2σ boundaries. Moreover, Model I shows more fluctuation around the zero residual line compared to the other models.

Ultimately, in this section, Model I was selected as the one with the best statistical fit, as it outperformed the others based on the evaluation measures including AIC, BIC, residual values, and CURE values.

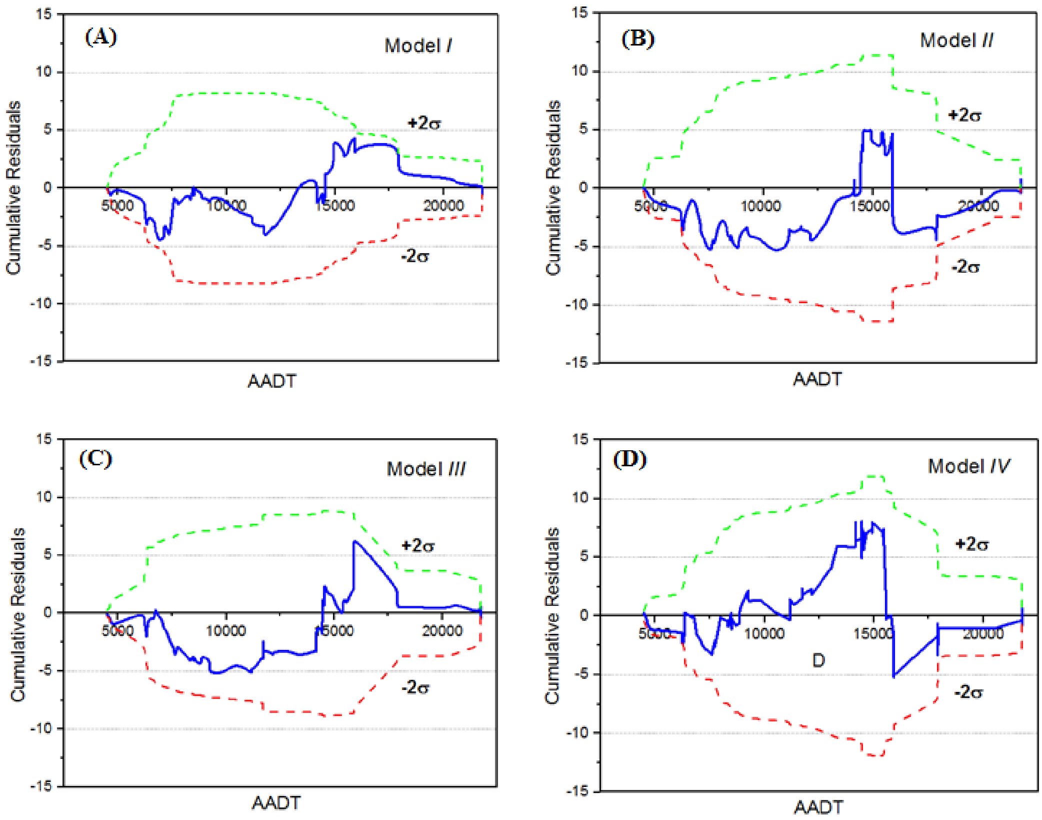

Figure 4.4 Cumulative residual (CURE) plots for intersection models. (A) Model *I*. (B) Model *II*. (C) Model *III*. (D) Model *IV*

4.3.3 Model Validation

After developing all Crash Prediction Models (CPMs) using the intersections data, the prediction ability of each model was tested using four performance measures discussed earlier in Chapter 3: Mean Squared Prediction Error (MSPE), Mean Absolute Deviation (MAD), Mean Squared Error (MSE), and Freeman-Tukey R-Squared coefficient (R^2_{FT}). Table 4.7 shows the performance for all crash prediction models based on the estimation dataset (2008-2013) and the validation dataset (2014-2015). It can be seen that the values of MSPE using the validation dataset and MSE using the estimation dataset are close to each other. In addition, the values of MAD using both datasets are similar. The R^2_{FT} test results were slightly different for the estimation datasets compared to the validation datasets. The overall results indicate that the four selected models have demonstrated the ability to estimate the road crashes reasonably over additional years.

Table 4.7 Performance measures for all crash prediction models

Performance measures	Model I		Model II		Model III		Model IV	
	2008-13[a]	2014-15[b]	2008-13[a]	2014-15[b]	2008-13[a]	2014-15[b]	2008-13[a]	2014-15[b]
MSPE	-	0.527	-	1.109	-	0.624	-	1.262
MSE	0.691	-	1.289	-	0.790	-	1.425	-
MAD	0.569	0.516	0.763	0.781	0.585	0.523	0.768	0.785
R^2_{FT} %	49.0	45.7	45.2	41.4	41.9	35.0	18.1	22.7

[a] Calculated based on estimation dataset 2008-2013
[b] Calculated based on validation dataset 2014-2015

Overall, based on the outcome from the goodness-of-fit measures described previously, all models can be accepted for further analysis (e.g., estimated CMFs). Model I as the best-fitted model was subsequently used to calculate the expected road crash frequency.

4.4 High-Risk Intersections

In this section, the Empirical Bayes (EB) approach was applied separately using Model I to identify the high-risk intersections or black spot sites in the study area. In the first step, Model I was used to estimate the predicted number of crashes for each intersection. In the second step, the weighting adjustment (ω) was calculated using the over-dispersion parameter (K) and the predicted number of crashes using the study period (2008-2013). In the third step, the expected number of crashes was estimated by combining the predicted number of crashes from Model I with the observed number of crashes (at study area) using the weighted adjustment factors. Finally, the potential for safety improvements (PSI) was calculated for ranking the intersections.

4.4.1 Identifying and Ranking High-Risk Intersections

The Highway Safety Manual (HSM) (AASHTO 2010) indicates that the advantage of using the CPMs is that the user will obtain a value for a long-term predicted crash number rather than a short-term observed crash number. Specifically, the expected number of crashes using EB adjustments was used in this study to increase the accuracy of safety estimation by accounting for the regression-to-mean (RTM) bias usually associated with road crash data. RTM is the possible bias caused by identifying the black spot sites for treatment, which look hazardous based on short-term observations (AASHTO 2010; Lu 2013). Table 4.8 presents the expected crash frequency as a weighted average of the predicted and observed number of crashes.

The Empirical Bayes (EB) approach is useful for ranking the intersections to identify the most hazardous intersections (i.e. black spot sites) that may require crash remedial measures. This ranking method depends on the values of potential for safety improvement (PSI), which were calculated as the difference between the expected and predicted number of crashes. Based on the PSI values the intersections were ranked, starting from the most hazardous ranked intersection as shown in Table 4.8. The positive value of PSI (i.e. PSI > 0.0, as the expected crash number is more than the predicted crash number) indicates that a potential for safety improvement exists. Likewise, the zero or negative value (i.e. PSI ≤ 0.0, as the expected crash number is less than or equal to the predicted crash number) indicates that no or limited potential for safety improvement exists. In Table 4.8, the first 44 intersections had positive values of PSI and 62 intersections had zero and negative values of PSI. From the predictions, the most dangerous intersection needing safety improvement was I_NW9 Bridge and Tor Streets and the safest one was I_NW28 Taylor and McDougall Streets. Appendix A provides the details of the outcomes for all intersections.

Table 4.8 Ranking intersections for safety improvement

Intersection ID	Observed Mean [a]	Predicted (cr./year)	EB Weighted Adjustment(w)	Expected (cr./year)	PSI	Rank
I_NW9	6.67	2.79	0.22	5.86	3.016	1
I_NE5	4.83	2.38	0.25	4.22	1.840	2
I_SE12	3.67	2.10	0.27	3.24	1.136	3
I_NW15	3.33	1.65	0.32	2.79	1.135	4
I_NE6	3.00	1.16	0.41	2.25	1.093	5
I_NW6	3.50	2.01	0.28	3.08	1.071	6
I_NE4	3.17	1.83	0.30	2.76	0.930	7
I_SW19	3.40	2.17	0.27	3.07	0.899	8
I_NW5	3.33	2.28	0.26	3.06	0.784	9
I_NE28	2.17	0.70	0.53	1.39	0.687	10
I_NW1	3.17	2.31	0.26	2.95	0.638	11
I_NW20	2.00	0.87	0.48	1.46	0.590	12
I_SW6	2.67	1.82	0.30	2.41	0.590	13
I_NE10	2.17	1.21	0.40	1.79	0.579	14
I_NE19	2.00	1.02	0.44	1.57	0.551	15
I_SW8	4.17	3.56	0.18	4.06	0.494	16
I_NE3	3.17	2.55	0.24	3.02	0.474	17
I_SW4	2.83	2.24	0.26	2.68	0.439	18
I_SW10	2.17	1.57	0.34	1.97	0.398	19
I_NW16	2.33	1.76	0.31	2.16	0.393	20
I_NW8	3.33	2.94	0.21	3.25	0.309	21
I_NW7	1.50	1.00	0.44	1.28	0.279	22
I_NE2	1.50	1.02	0.44	1.29	0.269	23
I_SE8	1.50	1.06	0.43	1.31	0.252	24
I_SW15	1.33	0.87	0.48	1.12	0.240	25
I_SW14	2.50	2.20	0.27	2.42	0.221	26
I_NW19	1.00	0.36	0.69	0.56	0.200	27
I_NW21	1.17	0.78	0.50	0.97	0.190	28
I_NE17	1.17	0.80	0.50	0.98	0.185	29
I_NW17	1.17	0.86	0.48	1.02	0.159	30
I_NW18	1.17	0.88	0.48	1.03	0.153	31
I_SE10	2.33	2.14	0.27	2.28	0.138	32
I_NE26	1.50	1.28	0.38	1.42	0.133	33
I_NE13	1.50	1.33	0.37	1.44	0.105	34
I_SW7	1.17	1.00	0.44	1.09	0.093	35
I_NE9	1.17	1.00	0.44	1.09	0.092	36
I_SE11	1.00	0.85	0.48	0.93	0.078	37
I_NW25	1.00	0.91	0.47	0.96	0.049	38
I_SW3	0.83	0.74	0.52	0.78	0.046	39
I_SW22	0.50	0.37	0.68	0.41	0.042	40
I_NW13	1.00	0.93	0.46	0.97	0.040	41
I_SE9	1.00	0.93	0.46	0.97	0.038	42
I_NW23	0.83	0.81	0.49	0.82	0.012	43

[a] The mean of the observed crash frequency during the study period 2008-2013

Table 4.8 Ranking intersections for safety improvement (continue)

Intersection ID	Observed Mean [a]	Predicted (cr./year)	EB Weighted Adjustment(w)	Expected (cr./year)	PSI	Rank
I_NE8	1.33	1.32	0.38	1.33	0.011	44
I_NE21	0.83	0.83	0.49	0.83	0.000	45
I_SW16	0.83	0.84	0.49	0.84	-0.004	46
I_NE14	0.67	0.68	0.54	0.67	-0.005	47
I_SE15	1.67	1.68	0.32	1.67	-0.010	48
I_NE20	0.67	0.70	0.53	0.68	-0.014	49
I_SW23	1.50	1.53	0.34	1.51	-0.018	50
I_NW12	1.67	1.70	0.32	1.68	-0.023	51
I_NW29	0.50	0.57	0.58	0.54	-0.028	52
I_NW10	0.67	0.77	0.51	0.72	-0.050	53
I_SE17	0.67	0.79	0.50	0.73	-0.060	54
I_SW2	2.00	2.09	0.28	2.03	-0.067	55
I_SW18	0.67	0.80	0.50	0.73	-0.069	56
I_SE18	0.83	0.96	0.45	0.89	-0.071	57
I_NE1	1.50	1.63	0.33	1.54	-0.088	58
I_SW1	0.83	1.02	0.44	0.91	-0.104	59
I_NW30	1.20	1.39	0.36	1.27	-0.119	60
I_SE13	2.67	2.83	0.22	2.70	-0.127	61
I_SE14	3.33	3.51	0.18	3.37	-0.142	62
I_SW12	0.50	0.81	0.50	0.65	-0.155	63
I_NE7	2.00	2.22	0.26	2.06	-0.162	64
I_NW11	2.50	2.71	0.23	2.55	-0.166	65
I_NW34	0.50	0.83	0.49	0.66	-0.166	66
I_SE6	0.17	0.57	0.58	0.40	-0.167	67
I_NW24	1.33	1.58	0.33	1.42	-0.167	68
I_SE5	0.33	0.69	0.53	0.53	-0.167	69
I_NW26	0.67	1.03	0.44	0.82	-0.205	70
I_NE27	0.67	1.04	0.43	0.83	-0.209	71
I_NE15	0.33	0.76	0.51	0.55	-0.211	72
I_NE18	1.00	1.35	0.37	1.13	-0.221	73
I_NW2	1.50	1.83	0.30	1.60	-0.231	74
I_SE3	0.33	0.80	0.50	0.56	-0.232	75
I_SW11	3.17	3.47	0.19	3.22	-0.246	76
I_NW3	0.33	0.82	0.49	0.57	-0.250	77
I_NE12	1.33	1.71	0.32	1.45	-0.254	78
I_NE11	2.00	2.34	0.25	2.09	-0.254	79
I_SW13	1.83	2.18	0.27	1.93	-0.255	80
I_SW5	1.17	1.55	0.34	1.30	-0.257	81
I_SW9	0.17	0.72	0.52	0.46	-0.267	82
I_NW4	1.50	1.89	0.30	1.62	-0.278	83
I_NW22	0.17	0.78	0.50	0.48	-0.305	84
I_SE7	0.17	0.79	0.50	0.48	-0.309	85
I_SE1	0.50	1.05	0.43	0.74	-0.312	86

[a] The mean of the observed crash frequency during the study period 2008-2013

Table 4.8 Ranking intersections for safety improvement (continue)

Intersection ID	Observed Mean [a]	Predicted (cr./year)	EB Weighted Adjustment(w)	Expected (cr./year)	PSI	Rank
I_NE16	0.17	0.81	0.50	0.48	-0.323	87
I_SE20	0.17	0.86	0.48	0.50	-0.359	88
I_NW31	1.50	2.03	0.28	1.65	-0.383	89
I_NW32	1.17	1.76	0.31	1.35	-0.410	90
I_NW33	0.17	0.81	0.50	0.48	-0.323	87
I_NW27	0.17	0.86	0.48	0.50	-0.359	88
I_SE19	1.50	2.03	0.28	1.65	-0.383	89
I_SW21	1.17	1.76	0.31	1.35	-0.410	90
I_SE2	0.83	1.47	0.35	1.06	-0.416	91
I_SW17	1.50	2.08	0.28	1.66	-0.418	92
I_NE24	0.17	0.95	0.45	0.52	-0.429	93
I_NE22	0.67	1.40	0.36	0.93	-0.465	94
I_SE21	0.50	1.27	0.38	0.80	-0.476	95
I_SW20	0.50	1.33	0.37	0.81	-0.520	96
I_NE25	1.33	2.05	0.28	1.53	-0.520	97
I_SE4	1.00	1.78	0.31	1.24	-0.537	98
I_NW14	0.67	1.52	0.34	0.96	-0.564	99
I_SE16	0.83	1.81	0.30	1.13	-0.681	100
I_NE23	0.33	1.48	0.35	0.73	-0.750	101
I_NW28	0.17	1.38	0.37	0.61	-0.768	102
I_NE16	1.50	2.53	0.24	1.75	-0.784	103
I_SE20	0.83	2.05	0.28	1.17	-0.878	104
I_NW31	1.00	2.28	0.26	1.33	-0.950	105
I_NW32	1.00	2.41	0.25	1.35	-1.058	106

[a] The mean of the observed crash frequency during the study period 2008-2013

4.5 Crash Modification Factors for Intersection Crashes

As mentioned earlier, crash modification factor (CMF) is a value representing the change in road safety after modifying the geometric design or operation of the facility. In general, CMFs can be estimated using different methods. The first method is based on a cross-sectional study of sites with and without the component (e.g. presence or absence of a median island). The second method is based on observations before and after where a specific safety improvement has been implemented. The third method is based on the opinion consensus of a panel of highway design and safety experts to determine the expected safety effect of a specific countermeasure. A newer method used in recent years, as part of a cross-sectional method, is to estimate the CMFs based on the CPMs and is called crash modification function (CMFunction) (Lord & Bonneson 2007; Park et al. 2014). This method was used in this study to estimate the

CMFs. All of these methods were discussed in detail in the literature review (Chapter 2).

4.5.1 Crash Modification Function

The crash modification function (CMFunction) method was used to estimate the road safety effect for all independent variables that were used in the development of CPMs to measure the effect of the suggested treatments on the road safety at the intersections. It is important to consider a base value for using developed CPMs to estimate crashes to reflect conditions after a treatment. The base conditions for all geometric and traffic characteristics that were analysed in this study were identified based on the previous studies and/or the mean values of the dataset. Table 4.9 gives the base values that were adopted for the intersection features in this study. However, the base condition for individual intersections may take different values to accommodate specific site conditions, and therefore they need to be adjusted to accommodate the actual site condition.

Table 4.9 Base conditions for different design elements for the intersection

Feature	**Base Values**
Number of intersection legs	4 legs
Number of through lanes entering	2 lanes per approach
Number of through lanes exiting	2 lanes per approach
Type of traffic control	0 (un-signalized)
Number of left turn lanes	0 (without left lane)
Number of right turn lanes	0 (without right lane)
AADT on major approach	12,000 vehicle per day
AADT on minor approach	6,000 vehicle per day
Number of slip lanes	0 (without slip lane)
Presence of median island	0 (without median)
Speed Limit	60 km/hr

Using these base values and variables parameters associated with the treatment type, the CMFs and standard error (Std. Er.) for each treatment. When the value of Std. Er. equals 0.1 or less this indicates that an estimated CMF is more accurate. Suitable models from Table 4.5 were then used to define CMFunction to estimate CMFs for proposed safety treatments, as detailed below:

Number of Intersecting Legs

CMFs were derived from Model I based on the number of intersection legs. The 4-legged intersection was used as a base condition to estimate CMFs as shown in Table 4.10. The results show that the intersections with fewer legs were associated with lower crash numbers. For instance, when an intersection changed from 4-legged to 3-legged intersection the number of crashes was reduced by 46%. This result was expected because usually the traffic volume and vehicle interactions are higher at intersections with more legs.

Table 4.10 CMFs based on the number of intersection legs

CMFunction	***Lg_i***	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{0.622\times[Lg_i-4]}$ (Base condition at 4-legs)	3	0.54	0.084
	4	1.00	0.157

[a] Estimated using model I

Number of Through Lanes Entering

The CMFs related to the number of through lanes entering on major and minor approaches were estimated using Model IV and Model III, respectively as shown in Table 4.11. In order to estimate the CMFs for the number of through lanes entering based on each entry approach, the relevant model parameters were divided by two for both major and minor approaches (Lord & Bonneson 2007; Li et al. 2010). The results indicate that the number of through lanes entering was associated with more crashes for both the major approach and the minor approach. The effect of the number of through lanes entering at a major approach is more significant than at a minor approach and this is probably due to the difference in traffic volume.

Table 4.11 CMFs based on the number of through lanes entering

CMFunction	***LN_i***	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{0.199\times[LN_1-2]}$	1	0.82	0.059	0.94	0.025
$CMFunction_{minor} = e^{0.058\times[LN_2-2]}$	2	1.00	0.073	1.00	0.026
(Base condition at 2 lanes)	3	1.22	0.089	1.06	0.028

[a] Estimated using model IV
[b] Estimated using model III

Number of Through Lanes Exiting

The CMFs related to the number of through lanes exiting were estimated for major and minor approaches using Model II and Model III, respectively, and the results are shown in Table 4.12. The independent variable for major approaches was included in both models (i.e., Model II and Model III) as shown in Table 4.4. However, Model II was selected to estimate CMFs for this variable because it has provided a better data fit than Model III. Similar to the number of through lanes entering, the CMFs were also estimated based on each approach. The results indicate that the number of through lanes exiting was associated with more crashes for both major and minor approaches. It can be seen that the effect of the number of through lanes exiting at a major approach is more significant than at a minor approach.

Table 4.12 CMFs based on the number of through lanes exiting

CMFunction	LE_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF** [b]	**Std. Er.**
$CMFunction_{major} = e^{0.224\times[LE_1-2]}$	1	0.80	0.049	0.92	0.048
$CMFunction_{minor} = e^{0.083\times[LE_2-2]}$	2	1.00	0.061	1.00	0.052
(Base condition at 2 lanes)	3	1.25	0.076	1.09	0.057

[a] Estimated using model II
[b] Estimated using model III

Traffic Control Type

The study also examined the effect of traffic control at intersections i.e., signalised and non-signalised intersections using Model IV and the results are shown in Table 4.13. The results found that adding a signal at non-signalised intersection reduced the crashes by 13%. This result agrees with previous studies (Pernia et al. 2002; Wang & Abdel-Aty 2014).

Table 4.13 CMFs based on the type of traffic control

CMFunction	TC_i	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{-0.136\times[TC-0]}$	0	1.00	0.253
(Base condition at non-signalised; 0)	1	0.87	0.221

[a] Estimated using model IV

Number of Left Turn Lanes *(exclusive lanes)*

Model I and Model III were used to estimate the CMFs for major and minor approaches, respectively, based on the goodness of fit test. The CMFs were determined based on the presence of left turn lanes on each approach i.e., each leg. The results revealed that the presence of a left turn lane at a major approach reduced road safety, while for a minor approach, the presence of a left turn lane increased road safety as shown in Table 4.14. The results also demonstrated that the presence of left turn lanes had only a slight effect on crash numbers.

Table 4.14 CMFs based on the number of left turn lanes

CMFunction	LT_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{0.028\times[LT_1 - 0]}$	0	1.00	0.071	1.00	0.074
$CMFunction_{minor} = e^{-0.038 * [LT_2 - 0]}$	1	1.03	0.073	0.96	0.072
(Base condition at 0 lane)	2	1.06	0.075	0.93	0.069

[a] Estimated using model I
[b] Estimated using model III

Number of Right Turn Lanes *(exclusive lanes)*

The CMFs were determined for the presence of an exclusive right turn lane at an intersection using the same models as in the previous paragraph i.e., number of left turn lanes. Table 4.15 shows that the presence of an exclusive right turn lane at major and minor approaches reduced the number of road crashes. As with the presence of exclusive left turn lanes, the presence of exclusive right turn lanes had a slight effect on the number of crashes.

Table 4.15 CMFs based on the number of right turn lanes

CMFunction	RT_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{-0.017\times[RT_1 - 0]}$	0	1.00	0.039	1.00	0.047
$CMFunction_{minor} = e^{-0.034\times[RT_2 - 0]}$	1	0.98	0.038	0.97	0.046
(Base condition at 0 lane)	2	0.97	0.038	0.94	0.045

[a] Estimated using model I
[b] Estimated using model III

Traffic Volume (AADT)

The likelihood of road crashes was found to increase with increasing traffic volumes on the major and minor approaches. The base condition for a major approach was 12,000 vehicles per day and for a minor approach was 6,000 vehicles per day using Model I as shown in Table 4.16. Other studies (Haleem et al. 2010; Wang & Abdel-Aty 2014; Park 2015) have also shown the same type of result when analysing road crashes at intersections. As mentioned earlier, to reflect the non-linear relationship between traffic volumes (AADT) and number of crashes, the logarithm of AADT was used. Figure 4.5 illustrates the relationship between traffic volumes and road safety for major and minor approaches, respectively. It should be noted that the value of CMF in this study is applicable to the traffic volume ranging from 4,500 to 21,800 vehicles per day for major approaches and from 1,600 to 15,000 vehicles per day for minor approaches.

Table 4.16 CMFs based on traffic volume

CMFunction	Q_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF** [a]	**Std. Er.**
$CMFunction_{major} = (Q_{major}/12{,}000)^{0.283}$	1,600	N/A	N/A	0.69	0.118
$CMFunction_{minor} = (Q_{minor}/6{,}000)^{0.281}$	6,000	0.82	0.160	1.00	0.170
(Base condition for major at 12,000 vehicles/day) (Base condition for minor at 6,000 vehicles/day)	12,000	1.00	0.195	1.22	0.207
	18,000	1.12	0.219	N/A	N/A

N/A, Non-Applicable based on the range of dataset
[a] Estimated using model I

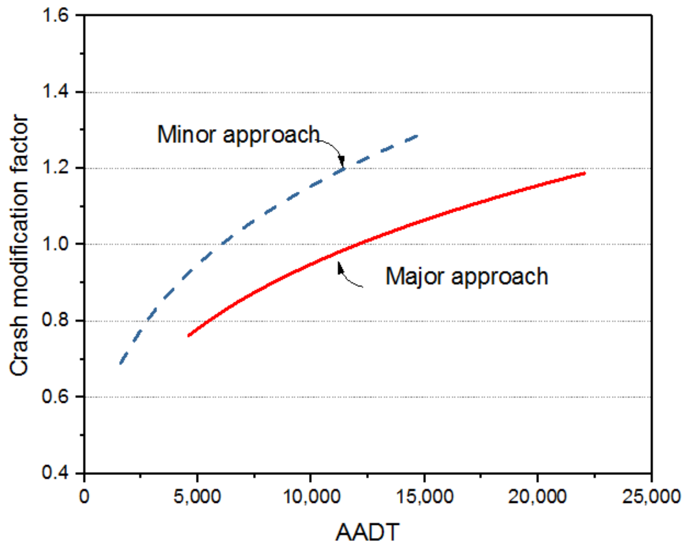

Figure 4.5 CMF for traffic volume

Number of Slip Lanes

Table 4.17 shows the values of CMFs for major and minor approaches using Model II and Model I, respectively. Regression parameters were divided by two to estimate the effect of slip lanes for each direction on major and minor approaches. The presence of a slip lane on a minor approach is associated with increased crash risk, due to the creation of more merging conflicts between the vehicles that use the slip lane with oncoming traffic from the major approach. The crash numbers were reduced after installing a slip lane on a major approach but not to a significant level. This is because the vehicles using the slip lane would merge with a low oncoming traffic volume from the minor approach.

Table 4.17 CMFs based on the number of slip lanes

CMFunction	***SL***	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{-0.034\times[SL_1-0]}$	0	1.00	0.052	1.00	0.091
$CMFunction_{minor} = e^{0.158\times[SL_2-0]}$	1	0.97	0.050	1.17	0.107
(Base condition at NO Slip lane)					

[a] Estimated using model II
[b] Estimated using model I

Presence of Median Island

The presence of a median island (raised median) on the major or minor approaches at intersections is associated with a reduced crash risk. Model II and Model I were used for major and minor approaches, respectively. The CMFs were estimated based on the presence of a median island on each approach i.e., each leg. The study found that the intersection approach with a median island has reduced the crash risk by 24% and 15% in major and minor approaches, respectively. The results in Table 4.18 indicate that a median island in a major approach has more effect on road safety than a median island in a minor approach and this result relates to the difference in traffic volume.

Table 4.18 CMFs based on the presence of a median island on one approach

CMFunction	***MIi***	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{-0.280\times[MI_1-0]}$	0	1.00	0.116	1.00	0.068
$CMFunction_{minor} = e^{-0.164\times[MI_2-0]}$	1	0.76	0.087	0.85	0.058
(Base condition at NO median)					

[a] Estimated using model II
[b] Estimated using model I

Speed Limit

Model I was used to estimate the effect of CMFs based on 60 km/hr as a base condition. Higher speed limits on major approaches were associated with higher road crashes compared with lower speed limits as shown in Table 4.19. Previous studies by Haleem et al. (2010) and Haque et al. (2010) have also found that intersection approaches with higher speed limits have a higher crash probability. Figure 4.6 illustrates the relationship between speed limit and road safety. The value of CMF is applicable to the posted speed limit ranging from 40 km/hr to 60 km/hr.

Table 4.19 CMFs based on the speed limit

CMFunction	V_i	CMF [a]	Std. Er.
$CMFunction = e^{0.038\times[V_i-60]}$ (Base condition at 60 km/hr)	40	0.47	0.010
	60	1.00	0.022

[a] Estimated using model I

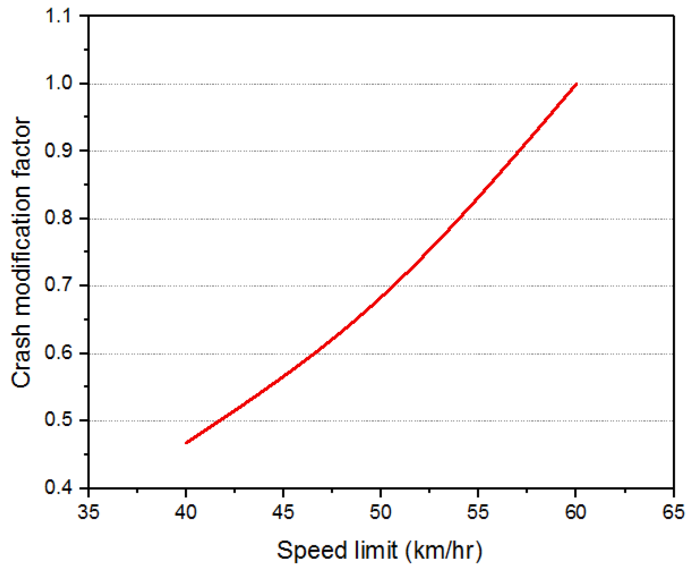

Figure 4.6 CMF for speed limit

Summary of the effects of Independent Variables

Table 4.20 provides a summary of the estimated safety effects of various explanatory variables at road intersections. This table shows the most and least effective variables on safety performance based on CMF results.

Table 4.20 Summary of the CMF results for intersection

Explanatory variables	Effect on safety performance		Comment
	Positive Effect	Negative Effect	
No. of legs		✓	Significant
No. of through lanes-entering			
Major-approach		✓	Significant
Minor-approach		✓	Insignificant
No. of through lanes-exiting			
Major-approach		✓	Significant
Minor-approach		✓	Insignificant
Traffic control type	✓		Significant
No. of left turn lanes			
Major-approach		✓	Insignificant
Minor-approach	✓		Insignificant
No. of right turn lane			
Major-approach	✓		Insignificant
Minor-approach	✓		Insignificant
AADT [a]			
Major-approach		✓	Significant
Minor-approach		✓	Significant
No. of slip lanes			
Major-approach	✓		Insignificant
Minor-approach		✓	Insignificant
Presence of median island			
Major-approach	✓		Significant
Minor-approach	✓		Significant
Speed limit (km/h)$_{Major}$		✓	Significant

4.6 Combined CMFs for Intersection Crashes

The top ten hazardous intersections have been identified using the Empirical Bayes (EB) method as presented earlier in Table 4.8. The properties of these intersections and operational conditions were incorporated to determine the possible treatments for each intersection, where CMFs were estimated for a single suggested treatment. The next step undertaken was to analyse the combined CMFs for multiple treatments using the four techniques discussed earlier in Chapter 3. The first technique was adopted by HSM (AASHTO 2010) and this technique assumed that each treatment is independent of other treatments. The second technique was introduced by Turner (2011) and in this technique, the specific weighted factor applied to the multiplication of the CMFs. The third technique was introduced by the US State of Alabama (NCHRP 2008), and assumed that the safety effects of the less effective treatment are systematically reduced. The fourth technique applied only the most effective safety treatment i.e., lowest CMF. The fourth technique was also proposed based on the survey performed by (NCHRP 2008). After reviewing related studies (Chapter 2), it can be observed that the combined CMFs results from the four existing techniques are different. Also, the related studies did not identify which of the four techniques provides best estimation of multiple treatments. Thus, the average of these four techniques (adjustment approaches) was adopted to estimate the effect of multiple treatments using the values of CMFs for single treatments. This approach was also adopted to avoid skewed benefit-cost outcomes.

4.6.1 Intersections Characteristics

This section considers the properties of the top ten hazardous intersections to identify and propose treatments for safety improvements.

1) Intersection of Bridge Street and Tor Street (I_NW9)

Figure 4.7 shows a 4-legged signalised intersection; where the major approaches (both approaches) have a total of four through lanes entering, four through lanes exiting, and two right turn lanes. The minor approaches have a total of four through lanes entering and three through lanes exiting (for both approaches). In addition, the major approaches have a raised median island and one slip lane on each approach, the minor approaches have one slip lane on one approach. The dots represent the severe crashes that occurred between 2008 and 2015. The traffic volumes on the major and minor approaches were 20,500 and 6,200 vehicles per day, respectively.

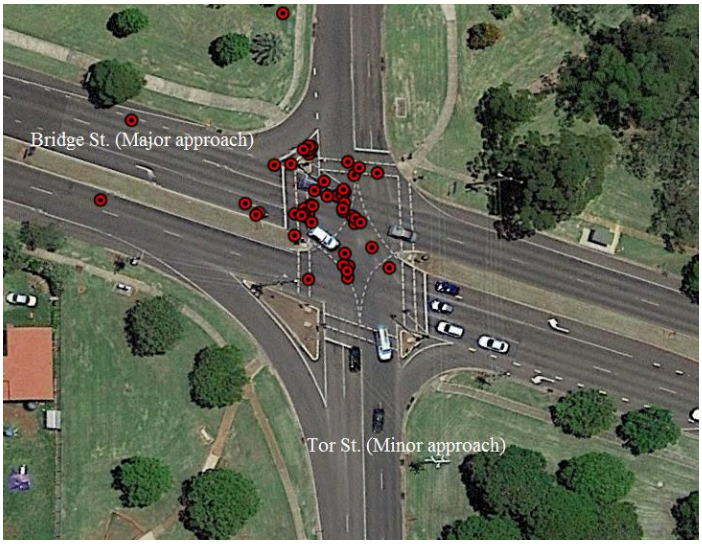

Figure 4.7 Intersection I_NW9 between Bridge Street and Tor Street
Source: Aerial Image from Google Earth Pro

2) Intersection of James Street and Hume Street (I_NE5)

Figure 4.8 shows a 4-legged signalised intersection, where the major approaches have four through lanes entering and four through lanes exiting (for both approaches). The minor approaches have a total of four through lanes entering, four through lanes exiting for both approaches and one right turn lane on one approach. Moreover, only one slip lane exists on one major approach and there is no raised median island on both major and minor approaches. During the study period, the average traffic volumes on the major and minor approaches were 15,900 and 10,900 vehicles per day, respectively.

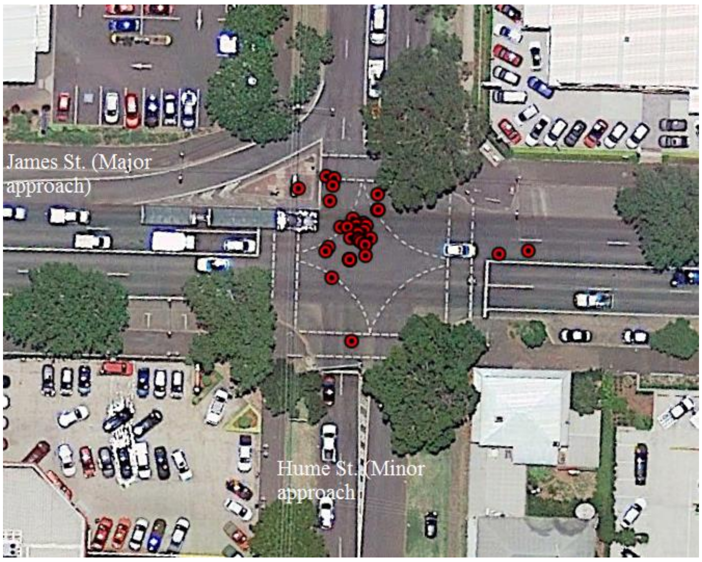

Figure 4.8 Intersection I_NE5 between James Street and Hume Street
Source: Aerial Image from Google Earth Pro

3) Intersection of Ruthven Street and South Street (I_SE12)

Figure 4.9 shows a 4-legged un-signalised intersection with stop sign and give-way sign on minor approaches. The major approaches have a total of four through lanes entering and four through lanes exiting, while the minor approaches have two through lanes entering and a two through lanes exiting (for both approaches). In addition, there is no median island exist on both major and minor approaches. The traffic volumes on the major and minor approaches were 14,400 and 7,700 vehicles per day, respectively.

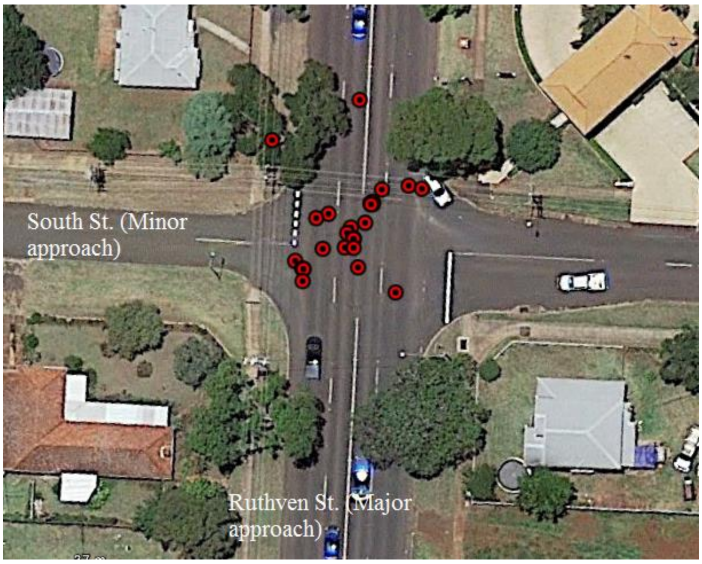

Figure 4.9 Intersection I_SE12 between Ruthven Street and South Street
Source: Aerial Image from Google Earth Pro

4) Intersection of Boundary Street and Hursley Road (I_NW15)

Figure 4.10 shows a 4-legged signalised intersection, where the major and minor approaches have a total of two through lanes entering, two through lanes exiting, and two right turn lanes for each one in both directions. Moreover, there is no raised median island and slip lane on major and minor approaches. The traffic volumes on the major and minor approaches were 8,000 and 7,600 vehicles per day, respectively.

Figure 4.10 Intersection I_NW15 between Boundary Street and Hursley Road
Source: Aerial Image from Google Earth Pro

5) Intersection of James Street and Geddes Street (I_NE6)

Figure 4.11 shows a 4-legged un-signalised intersection with stop sign on minor approaches. The major approaches have a total of four through lanes entering and four through lanes exiting on both directions, while the minor approaches have two left turn lanes and two through lanes exiting. In addition, there is no median island on major approaches. The traffic volumes on the major and minor approaches were 15,900 and 2,700 vehicles per day, respectively.

Figure 4.11 Intersection I_NE6 between James Street and Geddes Street
Source: Aerial Image from Google Earth Pro

6) Intersection of West Street and Margaret Street (I_NW6)

Figure 4.12 shows a 4-legged signalised intersection, where the major approaches have a total of four through lanes entering, four through lanes exiting, and two right turn lanes. The minor approaches have a total of four through lanes entering and two through lanes exiting. Moreover, there is no raised median island and slip lane on both major and minor approaches. The traffic volumes on the major and minor approaches were 15,800 and 7,600 vehicles per day, respectively.

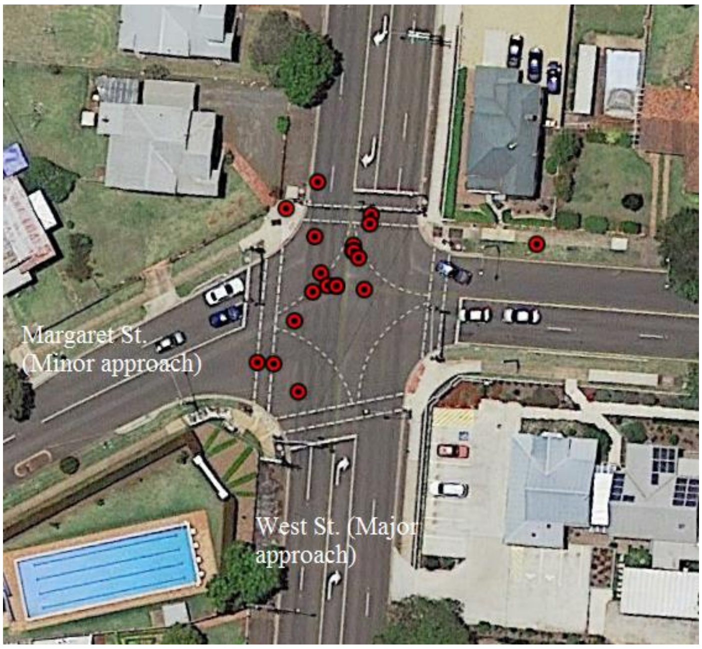

Figure 4.12 Intersection I_NW6 between West Street and Margaret Street
Source: Aerial Image from Google Earth Pro

7) Intersection of James Street and Neil Street (I_NE4)

Figure 4.13 shows a 4-legged signalised intersection, where the major approaches have a total of four through lanes entering, four through lanes exiting, and two left turn lanes. In addition, the minor approaches have a total of four through lanes entering and two through lanes exiting. Moreover, there is no raised median island or slip lane on both major and minor approaches. The traffic volumes on the major and minor approaches were 15,900 and 2,900 vehicles per day, respectively.

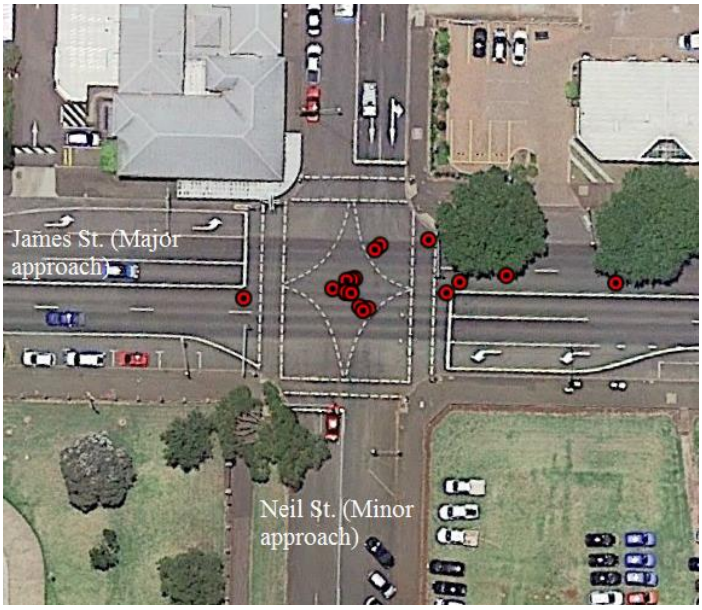

Figure 4.13 Intersection I_NE4 between James Street and Neil Street
Source: Aerial Image from Google Earth Pro

8) Intersection of Anzac Avenue and Alderley Street (I_SW19)

Figure 4.14 shows a 4-legged signalised intersection, where the major approaches have four through lanes entering, and four through lanes exiting. In addition, the minor approaches have a total of four through lanes entering and two through lanes exiting. Moreover, the median island is only present on major approaches and only one slip lane is present on one major approach as shown in the figure. The traffic volumes on the major and minor approaches were 14,500 and 8,600 vehicles per day, respectively.

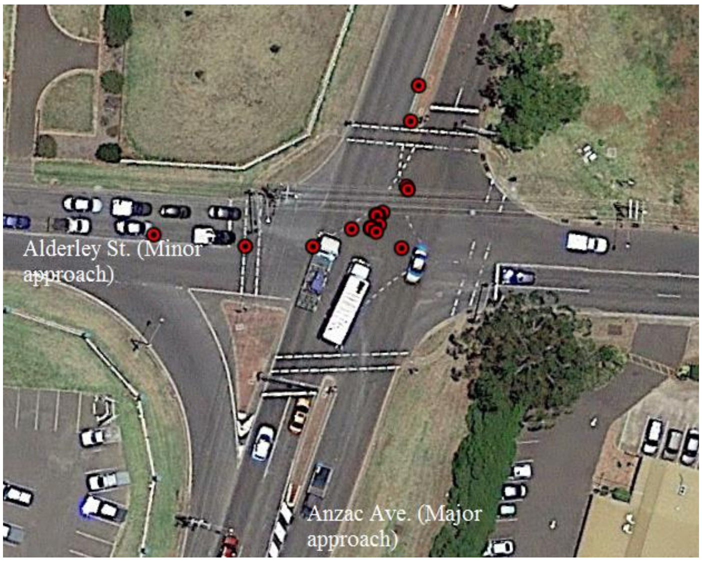

Figure 4.14 Intersection I_SW19 between Anzac Avenue and Alderley Street
Source: Aerial Image from Google Earth Pro

9) Intersection of West Street and Bridge Street (I_NW5)

Figure 4.15 shows a 4-legged signalised intersection, where the major road has a total of four through lanes entering, four through lanes exiting, and two right turn lanes. The minor road has a total of four through lanes entering, four through lanes exiting and two right turn lanes. Moreover, the raised median island is present on both major and minor approaches, and one slip lane exists on both major and minor approaches. The traffic volumes on the major and minor approaches were 13,300 and 14,800 vehicles per day, respectively.

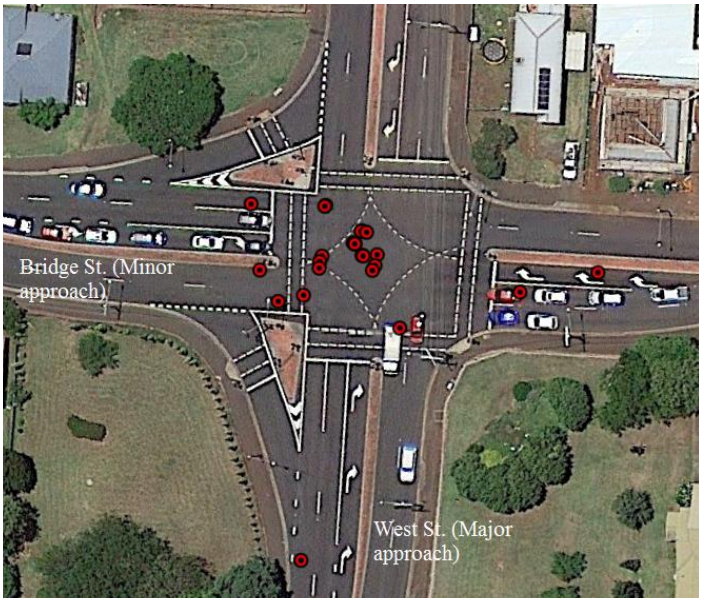

Figure 4.15 Intersection I_NW5 between West Street and Bridge Street
Source: Aerial Image from Google Earth Pro

10) Intersection of Cohoe Street and James Street (I_NE28)

Figure 4.16 shows a 3-legged un-signalised intersection, where the major road has four through lanes entering, four through lanes exiting, and one right turn lane. The minor road has one through lane exiting, one right turn lane, and one left turn lane. Moreover, the raised median island is present on both major and minor approaches. The traffic volumes on the major and minor approaches were 14,900 and 5,000 vehicles per day, respectively.

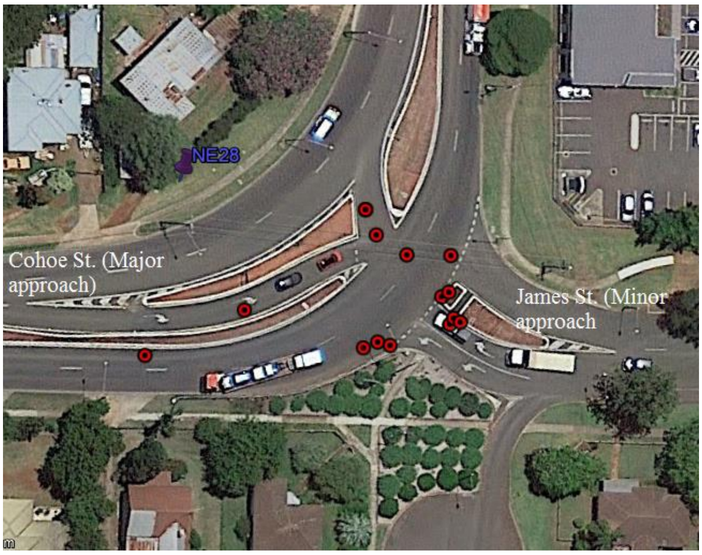

Figure 4.16 Intersection I_NE28 between Cohoe Street and James Street
Source: Aerial Image from Google Earth Pro

4.6.2 Intersection Treatment Identification

After considering the key characteristics of the top ten poorly performing intersections, possible treatments for each intersection were determined. Table 4.21 shows the proposed treatments for each intersection, where CMFs were estimated for a single proposed treatment. The highlighted row identify the most effective single treatment. The next step undertaken was to analyse the CMFs for combined treatments using the four techniques described earlier. The CMFs for treatments were ranked starting with the most effective single treatment and later they were combined to estimate the combined CMFs, as shown in Table 4.22. In other words, to identify the effect of each single treatment on road safety, the combined CMFs were estimated gradually starting with the most effective treatments.

The study revealed three treatments for intersection I_NW9. The estimated road crash reduction after applying the suggested treatments was 42%. Five treatments were suggested for the intersections I_NE5 and I_NW15 with crash reductions of 61% and 60%, respectively. Four treatments were suggested for intersection I_SE12 with a crash reduction of 60% after applying these treatments together. Three treatments were suggested for the intersection I_NE6 with a crash reduction of 62%. Five treatments were suggested for the intersection I_NW6 with a crash reduction of 61%. Seven treatments were suggested for the intersections I_NE4 and I_SW19 with crash reductions of 66% and 49%, respectively. Four treatments were suggested for intersection I_NW5 with a crash reduction of 34%. Finally, two treatments were suggested for intersection I_NE28 with a crash reduction of 34%.

The most effective single treatment for the intersections I_NE5, I_SE12, I_NW15, I_NE6, I_NW6, and I_NE4 was adding a raised median island on the major road for both directions. For the intersections I_NW9, I_SW19, I_NW5, and I_NE28, the most effective single treatment was changing the post speed limit on major approaches from 60 km/hr to 50 km/hr. It was also observed that the combined CMFs results from the four existing techniques differed from each other. In order to estimate combined CMFs with more reliability, the average of the existing techniques (adjustment approaches) were adopted. The results also indicated that the effect of treatments on road safety depends not on the number of treatments that have been applied but on the quality and the suitability of these treatments relative to the intersection's operating environment. For instance, seven treatments were suggested for intersection I_SW19 with a total crash reduction of 49% whereas only three treatments were suggested for intersection I_NE6 with a total crash reduction of 62%.

Table 4.21 Estimated CMFs for single treatment at intersections

Proposed treatments	**Labelling**	**CMF**	**Std. Er.**	**Suitable for intersection**
Reduce posted speed on major approaches from 60 to 50 km/hr	$V_{60\text{-}50}$	0.68	0.015	I_NW9, I_NE5, I_SE12, I_NW15, I_NE6, I_NW6, I_NE4, I_SW19, I_NW5, I_NE28
Add a median island on minor approaches	AM_{minors}	0.72[a]	0.099	I_NW9, I_NE5, I_SE12, I_NW15, I_NW6, I_NE4, I_SW19, I_NE6
Add a median island on major approaches	AM_{majors}	0.58[a]	0.132	I_NE5, I_SE12, I_NW15, I_NE6, I_NW6, I_NE4
Add one left turn lane on one minor approach	$A1LT_{1minor}$	0.96	0.073	I_NW9, I_NW5
Add one left turn lane on minor approaches	$A1LT_{minors}$	0.92[a]	0.138	I_NE5, I_NW15, I_NW6, I_NE4, I_SW19
Reduce number of through lane entering on minor approaches (i.e., from 2 to 1)	$R1TL_{minors}$	0.88[a]	0.047	I_NE5, I_NW6, I_NE4, I_SW19
Add one slip lane to one major approach	$A1SL_{1major}$	0.97	0.050	I_SW19, I_NW5
Add one slip lane to all major approaches	$A1SL_{majors}$	0.94[a]	0.097	I_NW15, I_SW19
Introduce signalisation	Signal	0.87	0.221	I_SE12, I_NE28
Add one right turn lane on major approaches	$A1RL_{majors}$	0.96[a]	0.075	I_SW19, I_NE4
Reduce number of through lane entering on a minor approach from 2 to 1	$R1TL_{1minor}$	0.94	0.025	I_NW5
Reduce number of through lane entering on major approaches from 2 to 1	$R1TL_{1major}$	0.67[a]	0.098	I_NE4

[a] CMF value was estimated for both road approaches i.e., in two directions

Table 4.22 Estimated CMFs for multiple treatments at intersections

ID	**Suggested treatment**	**Combined CMFs**				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
I_NW9	$V_{60\text{-}50}$+ AM_{minors}	0.49	0.66	0.54	0.68	0.59
	$V_{60\text{-}50}$+ AM_{minors}+ $A1LT_{1minor}$	0.47	0.65	0.53	0.68	**0.58**
I_NE5	AM_{majors}+ $V_{60\text{-}50}$	0.39	0.59	0.42	0.58	0.50
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}	0.28	0.52	0.33	0.58	0.43
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$	0.25	0.5	0.30	0.58	0.41
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+$A1LT_{minors}$	0.23	0.49	0.28	0.58	**0.39**
I_SE12	AM_{majors}+$V_{60\text{-}50}$	0.39	0.59	0.42	0.58	0.50
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}	0.28	0.52	0.28	0.58	0.43
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}+ Signal	0.24	0.49	0.22	0.58	**0.40**
I_NW15	AM_{majors}+$V_{60\text{-}50}$	0.39	0.59	0.42	0.58	0.50
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}	0.28	0.52	0.28	0.58	0.43
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}+ $A1LT_{minors}$	0.26	0.51	0.24	0.58	0.41
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}+ $A1LT_{minors}$+$A1SL_{majors}$	0.24	0.49	0.21	0.58	**0.40**
I_NE6	AM_{majors}+$V_{60\text{-}50}$	0.39	0.59	0.42	0.58	0.50
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}	0.28	0.52	0.33	0.58	**0.38**
I_NW6	AM_{majors}+$V_{60\text{-}50}$	0.39	0.59	0.42	0.58	0.5
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}	0.28	0.52	0.33	0.58	0.43
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$	0.25	0.5	0.30	0.58	0.41
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+$A1LT_{minors}$	0.23	0.49	0.28	0.58	**0.39**

[a] Highway Safety Manual (HSM) technique
[b] Turner technique
[c] systematic reduction of subsequent CMFs technique
[d] apply only the most effective CMF technique

Table 4.22 Estimated CMFs for multiple treatments at intersections (continue)

ID	Suggested treatment	Combined CMFs				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
I_NE4	AM_{majors}+ $R1TL_{majors}$	0.39	0.59	0.42	0.58	0.5
	AM_{majors}+ $R1TL_{majors}$+ $V_{60\text{-}50}$	0.26	0.51	0.31	0.58	0.41
	AM_{majors}+ $R1TL_{majors}$+ $V_{60\text{-}50}$ +AM_{minors}	0.19	0.46	0.24	0.58	0.37
	AM_{majors}+ $R1TL_{majors}$+ $V_{60\text{-}50}$ +AM_{minors}+ $R1TL_{minors}$	0.17	0.45	0.21	0.58	0.35
	AM_{majors}+ $R1TL_{majors}$+ $V_{60\text{-}50}$ +AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$	0.15	0.43	0.20	0.58	0.34
	AM_{majors}+ $R1TL_{majors}$+ $V_{60\text{-}50}$ +AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$+ $A1RT_{majors}$	0.15	0.43	0.20	0.58	**0.34**
I_SW19	$V_{60\text{-}50}$+ AM_{minors}	0.49	0.66	0.54	0.68	0.59
	$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$	0.43	0.62	0.50	0.68	0.56
	$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$	0.4	0.6	0.48	0.68	0.54
	$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$+ $A1RT_{minors}$	0.37	0.58	0.47	0.68	0.53
	$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$+ $A1RT_{minors}$+ $A1RT_{majors}$	0.36	0.57	0.46	0.68	0.52
	$V_{60\text{-}50}$+ AM_{minors}+ $R1TL_{minors}$+ $A1LT_{minors}$+ $A1RT_{minors}$+ $A1RT_{majors}$ + $A1SL_{1major}$	0.35	0.57	0.46	0.68	**0.51**
I_NW5	$V_{60\text{-}50}$+ $R1TL_{1minor}$	0.64	0.76	0.65	0.68	0.68
	$V_{60\text{-}50}$+ $R1TL_{1minor}$+ $A1LT_{1minor}$	0.61	0.74	0.64	0.68	0.67
	$V_{60\text{-}50}$+ $R1TL_{1minor}$+ $A1LT_{1minor}$+ $A1SL_{1major}$	0.6	0.73	0.63	0.68	**0.66**
I_NE28	$V_{60\text{-}50}$+Signal	0.59	0.73	0.63	0.68	**0.66**

[a] Highway Safety Manual (HSM) technique
[b] Turner technique
[c] systematic reduction of subsequent CMFs technique
[d] apply only the most effective CMF technique

4.7 Simulation of Traffic Operations at Treated Intersections

After identifying the values of CMF and most suitable treatments for the identified hazardous intersections in the study area, traffic simulation was employed to investigate the effect of the proposed treatments on traffic operations. Using the micro-simulation software PTV VISSM version 9.0, all hazardous intersections were simulated and the measure of treatment effectiveness was estimated using three steps.

In the first step, the intersections were modelled using the existing conditions (i.e., before treatments). These conditions included the geometric characteristics, traffic operation conditions, and traffic volume at the intersections. The models were then validated in the second step using the existing intersection conditions to ensure that the model provided realistic simulations and to ensure the applicability of the software with the traffic operation in the study area. Two intersections, West Street with Bridge Street (I_NW5) and West Street with Margaret Street (I_NW6) were selected to further validate the models by using the average value of delay and Level of Service (LOS) from Toowoomba Regional Council (TRC) data. Table 4.23 represented the observed and simulated values for the measure of effectiveness (i.e., average delay and LOS).

Table 4.23 Validation results of the intersections I_NW5 and I_NW6

Intersection ID	**Observed [a]**		**Simulated**		**Error [b]**
	Delay (sec)	**LOS**	**Delay (sec)**	**LOS**	**%**
I_NW5	16.60	B	17.92	B	7.9
I_NW6	22.50	C	21.19	C	-5.8

[a] Obtained from Toowoomba Regional Council
[b] Error = [Sim. Delay-Obs. Delay)/ Obs. Delay] x100%

The table shows that the difference of average delay between observed data and simulated results for the selected intersections is within 10 %, which is considered to be acceptable (Leng et al. 2008). The modelled levels of service for the intersections I_NW5 and I_NW6 were the same as the observed values. These results confirmed that PTV VISSIM was suitable for the study area conditions. In the final step, the intersection characteristics were changed according to the suggested treatments to identify any change in the traffic operation conditions for the hazardous intersections before and after implementation of the treatments. The ten simulation runs with random seed values for each intersection were generated using the base conditions (i.e., without any changing). Likewise, ten simulation runs were generated for each treated intersection. Average delay and level of service were used to evaluate the impact of suggested treatments on traffic operations. Table 4.24 shows the traffic operation conditions for the intersections before and after treatments.

Table 4.24 Comparison of delay and LOS between before and after treatments

Intersection ID	**Before treatments**		**After treatments**	
	Delay (sec/veh)	**LOS**	**Delay (sec/veh)**	**LOS**
I_NW9	15.87	B	15.68	B
I_NE5	13.51	B	13.35	B
I_SE12	8.55	A	14.34	B
I_NW15	17.49	B	14.53	B
I_NE6	3.24	A	3.29	A
I_NW6	21. 19	C	20.31	C
I_NE4	16.78	B	15.70	B
I_SW19	21.19	C	11.66	B
I_NW5	17.92	B	18.08	B
I_NE28	10.90	B	12.01	B

As shown in this table, the traffic operations have not been significantly affected after implementation of the treatments. Two intersections (i.e., I_SE12 and I_NE28) where there was a negative impact on the delay time resulting from the installation of a signal at these un-signalised intersections. This because the delay time is associated with the time lost to a vehicle due to the geometric and traffic conditions as well as the operation of traffic signals at a signalised intersection. The presence of traffic control (i.e., traffic signals) could increase the vehicle delay at signalised intersections compared to un-signalised intersections where the traffic operation depends only on the priority of traffic movements. Figure 4.17 and Figure 4.18 show the typical simulation process using PTV VISSIM for the intersection I_NW5 (West Street and Bridge Street). The figures also display the geometric characteristics and traffic operation before and after treatment implementation.

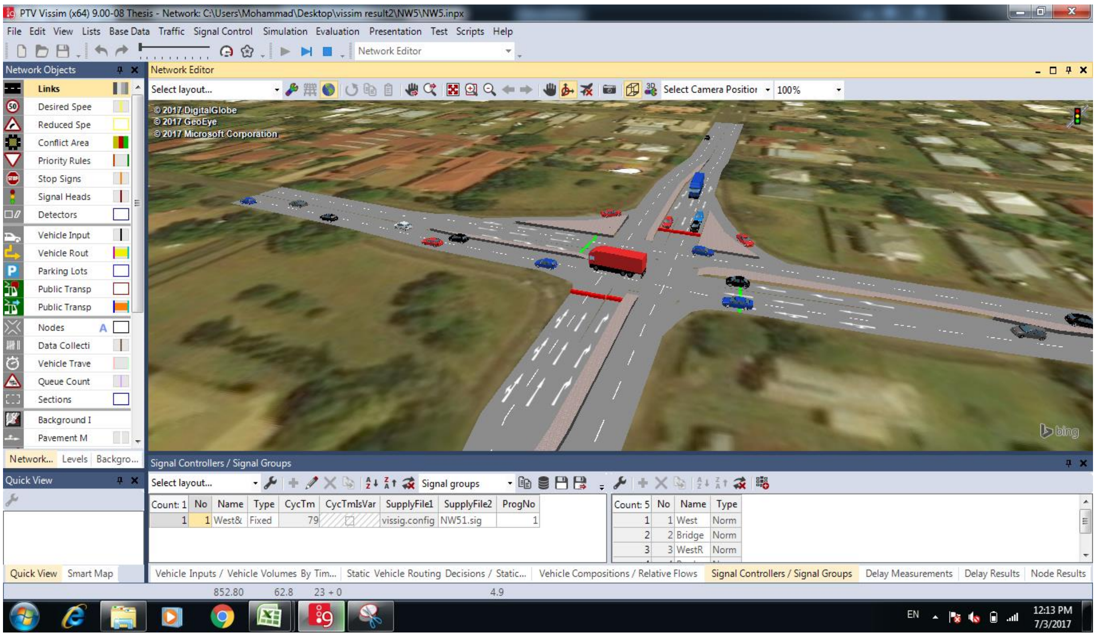

Figure 4.17 Intersection I_NW5 before treatment implementation

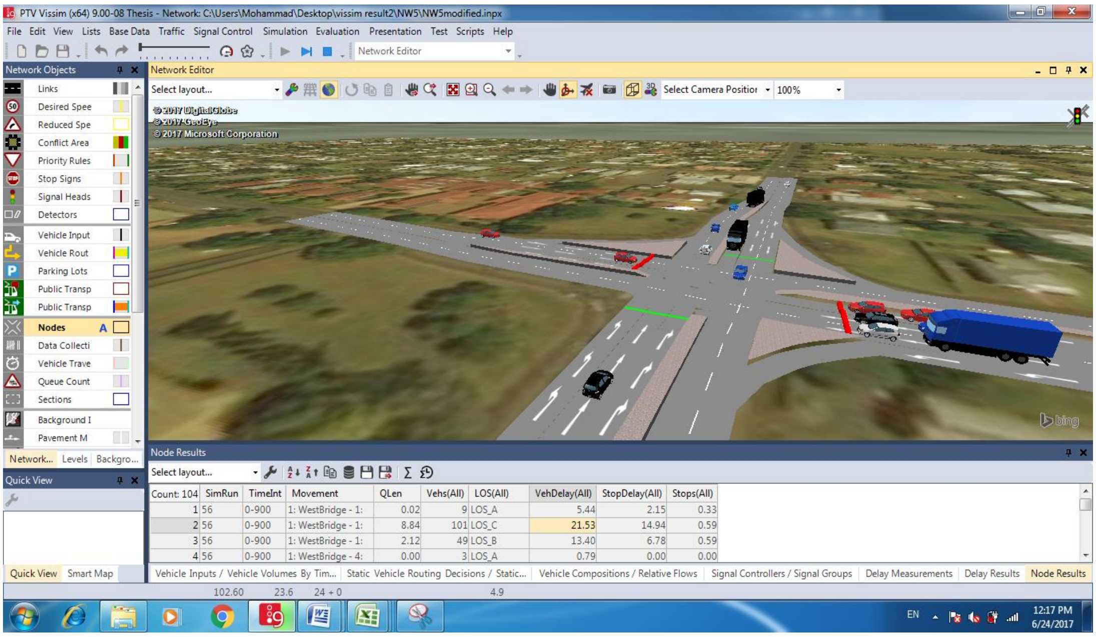

Figure 4.18 Intersection I_NW5 after treatment implementation

4.8 Benefits and Costs of Treatments

4.8.1 Benefits

In this section, the road safety treatments at intersections were evaluated based on the total discounted benefits. The results of this section provide an important step to find cost-effective treatments for road crashes at treated intersections.

The study has analysed road safety considering two type of crashes, fatal and serious injury crashes. This restriction was imposed because the type of "proposed damage only" crashes has not been reported in the study area after 31 December 2010. To estimate the crash cost reduction after treatment implementation, the percentages of both fatal and serious injury crashes were determined using the crashes that occurred in the study area during the period 2008-2015, as shown in Table 4.25. Using these percentages, the number of fatal and serious injury crashes can be estimated directly from the total crash frequencies.

Table 4.25 Number of road crashes in the study area based on the severity level

Year	**Number of crashes**		**Total**
	Fatality	**Injury**	
2008	16	679	695
2009	14	628	642
2010	8	586	594
2011	10	572	582
2012	12	540	552
2013	19	503	522
2014	11	503	514
2015	14	543	557
Grand Total	104	4554	4658
Percent (%)	2.2	97.8	100

The crash prediction models and crash reduction factors were used to estimate the number of road crashes before and after combined treatment implementation. BITRE (2009) estimated the average cost of road crashes based on the crash outcome in Queensland, Australia. The cost of road crashes per each fatality and injury in 2006 were reported as \$2,664,622 and \$266,016 (AUD), respectively. These values were used to determine the cost of road crashes before and after treatments as shown in Table 4.26. Since the crash costs have been estimated based on the year 2006, the study estimated the cost of road crashes for the year 2017 using the average value of inflation rate between 2006 and 2017 as has been discussed in Chapter 3. The reflected cost of road crashes per each fatality and injury in 2017 were reported as $3,496,215 and $349,036 (AUD), respectively.

Table 4.26 Total cost of road crashes before and after treatment implementation

Intersection ID	CMF	Ave. crash/ year [a]		Crash cost/year ($AUD)		Saved /year (2006)
		Before	After	Before	After	
I_NW9	0.58	5.50	3.20	1,753,319	1,019,847	733,472
I_NE5	0.39	4.00	1.58	1,275,141	502,618	772,523
I_SE12	0.39	2.90	1.16	924,477	370,561	553,916
I_NW15	0.40	3.30	1.32	1,051,992	421,673	630,319
I_NE6	0.38	2.50	0.94	796,963	300,190	496,773
I_NW6	0.39	2.60	1.02	828,842	326,702	502,140
I_NE4	0.34	2.90	0.99	924,477	315,093	609,384
I_SW19	0.51	2.50	1.29	796,963	409,772	387,191
I_NW5	0.66	2.60	1.72	828,842	547,726	281,116
I_NE28	0.66	1.90	1.25	605,692	397,738	207,954

[a] based on the study period 2008-2015

4.8.2 Benefit-Cost Analysis

In this study, the present value (PV) refers to the total crash cost reduction (benefits) for each intersection based on a 10-year treatment life. For PVs estimation, the values of inflation rate and discount rate were adopted at 2.5 % and 4.0 %, respectively. The value of benefit discounted rate was adopted at a lower value since the discount rate is inappropriate for evaluating human risk (Litman 2009). Table 4.27 shows the PVs of crash costs after applying the combined treatments for each intersection. It can be noted that the expected costs saved after the next 10 years of treatments range between $2.2 and $8.2 million. The highest crash cost reduction occurred at intersection I_NE5 resulting from a 61% crash reduction. The difference in the crash costs reduction depends on the type and number of suggested treatments and the conditions of the treated site. However, more details on the values of PV for combined treatments are provided in Appendix D.

Table 4.27 Present values for the intersections

Intersection ID	**Cost saved / year(2006)**	**Cost saved / year(2017)** [a]	**PV** [b] **($AUD)**
I_NW9	733,472	962,379	7,805,753
I_NE5	772,523	1,013,617	8,221,344
I_SE12	553,916	726,786	5,894,885
I_NW15	630,318	827,032	6,707,972
I_NE6	496,774	651,810	5,286,765
I_NW6	502,140	658,851	5,343,874
I_NE4	609,385	799,566	6,485,193
I_SW19	387,191	508,029	4,120,567
I_NW5	281,116	368,848	2,991,687
I_NE28	207,954	272,854	2,213,091

[a] Using the average inflation rate 2.5% between 2006-2017
[b] Discount rate (r) used equal 4%

The study estimated the total discounted benefits associated with each type of treatment to illustrate how the method can be used by practitioners to identify the expected Benefit-Cost ratio (B/C) for a treated site. The estimated benefits of crash cost reduction can be used to determine the B/C ratio through use of the direct costs associated with each treatment option. The exact direct costs associated with a proposed treatment will vary significantly with site location. As an example, four types of proposed treatments at intersection I_NW6 were evaluated in terms of crash cost reduction and the implementation cost as shown in Table 4.28. The values of B/C ratio in the table provide a clear indication that the cost-effectiveness over the full treatment life (i.e., 10 years) is economically feasible. In addition, to reduce the cost of treatment implementation, some of the treatments can be applied simultaneously. Ultimately, decisions should be based on the economic feasibility of each proposed treatment, which means that the best treatment should be the one that produces the highest return for every dollar invested.

Table 4.28 Example of the economic feasibility assessment at intersection I_NW6

Description	**CMF**	**Cost saved / year (2017)**	**PV [c]**	**Treatment Cost [b]**	**B/C**
Add median island on major approaches	0.58 [a]	456,755	3,704,694	100,000	37.05
Add median island on minor approaches	0.72 [a]	304,503	2,469,796	50,000	49.40
Reduce number of through lanes entering on minor approaches from 2 to 1	0.88 [a]	130,501	1,058,484	10,000	105.85
Add one left turn lane on minor approaches	0.92 [a]	87,001	705,656		70.56

[a] Estimated for both road approaches, see table 4.21
[b] Source: Toowoomba Regional Council
[b] Based on 10-year treatment life

4.9 Overview of Intersection-Related Treatments

In order to show the effect of each treatment on road safety, treatments were gradually added starting with the most effective treatment in the treated site. A set of finalized treatment plans for the top 10 hazardous intersections with the expected crash reduction and cost savings are summarised below.

- The study revealed three possible treatments for intersection I_NW9 between Bridge Street and Tot Street (see Figure 4.7). They were: reducing the posted speed on major approaches from 60 to 50 km/hr; adding a median island on minor approaches; and adding one left-turn lane on one minor approach. The estimated road crash reduction after applying the proposed treatments was 42%. However, the estimated crash reduction after applying only the first and second treatments was 41%, meaning that the third treatment did not significantly affect the safety. Thus, the applying of the third treatment can be restricted by available budget. In addition, the presence of a clear zone on both sides of the miner approaches gives the ability to add a median island and left turn lane on minor approaches. The expected crash cost reduction associated with all proposed treatments was approximately $AUD 7.8 million. The expected level of service (LOS) at this intersection before and after the suggested treatments was B.
- Five treatments were proposed for intersection I_NE5 between James Street and Hume Street (see Figure 4.8). They were: adding a median island on major approaches; reducing the posted speed on the major approaches from 60 to 50

km/hr; adding a median island on minor approaches; reducing one entering through lane on minor approaches; and adding one left turn lane on minor approaches. The fourth and fifth treatments can be applied by modifying the pavement arrows from straight-through to left-turn movement. The estimated road crash reduction after applying the proposed treatments was 61%. The expected crash cost reduction associated with all proposed treatments was approximately $AUD 8.2 million. The expected LOS at this intersection before and after the suggested treatments was B.

- Four treatments were proposed for intersection I_SE12 between Ruthven Street and South Street (see Figure 4.9). They were: adding a median island on major approaches; reducing posted speed on the major approaches from 60 to 50 km/hr; adding a median island on minor approaches; and introducing signalisation. The presence of a clear zone on both sides of the major and minor approaches enable the addition of a median island. The estimated road crash reduction after applying the proposed treatments was 60%. The expected crash cost reduction associated with all proposed treatments was approximately $AUD 5.9 million. Moreover, the LOS would be expected to change from A to B after applying the proposed treatments, especially introducing signalisation where the estimated delay at this intersection was increased by approximately 6.0 second/vehicle.

- Five treatments were proposed for intersection I_NW15 between Boundary Street and Hursley Road (see Figure 4.10). They were: adding a median island on the major approaches; reducing the posted speed on the major approaches from 60 to 50 km/hr; adding a median island on the minor approaches; adding one left turn lane on one minor approach; and adding one slip lane to one major approach. The estimated road crash reduction after applying the suggested treatments was 60%. However, it is worth mentioning that the fifth treatment did not significantly affect the safety, as the estimated crash reduction was 59% before applying this treatment. The expected crash cost reduction associated with the proposed treatments was approximately $AUD 6.7 million. The expected LOS at this intersection before and after the treatments was B.

- Three treatments were proposed for intersection I_NE6 James Street and Geddes Street (see Figure 4.11). They were: adding a median island on the major approaches; reducing the posted speed on the major approaches from 60 to 50 km/hr; and adding a median island on minor approaches. The estimated road crash reduction after applying the suggested treatments was 62%. The expected crash cost reduction associated with the proposed treatments was approximately $AUD 5.3 million. The expected LOS at this intersection before and after the treatments was A.

- Five treatments were proposed for intersection I_NW6 between West Street and Margaret Street (see Figure 4.12). They were: adding a median island on major approaches; reducing the posted speed on the major approaches from 60 to 50 km/hr; adding a median island on minor approaches; reducing one entering through lane on minor approaches; and adding one left turn lane on minor approaches. The presence of a clear zone on both sides of the major and minor approaches gives the ability to add a median island. Likewise, the fourth and fifth treatments can be applied by modifying the pavement arrows from straight-through to left-turn movement. The estimated road crash reduction after applying the suggested treatments was 61%. The expected crash cost reduction associated with the proposed treatments was approximately $AUD 5.3 million. The expected LOS at this intersection before and after the treatments was C.

- Seven treatments were proposed for intersection I_NE4 between James Street and Neil Street (see Figure 4.13). They were: adding a median island on major approaches; reducing one entering through lane on major approaches; reducing the posted speed on the major approaches from 60 to 50 km/hr; adding a median island on minor approaches; reducing one entering through lane on minor approaches; adding one left turn lane on minor approaches; and adding one right turn lane on major approaches. The estimated road crash reduction after applying the suggested treatments was 66%. It is worth mentioning that the estimated crash reduction after applying the sixth and seventh treatments was not significantly affected while these treatments are associated with reducing the implementation costs for fifth and second treatments, respectively. The expected crash cost reduction associated with the proposed treatments was approximately $AUD 6.5 million. The expected LOS at this intersection before and after the treatments was B.

- Seven treatments were proposed for intersection I_SW19 between Anzac Avenue and Alderley Street (see Figure 4.14). They were: reducing the posted speed on the major approaches from 60 to 50 km/hr; adding a median island on minor approaches; reducing one entering through lane on minor approaches; adding one left turn lane on minor approaches; adding one right turn lane on minor approaches; adding one right turn lane on major approaches; and adding one slip lane to one major approach. The third and fourth treatments can be applied by modifying the pavement arrows from straight-through to left-turn movement. However, the presence of a clear zone on both sides of the major and minor approaches enable the application of the suggested treatments. The estimated road crash reduction after applying the suggested treatments was 49%. Moreover, the expected crash cost reduction associated with the proposed treatments was approximately $AUD 4.1 million. The LOS at this intersection is expected to improve from C to B after applying the proposed treatments.

- Four treatments were proposed for intersection I_NW5 between West Street and Bridge Street (see Figure 4.15). They were: reducing the posted speed on the major approaches from 60 to 50 km/hr; reducing one entering through lane on one minor approach; adding one left turn lane on one minor approach; and adding one slip lane to one major approach. The third treatment did not significantly affect the safety, meaning it can be restricted by available budget. The estimated road crash reduction after applying the suggested treatments was 34%. In addition, the expected crash cost reduction associated with the proposed treatments was approximately $AUD 3.0 million. The expected LOS at this intersection before and after the proposed treatments was B.

- Two treatments were proposed for intersection I_NE28 between Cohoe Street and James Street (see Figure 4.16). They were: reducing the posted speed on the major approaches from 60 to 50 km/hr; and introducing signalisation. The total delay at this intersection would be expected to increase by 2.0 second/vehicle implemented the treatments. The estimated road crash reduction after applying the proposed treatments was 34%. The expected crash cost reduction associated with the treatments was approximately $AUD 2.2 million. The expected LOS at this intersection before and after the proposed treatments was B.

It can be observed that the most of the later treatments did not contribute significantly to crash reduction at treated sites. Indeed, these treatments were included in the treatment plans for cost-effectiveness by reducing the implementation costs for other significant treatments. The expected traffic conditions after applying the suggested treatments at the intersections were not significantly affected but in most cases were slightly improved.

4.10 Sample of Calculation

This section is provided as an example of the calculations that underpin this Chapter. The example uses intersection I_NW6 and *Model I*.

1- The goodness fit of *Model I* was identified using the following equations:

- Akaike's Information Criterion (AIC)

  $AIC = -2\log L + 2P$

  Where:

  logL –is the maximum log-likelihood of the *Model I*, (-117.083 from Appendix B)

  P –is the number of parameters in the *Model I* excluding the constant (8 variables)

  $AIC = -2\,(-117.083) + 2(8) = 250.116$ (Slightly less than the program's result = 254.166 see Table 4.6)

- Bayesian Info. Criterion (BIC)

  $BIC = -2\log L + \ln(n)\,S$

  Where:

  n –the number of data points (sample size = 106 intersections)

  S –is the number of parameters in the *Model I* including the constant (= 9 variables)

  $BIC = -2\,(-117.083) + \ln(106) \times 9 = 276.137$ (Slightly less than the program's result = 280.801 see Table 4.6)

2- Predicted number of crashes ($N_{pre,i}$) using *Model I*:

$$N_{pre,I\_NW6} = Q_{major}^{.283} \times Q_{minor}^{.281} \times e^{(-9.251 + .622\,Lg_i + .056\,LT_1 - .034\,RT_1 + .316\,SL_2 - .329\,MI_2 + .038\,V_i)}$$

$$N_{pre,I\_NW6} = 15{,}787^{.283} \times 7{,}606^{.281} \times e^{(-9.251 + .622 \times 4 + .056 \times 0 - .034 \times 2 + .316 \times 0 - .329 \times 0 + .038 \times 60)}$$

$$N_{pre,I\_NW6} = \mathbf{2.01\ crash/year}$$

3- Expected number of crashes ($N_{exp,i}$) and potential for safety improvement (PSI) value:

$$N_{exp,i} = \omega_i \times N_{pre,i} + (1 - \omega) \times N_{obs,i}$$

and, $\omega_i = \frac{1}{1+K\times\sum_{n=1}^{N} N_{pre,i}}$

Where:

$N_{exp,i}$ –is the expected crash frequency at intersection i,

ω_i –is the weighting adjustment to model prediction,

$N_{pre,i}$ –is the predicted crash frequency in a period time n,

$N_{obs,i}$ –is the observed crash frequency, and

K –is the over-dispersion parameter from the predicted model.

$$\omega_i = \frac{1}{1 + K \times \sum_{n=1}^{N} N_{pre,,i}} = \frac{1}{1 + .210 \times 2.01 \text{ x6}} = 0.283$$

$N_{exp,I\_NW6} = 0.283 \times 2.01 + (1 - .283) \times \frac{21}{6} = \mathbf{3.08}\ \boldsymbol{crash/year}$ (Table 4.8)

PSI $_{I\_NW6} = N_{exp,I\_NW6} - N_{pre,I\_NW6} = 3.08 - 2.01 = \mathbf{1.071}\ \boldsymbol{crash/year}$ (Table 4.8)

4- Crash modification factor after changing speed limits from 60 km/hr to 50 km/hr:

$CMF = e^{0.038 \times [50-60]} = \mathbf{0.68}$ (Figure 4.6; Table 4.21)

$CRF = (1.0 - CMF) \times 100 = (1.0 - 0.68) \times 100 = \mathbf{32}$ % (Crash Reduction Factor)

Standard error (Std. Er.) for the predictor variable Speed Limit was equal 0.0226 (see Appendix B)

$$CMF_{Std.\ Er.} = \frac{(e^{0.038\times[50-60]+0.0226} - e^{0.038\times[50-60]-0.0226})}{2} = \mathbf{0.015}$$

5- Benefit analysis:

$$\text{Present value (PV)} = \sum_{n=1}^{n=N} \frac{C}{(1+r)^n}$$

C –Net annual benefit

r –Discount rate (4% -10%)

N –Number of years of benefit (depend on the treatment life)

- The cost of crashes before and after treatments was calculated as follows:

$$\text{Crash cost/year}_{\,before} = 0.022 \times 2{,}664{,}622 \times 2.60 + 0.978 \times 266{,}016 \times 2.60 = \$\text{AUD } 828{,}842$$

$$\text{Crash cost/year}_{\,after} = 0.022 \times 2{,}664{,}622 \times 1.02 + 0.978 \times 266{,}016 \times 1.02 = \$\text{AUD } 326{,}702$$

- The expected cost saved after treatments implementation:

$$\text{Cost Saved} = \text{Crash cost/year}_{\,before} - \text{Crash cost/year}_{\,after}$$

$$\text{Cost Saved}_{2006} = 828{,}842 - 326{,}702 = \$\text{AUD } 502{,}140$$

$$\text{Cost Saved}_{2017} = \text{Cost Saved}_{2006} \times (\, i + 1)^{n}$$

$$\text{Cost Saved}_{2017} = 502{,}140 \times (\, 0.025 + 1)^{11} = \$\text{AUD } 658{,}851$$

- Finally, the present value after 10-year treatments life using 4% discount rate and 10-year treatments life:

$$PV_{(benefit)} = \sum_{n=1}^{10} \frac{\text{Cost Saved}_{2017}}{(1+r)^{n}} = \sum_{n=1}^{10} \frac{658{,}851}{(1+0.04)^{n}} = \$\text{AUD } 5{,}343{,}874 \text{ (Table 4.27)}$$

4.11 Summary

The research reported here recommends the most appropriate road safety measures that may be applied for hazardous (non-roundabout) intersections in Toowoomba City. Excellent potential for application to other regional cities with similar demographics and road networks exist. Crash Prediction Models (CPMs) have been developed for crash data collected from 106 intersections in the case study, namely the regional Queensland city of Toowoomba in Australia. The research found that four models capable of incorporating a range of intersection geometric features and operational conditions were worthy of further investigation. These models were employed to estimate crash modification factors for changing geometric and operational conditions.

The Empirical-Bayes method was used to finalize the safety outcome from the observed data where the outcome was portrayed as a value representing the potential for safety improvement (PSI) at each intersection. The PSI value was also used to identify the most hazardous intersections in Toowoomba for further investigation. Thereafter, the four techniques for estimating combined crash modification factors were utilised to propose effective road safety measures for the hazardous intersections. The highest crash reduction factor (i.e., CRF = 42%) for a single treatment was obtained by adding a median island on both major approaches. Likewise, the highest crash reduction (i.e., CRF = 66%) for combined treatments was obtained at intersection I_NE4. The combined treatments for this intersection included adding a median island on both major and minor approaches, adding one right turn lane on both major and minor approaches, reducing speed limit on major approaches from 60 to 50 km/hr, adding one left turn lane on minor approaches, and adding one right turn lane on major approaches.

The traffic simulation software PTV VISSIM was used to assess the performance measures at intersections after applying the suggested treatments for safety improvements. Two types of performance measures, average delay (sec/veh) and level of service (LOS) were used to identify the impact of treatments on the traffic operations. The results showed that there is no significant degradation of traffic operations at treated intersections.

Finally, the crash cost reductions that are associated with particular treatment types were estimated using the present value (PV) based on 10-year treatment life. It would be expected that the highest expected benefit values of $8.2 million would be obtained at intersection I_NE5 after applying five treatments together. Overall, the methodology identified has the potential to help decision makers to select the most appropriate treatments for safety improvements based on the crash costs reduction and the costs of suggested treatments.

Chapter 5
Roundabout Safety Analysis

5.1 Introduction

Roundabouts are usually associated with a positive impact on traffic safety compared to other types of at-grade intersections. Thus, road authorities frequently consider roundabouts as the preferred choice over other types of traffic control such as stop signs and traffic signals (Polders et al. 2015). In particular, roundabouts have a relatively low number of potential conflict points and their geometry motivates motorists to reduce their vehicle speed to a level where it helps to reduce delays and the number of decision points for road users (Daniels et al. 2011). In regional areas where the traffic volume through an at-grade intersection is moderate, the use of roundabouts has increased as an effective way of controlling traffic.

In Australia, roundabouts have been used widely in both urban and rural areas. As the number of roundabouts increases in regional areas, it is important to ensure that both existing and new roundabouts are safer for road users. In particular, there is a need to consider the traffic and geometric characteristics of roundabouts that can significantly affect both crash frequency and severity. Minor traffic and geometric modifications can lead to major changes in safety and/or operational performance at roundabouts (Kamla et al. 2016). This chapter provides details of the analysis of traffic and geometric characteristics of roundabouts and their influences on road safety in Toowoomba city using Negative Binomial (NB) and/or Poisson statistical models. The hazardous roundabouts were identified using an Empirical Bayes (EB) approach and combined Crash Modification Factors (CMFs) have been developed to suggest appropriate treatments. The suggested treatments were evaluated using the simulation software VISSIM and benefit-cost analysis. The study results apply to similar regional roundabouts with similar geometric and traffic conditions.

5.2 Data Preparation

The current study analysis is conducted using the crash data from 49 roundabouts in Toowoomba city, Australia. For all roundabouts, crash data were collected from the

Department of Transport and Main Roads, Queensland in Excel spreadsheet format for all Queensland's roads. In total, 126 crash reports containing severe crashes (fatal and serious injury) occurred in the period 2010 – 2015 (six years). The crash reports include detailed information on each crash, such as crash time, crash location, crash type, severity level, speed limit, number of vehicles and persons involved. Severe crashes that occurred at the roundabout area and within 20 metres measured towards upstream from the give way line were included in the dataset, as shown in Figure 5.1. These roundabouts were not selected at random but based on the geographic location in the study area to prevent bias towards high or low crash frequency locations as described in Chapter 4. The study area was divided into four quadrants (i.e., NE, NW, SE, and SW) to provide a uniform distribution for data selection as shown earlier in Figure 4.1. The roundabout was defined using the quadrant symbol with numbered, an example is given below.

Roundabout ID	Road Name
R_NE4	Bridge Street and Mackenzie Street
R_NW4	North Street and Holberton Street
R_SE4	Hume Street and Spring Street
R_SW4	Greenwattle Street and South Street

(See appendix A for all roundabouts)

The datasets were divided into two groups. The first group was used to develop the crash prediction models based on three years of data (2010-2012). The second group was used for validation of the models against three additional years of data (2013-2015) for the same roundabouts used in the development of the models. This validation was used to evaluate the capability of models to predict crashes across time. Twenty-one explanatory variables describing traffic and road geometry were used in modelling as the most common factors associated with road crashes at the roundabouts. The following is a detailed description of these variables:

1. Number of legs: This variable shows the number of roundabout legs, i.e. 3, 4, or 5 legs.
2. Number of lanes entering: This variable shows the total number of lanes entering for major approaches and in the same way for minor approaches.
3. Number of lanes exiting: This variable shows the total number of lanes exiting for major approaches and in the same way for minor approaches.

4. Entry lane width: This is the distance measured perpendicularly from the left edge of the entry to the crossing point of the right edge line and the inscribed circle.
5. Exit lane width: This is the distance measured perpendicularly from the left edge of the exit to the crossing point of the right edge line and the inscribed circle.
6. Average Entry path radius: This is defined as the minimum radius on the fastest through path before the yield line (Austroads 2015), measured 1.5 metres from the canter line or the curb face and 1.0 from the edge line, noted as R1 in the Figure 5.2.
7. Average Exit path radius: This is defined as the minimum radius on the fastest through path into the exit (Austroads 2015), measured 1.5 metres from the canter line or the curb face and 1.0 from the edge line, noted as R2 in the Figure 5.2.
8. Presence of fixed object: is any fixed objects (e.g., trees, rocks, etc.) within the central island.
9. Road AADT: This variable shows the Annual Average Daily Traffic (AADT) on major approach and on minor approach.
10. Circulatory roadway width: This is the width between the edge of the central island and the outer edge of the circulatory roadway, excluding the width of any apron.
11. Length and width of weaving section: The weaving section is the area inside the roundabout where combined movement of both merging and diverging movements occur in the same direction. The width and length of this section are represented in Figure 5.2.
12. Central island diameter: This is the diameter of the raised area in the centre of a roundabout around which vehicles rotating.
13. Speed Limit (km/hr): This variable shows the speed limit in kilometres per hour on the major approach.

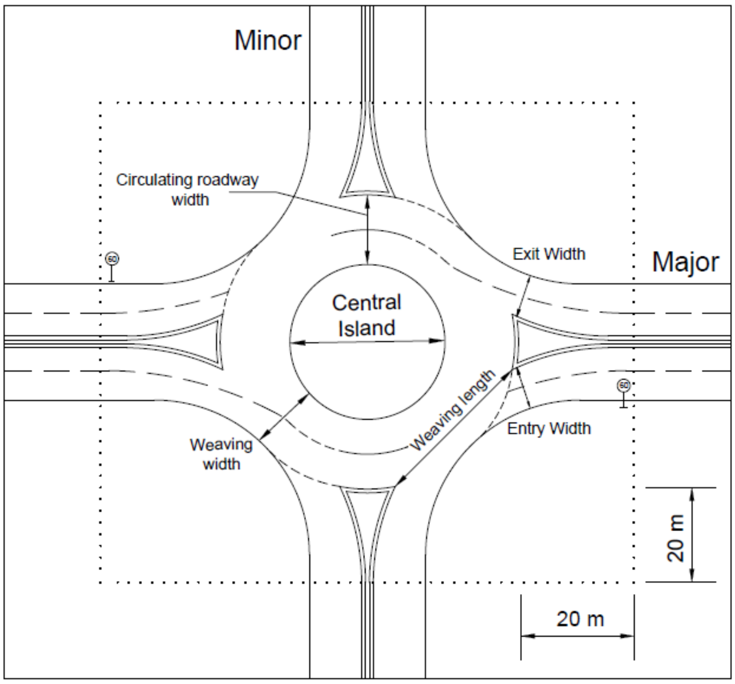

Figure 5.1 A typical roundabout representing explanatory variables

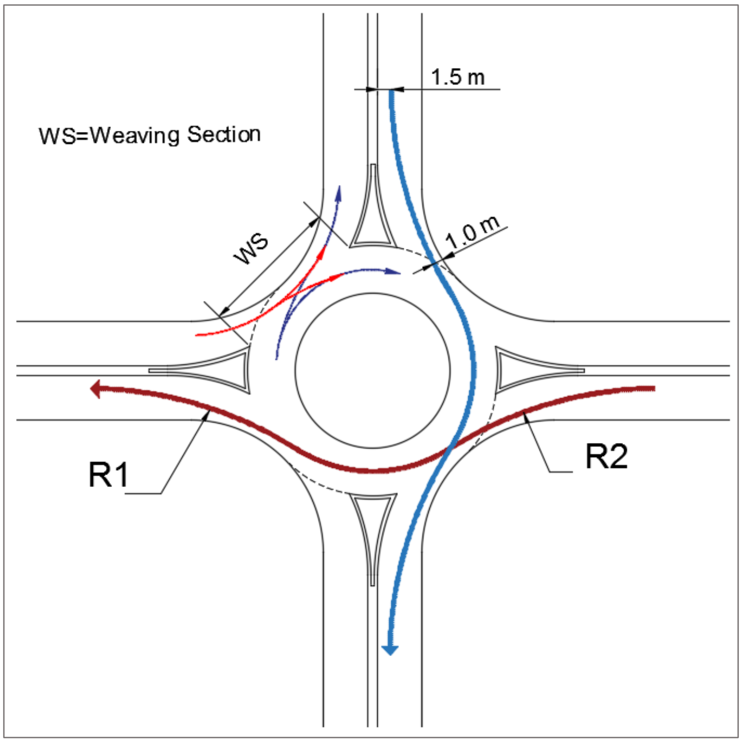

Figure 5.2 Entry and exit path radius

5.3 Developing Crash Prediction Models for Roundabouts

The CPMs at roundabouts were developed using a generalised linear modelling (GLM) approach. Two types of GLM were considered for use in this study: negative binomial (NB) and Poisson distribution. In order ascertain which of these two types was suitable for estimating safety outcomes, the study adopted the over-dispersion assumption. This assumption has been tested based on the value of the deviance divided by the degree of freedom (*df*) as well as the value of the Pearson Chi-square (x^2) divided by the degree of freedom (*df*). As discussed early, if the result of these tests lies between 0.8 and 1.2, the NB model assumption will be accepted. However, if it is out of this range the Poisson model will be used instead of the NB model (Abdul Manan et al. 2013).

5.3.1 Identifying Possible Models using Correlation Matrix

Analysis of the data collected for roundabouts provided some correlation among the explanatory variables. Table 5.1 illustrates correlations values within the data based on the correlation matrix for the dataset. The correlations among the explanatory variables were tested to prevent the use of strongly correlated variables together within a model, i.e. strong correlation variables would strongly affect the other parameters in the same model. The variable parameters were considered to be statistically significant at 0.1 significance level (using 90% confidence). Based on the correlation matrix and 0.1 significance level, five road safety models were identified as shown in Table 5.2.

Table 5.1 Pearson correlation matrix for roundabout

Variable		Lg_r	LN_{r1}	LN_{r2}	LE_{r1}	LE_{r2}	En_1	En_2	Ex_1	Ex_2	Rn_1	Rn_2	Rx_1	Rx_2	Q_{major}	Q_{minor}	F	CW	WL	WW	CD	V_r
Lg_r	Pearson Correlation [a]	1																				
	Sig. (2-tailed)																					
LN_{r1}	Pearson Correlation [a]	.506	1																			
	Sig. (2-tailed)	.000																				
LN_{r2}	Pearson Correlation [a]	.878	.750	1																		
	Sig. (2-tailed)	.000	.000																			
LE_{r1}	Pearson Correlation [a]	.597	.814	.909	1																	
	Sig. (2-tailed)	.000	.000	.000																		
LE_{r2}	Pearson Correlation [a]	1.000	.506	.878	.597	1																
	Sig. (2-tailed)	.000	.000	.000	.000																	
En_1	Pearson Correlation [a]	.431	.822	.616	.654	.431	1															
	Sig. (2-tailed)	.002	.000	.000	.000	.002																
En_2	Pearson Correlation [a]	.292	.577	.422	.619	.292	.798	1														
	Sig. (2-tailed)	.042	.000	.000	.000	.042	.000															
Ex_1	Pearson Correlation [a]	.234	.475	.453	.454	.234	.615	.796	1													
	Sig. (2-tailed)	.106	.001	.001	.000	.106	.000	.000														
Ex_2	Pearson Correlation [a]	.116	.399	.229	.282	.116	.573	.633	.744	1												
	Sig. (2-tailed)	.426	.004	.113	.049	.426	.000	.000	.000													
Rn_1	Pearson Correlation [a]	-.106	-.105	-.115	-.100	-.106	-.219	-.292	-.410	-.405	1											
	Sig. (2-tailed)	.470	.473	.432	.495	.470	.130	.042	.003	.004												
Rn_2	Pearson Correlation [a]	.063	.102	.103	.118	.063	.088	-.065	-.268	-.256	.575	1										
	Sig. (2-tailed)	.669	.486	.480	.418	.669	.548	.657	.063	.075	.000											
Rx_1	Pearson Correlation [a]	-.262	-.287	-.285	-.248	-.262	-.391	-.377	-.457	-.376	.285	.262	1									
	Sig. (2-tailed)	.069	.045	.047	.085	.069	.006	.008	.001	.008	.047	.068										
Rx_2	Pearson Correlation [a]	.009	-.280	-.128	-.223	.009	-.440	-.315	-.206	-.319	-.049	-.200	.354	1								
	Sig. (2-tailed)	.952	.052	.380	.124	.952	.002	.028	.155	.026	.738	.169	.012									
Q_{major}	Pearson Correlation [a]	.186	.306	.241	.241	.186	.278	.208	.167	-.078	.084	.220	-.187	-.373	1							
	Sig. (2-tailed)	.201	.032	.095	.095	.201	.053	.152	.250	.595	.566	.129	.198	.008								
Q_{minor}	Pearson Correlation [a]	.072	.185	.080	.071	.072	.260	.178	.030	-.057	.182	.286	-.164	-.356	.263	1						
	Sig. (2-tailed)	.622	.204	.585	.629	.622	.071	.222	.840	.696	.210	.046	.259	.012	.000							
F	Pearson Correlation [a]	.092	.096	.126	.130	.092	.011	-.133	-.044	-.075	.050	.226	-.209	-.276	-.051	-.044	1					
	Sig. (2-tailed)	.531	.512	.390	.372	.531	.942	.361	.766	.608	.732	.119	.149	.055	.728	.762						
CW	Pearson Correlation [a]	.520	.427	.533	.438	.520	.326	.222	.088	-.046	-.057	-.022	-.291	-.109	.478	.319	-.124	1				
	Sig. (2-tailed)	.000	.002	.000	.002	.000	.022	.124	.546	.753	.697	.878	.043	.454	.001	.026	.397					
WL	Pearson Correlation [a]	-.079	-.142	-.080	-.065	-.079	.163	.482	.599	.565	-.336	-.409	-.272	-.062	.114	.052	-.407	-.001	1			
	Sig. (2-tailed)	.592	.331	.586	.656	.592	.262	.000	.000	.000	.018	.003	.059	.672	.434	.725	.004	.993				
WW	Pearson Correlation [a]	.357	.624	.495	.318	.357	.525	.429	.323	.253	-.068	-.053	-.402	-.259	.364	.253	-.087	.738	.022	1		
	Sig. (2-tailed)	.012	.000	.000	.000	.012	.000	.002	.024	.079	.643	.718	.004	.072	.010	.080	.551	.000	.882			
CD	Pearson Correlation [a]	.066	.036	.069	.058	.066	.327	.365	.386	.651	-.366	-.322	-.325	-.172	.102	.048	-.257	-.181	.175	.006	1	
	Sig. (2-tailed)	.651	.805	.635	.690	.651	.022	.000	.000	.000	.010	.024	.023	.239	.486	.742	.075	.213	.000	.967		
V_r	Pearson Correlation [a]	.167	.068	.111	.041	.167	.285	.267	.227	.197	-.219	-.101	-.237	-.018	.034	-.016	-.065	-.078	.207	.121	.345	1
	Sig. (2-tailed)	.251	.645	.446	.782	.251	.047	.064	.117	.176	.130	.489	.101	.903	.815	.916	.655	.595	.153	.407	.015	

[a].Listwise N=49

Table 5.2 Variables included in the selected roundabout models

Variable	SPSS labelling	Model I	Model II	Model III	Model IV	Model V
Number of legs	Lg_r		✓			
Number of entry lanes on major approach	LN_{r1}				✓	
Number of entry lanes on minor approach	LN_{r2}			✓		
Number of exit lanes on major approach	LE_{r1}	✓				
Number of exit lanes on minor approach	LE_{r2}					✓
Entry width lanes on major approach	En_1					✓
Entry width lanes on minor approach	En_2			✓		
Exit width lanes on major approach	Ex_1	✓				
Exit width lanes on minor approach	Ex_2				✓	
Entry radius on major approach	Rn_1				✓	
Entry radius on minor approach	Rn_2		✓			
Exit radius on major approach	Rx_1				✓	
Exit radius on minor approach	Rx_2			✓		
AADT on major approach	Q_{major}	✓	✓	✓	✓	✓
AADT on minor approach	Q_{minor}	✓		✓		✓
Fixed object on central island	F		✓			
Circulatory roadway width	CW				✓	
Weaving length	WL		✓			
Weaving width	WW	✓				
Central island diameter	CD	✓	✓	✓		
Speed Limit (km/hr)$_{Major}$	V_r	✓	✓			

Table 5.3 shows a statistical summary of the dependent variable (i.e., number of road crashes per 3 years) and independent variables that were used for the purpose of constructing the models. The descriptive statistics of the explanatory variables of the roundabouts used in this analysis are also presented in the table. The explanatory variables are divided into count data, continuous data, and categorical data (representing the presence or absence of geometric features). The roadways are defined as a major approach or as a minor approach based on the roundabout geometric and traffic volume features. Appendix A provides full details of selected roundabouts in this study (49 roundabouts). This dataset was used to estimate the model parameters as presented in the next sub-section 5.3.2.

Table 5.3 Statistical summary of the roundabout dataset

Variable Description	**N**	**Min.**	**Max.**	**Mean**	**Standard Deviation**	**SPSS labelling**	**Variable Type**
No. of Legs	49	3	5	3.98	0.249	Lg_r	Count
No. of lanes Entering							
Major-approach	49	2	4	2.08	0.344	LN_{r1}	Count
Minor-approach	49	1	5	2.02	0.478	LN_{r2}	Count
No. of lanes Exiting							
Major-approach	49	2	4	2.04	0.286	LE_{r1}	Count
Minor-approach	49	1	3	1.98	0.249	LE_{r2}	Count
Entry width (m)							
Major-approach	49	2.9	8.6	3.99	1.026	En_1	Continuous
Minor-approach	49	2.9	6.8	3.84	0.698	En_2	Continuous
Exit width (m)							
Major-approach	49	3.2	8.0	4.44	0.910	Ex_1	Continuous
Minor-approach	49	3.1	7.2	4.36	0.691	Ex_2	Continuous
Entry Radius							
Major-approach	49	31.0	101.0	64.24	13.849	Rn_1	Continuous
Minor-approach	49	28.0	105.0	64.45	15.379	Rn_2	Continuous
Exit Radius							
Major-approach	49	34.0	98.0	58.63	14.464	Rx_1	Continuous
Minor-approach	49	30.0	119.0	60.14	14.790	Rx_2	Continuous
AADT(ln AADT)							
Major-approach	49	1288 (7.161)	16071 (9.685)	6966 (8.701)	3430.7 (0.594)	Q_{major}	Continuous
Minor-approach	49	1200 (7.090)	10002 (9.211)	4341 (8.215)	2322.4 (0.601)	Q_{minor}	Continuous
Fixed object on central island	49	0	1	0.55	0.503	F	Categorical
Circulatory roadway width (m)	49	4.8	9.3	6.82	0.824	CW	Continuous
Weaving length (m)	49	9.0	36.0	15.57	3.969	WL	Continuous
Weaving width (m)	49	5.8	10.7	7.34	0.947	WW	Continuous
Central island diameter (m)	49	5.8	90.0	15.09	11.737	CD	Continuous
Speed Limit (km/hr)$_{Major}$	49	40	70	58.78	4.393	V_r	Continuous

[a] AADT = Annual Average Daily Traffic

5.3.2 Modelling and Measuring Goodness-of-Fit

The data analysis and model development was undertaken using SPSS software version 23. Different models were developed and fitness of results were assessed based on the confidence levels and the correlation values between the variables. After several trials of a different combination of variables, five models were developed using Negative Binomial (NB) error structure with log link function. The estimated regression parameters for the selected crash models for the roundabouts are presented in Table 5.4. The parameters listed in Table 5.4 can be substituted into Equations to estimate the road crashes at roundabouts as presented in Table 5.5.

In Model I, all the predictor variables are significant except for Annual Average Daily Traffic (AADT) on the major approach at 90% level of confidence. Likewise, all the predictor variables in Model II are significant except for the number of legs and speed limit. In Model III, all predictor variables are significant except for the number of lanes entering on the minor approach. In Model IV, all the predictor variables are significant except for the roundabout circulatory roadway width. In Model V, all the predictor variables are significant except for the number of lanes exiting on the minor approach. It is worth mentioning that some explanatory variables (e.g., AADT on major approach, Model I) showed significant correlation with the other variables and have p-value higher than 0.1.

Tests on the selected models were performed to verify if there was an over-dispersion. The Deviance and Pearson Chi-square (x^2) statistics divided by their degrees of freedom (*df*) were estimated as shown in Table 5.6. It can be observed that the values of these two tests are within the allowable range of 0.80 and 1.20, indicating that the NB distribution assumption is acceptable. As mentioned earlier, when the dispersion coefficient (K) is positive and greater than zero (i.e., K > 0.0, suggesting over-dispersion), the NB model is appropriate.

A comparison of the selected prediction models was then performed using Akaike Information Criterion (AIC) and Bayesian Information Criterion (BIC). The smaller of the AIC and BIC values was considered better than the other models with higher values (Cafiso et al. 2010; Abdul Manan et al. 2013; Young & Park 2013). Based on the Goodness-Of-Fit (GOF) test results in Table 5.6, the predicted models were ranked (best to worst) with the order as follows: Model V, Model III, Model II, Model IV, and Model I.

Table 5.4 Negative binomial parameter estimates for selected roundabout models

Parameter	Model I		Model II		Model III		Model IV		Model V	
	β	***P*-Value [b]**	**β**	***P*-Value [b]**	**β**	***P*-Value [b]**	**β**	***P*-Value [b]**	**β**	***P*-Value [b]**
Intercept	-15.930	.000	-15.471	.000	-10.618	.000	-10.616	.003	-12.606	.000
No. of Legs (Lg_r)	-	-	.467	.121	-	-	-	-	-	-
No. of lanes Entering										
Major-approach (LN_{r1})	-	-	-	-	-	-	.564	.000	-	-
Minor-approach (LN_{r2})	-	-	-	-	.022	.233	-	-	-	-
No. of lanes Exiting										
Major-approach (LE_{r1})	.338	.008	-	-	-	-	-	-	-	-
Minor-approach (LE_{r2})	-	-	-	-	-	-	-	-	.079	.267
Entry width										
Major-approach (En_1)	-	-	-	-	-	-	-	-	.307	.000
Minor-approach (En_2)	-	-	-	-	.367	.004	-	-	-	-
Exit width										
Major-approach (Ex_1)	-.068	.000	-	-	-	-	-	-	-	-
Minor-approach (Ex_2)	-	-	-	-	-	-	-.005	.108	-	-
Entry Radius										
Major-approach (Rn_1)	-	-	-	-			0.032	.000		
Minor-approach (Rn_2)	-	-	.035	.000			-	-		
Exit Radius										
Major-approach (Rx_1)	-	-	-	-	-	-	-.020	.000	-	-
Minor-approach (Rx_2)	-	-	-	-	-.024	.000	-	-	-	-
AADT										
Major-approach (Q_{major})	.241	.117	1.163	.000	.403	.063	.954	.000	.438	.004
Minor-approach (Q_{minor})	1.121	.000	-	-	.915	.000	-	-	.923	.000
Fixed object on central island (F)[a]	-	-	-.052	.103	-	-	-	-	-	-
Circulatory roadway width (CW)	-	-	-	-	-	-	.063	.208	-	-
Weaving length (WL)	-	-	-.010	.006	-	-	-	-	-	-
Weaving width (WW)	.305	.033	-	-	-	-	-	-	-	-
Central island diameter (CD)	-.005	.001	.012	.037	-.020	.000	-	-	-	-
Speed Limit (km/hr)$_{Major}$ (V_r)	.038	.057	.023	.138	-	-	-	-	-	-
Dispersion (K)	.208 [a]		.110 [a]		.200 [a]		.220 [a]		.203 [a]	

[a] Computed based on the Pearson Chi-square
[b] Significance at 0.1 level
[c] Fixed object =1 if present; = 0 if not present

Table 5.5 Summary of the selected models to estimate roundabout crashes

Model No.	Model Form
I	$N_{pre.i} = Q_{major}^{.241} \cdot Q_{minor}^{1.121} \cdot\ e^{(-15.930 + .338\, LE_{r1} - .068\, Ex_1 + .305\, WW - .005\, CD + .038\, V_r)}$
II	$N_{pre.i} = Q_{major}^{1.163}\ \cdot\ e^{(-15.471 + .467\, Lg_r + .035\, Rn_2 - .052\, F - .010\, WL + .012\, CD + .023 V_r)}$
III	$N_{pre.i} = Q_{major}^{.403} \cdot Q_{minor}^{.915} \cdot\ e^{(-10.618 + .022\, LN_{r2} + .367\, En_2 - .024\, Rx_2 - .020\, CD)}$
IV	$N_{pre.i} = Q_{major}^{.954} \cdot\ e^{(-10.616 + .564\, LN_{r1} - .005\, Ex_2 + .032\, Rn_1 + -.020\, Rx_1 + .063\, CW)}$
V	$N_{pre.i} = Q_{major}^{.438} \cdot Q_{minor}^{.923} \cdot\ e^{(-12.606 + .079\, LE_{r2} + .307\, En_1)}$

$N_{pre,i}$ = predicted crashes number at i[th] roundabout in 3 years

Table 5.6 Goodness-of-fit tests for roundabouts models

Model	Parameter	Value	*df*	Value/*df*
I	Deviance	37.557	41	0.916
	Pearson Chi-Square (x^2)	35.266		0.860
	Akaike's Info. Criterion (AIC)	156.265		.
	Bayesian Info. Criterion (BIC)	171.400		.
II	Deviance	40.348	41	0.984
	Pearson Chi-Square (x^2)	37.179		0.907
	Akaike's Info. Criterion (AIC)	153.512		.
	Bayesian Info. Criterion (BIC)	168.647		.
III	Deviance	35.937	42	0.856
	Pearson Chi-Square (x^2)	36.584		0.871
	Akaike's Info. Criterion (AIC)	152.227		.
	Bayesian Info. Criterion (BIC)	165.470		.
IV	Deviance	48.262	42	1.177
	Pearson Chi-Square (x^2)	44.118		1.076
	Akaike's Info. Criterion (AIC)	154.373		.
	Bayesian Info. Criterion (BIC)	169.508		.
V	Deviance	46.719	43	1.086
	Pearson Chi-Square (x^2)	46.490		1.081
	Akaike's Info. Criterion (AIC)	147.967		.
	Bayesian Info. Criterion (BIC)	159.318		.

The GOF for the selected models was also investigated using the cumulative residuals (CURE) plot. As outlined in Chapter 3, this method must achieve two conditions to indicate that the model fitted the data well: (i) the curve lies within two standard deviations (+2σ and -2σ boundaries) of the mean and (ii) the curve oscillates around zero. Figure 5.4 shows the CURE plot, as a function of AADT, for all selected models. As noted in this figure, the CURE curve for all models is within the standard deviation boundaries, which means that all models are fitting the data well.

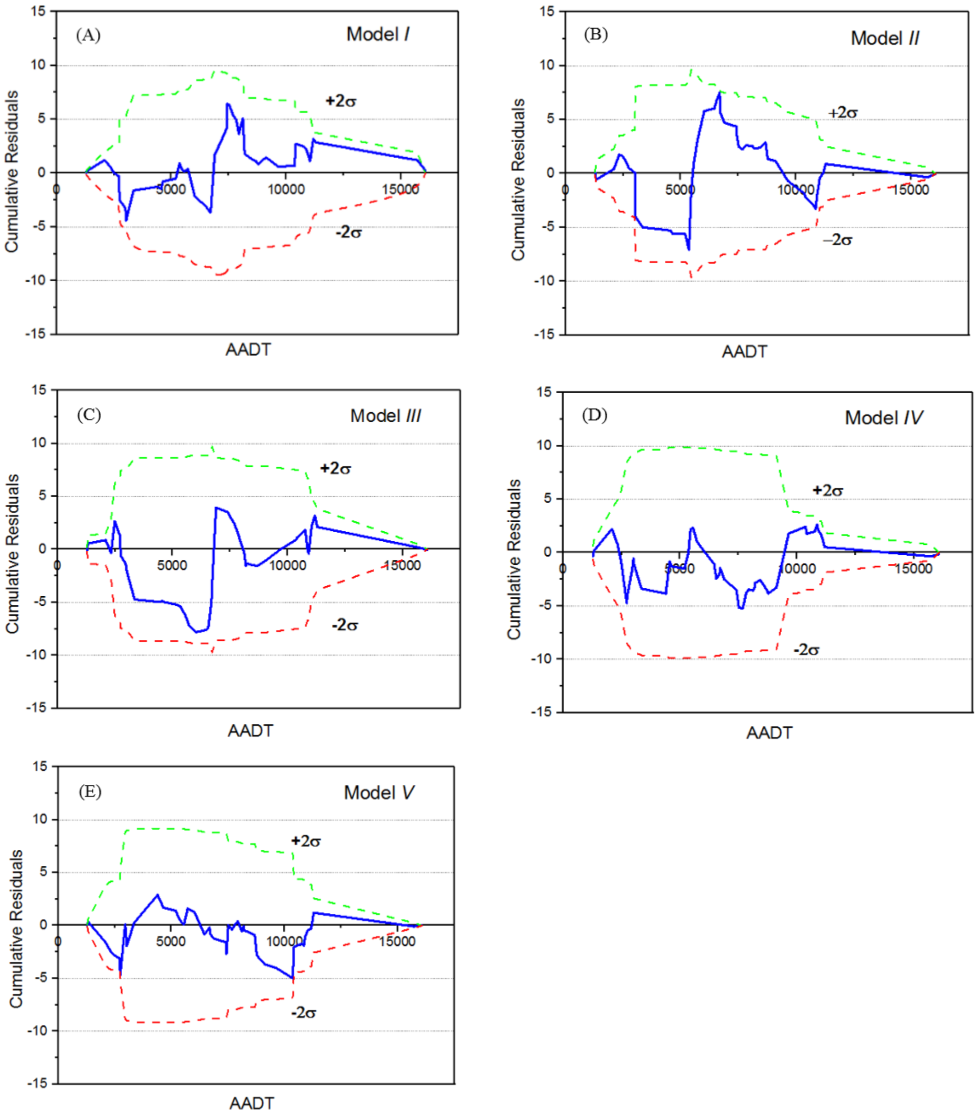

Figure 5.3 Cumulative residual (CURE) plots for roundabout models. (A) Model *I*. (B) Model *II*. (C) Model *III*. (D) Model *IV*. (E) Model *V*

The quality of fit was also investigated using the residual values from a fitted model to identify the appropriate model that fitted the data well (the residual being the difference between the observed and predicted number of crashes). Figure 5.4 shows the plot of the residuals at each roundabout against one of the key explanatory variables (Log-AADT on the major approach). This plot was obtained by ranking the residual values in an increasing order for the Log-AADT variable. The indication that the predicted model has well-fitted data points is when the residual values oscillate around the zero line and the residual values are not widely spread. From Figure 5.4 it is seen that Model V is more appropriate than the other models because it has the smallest spread, whereas the residual values for Model V range from -2.11 to 4.85. Furthermore, the spread of the average residuals for the Model V was 0.92, while for

Model I, Model II, Model III and Model IV the values were 0.94, 0.91, 0.94, and 1.01, respectively.

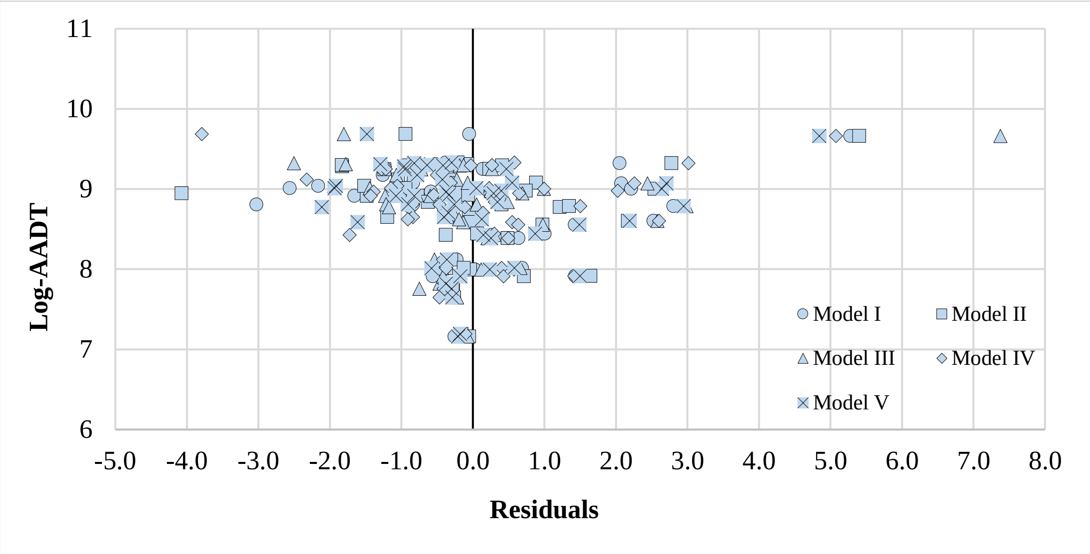

Figure 5.4 Plot of the Residuals with Log-AADT on the major approach at roundabouts

5.3.3 Model Validation

This section presents validation results for the five roundabouts safety models. The validation tests were used to assess the ability of models to predict road crashes over subsequent additional years. Several performance measures were used to validate the models including the mean squared prediction error (MSPE), mean absolute deviation (MAD), mean squared error (MSE), and Freeman-Tukey R-Squared coefficient (R^2_{FT}). These performance measures were defined previously in Chapter 3. In general, a smaller value (closer to zero) of MSPE, MAD, or MSE refers to lower prediction error. Likewise, the higher values of R^2_{FT} indicate a better prediction performance. Table 5.7 shows the results of the validation tests for the estimation dataset (2010-2012) and the validation dataset (2013-2015). The models were developed using the estimation dataset. The values of MSPE using validation dataset and MSE using estimation dataset are similar for all developed models, which represents a high level of transferability of the models. The same result was obtained for MAD where the estimation dataset and the validation dataset were similar for all developed models, whereas the R^2_{FT} test results were slightly lower for the validation dataset than that for the estimation dataset.

Table 5.7 Performance measures for all crash prediction models for roundabout

Performance measures	Model I		Model II		Model III		Model IV		Model V	
	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]
MSPE	-	1.639	-	2.240	-	2.021	-	2.229	-	1.710
MSE	1.942	-	2.253	-	2.596	-	2.280	-	1.873	-
MAD	0.944	0.966	0.915	1.070	0.940	0.963	1.013	0.974	0.923	0.929
R^2_{FT} %	56.1	43.7	57.1	31.4	58.7	46.7	55.6	39.3	59.8	47.6

[a] Calculated based on estimation dataset 2010-2012
[b] Calculated based on validation dataset 2013-2015

Overall, the GOF measures used in this study show that all models fit the data very well and can be adequately accepted for further analysis. Model V was the best-fitted model and was used to calculate the expected road crash frequency as discussed in the following section.

5.4 High-Risk Roundabouts

This section describes the procedure for identifying and ranking high-risk roundabouts using an Empirical Bayes (EB) approach. Firstly, Model V was used to estimate road crashes for each roundabout in the study area. Secondly, the weighting adjustment factor (ω) was calculated using the over-dispersion parameter (K) and the predicted number of crashes for the study period (2010-2012). Thirdly, the expected number of crashes was estimated by combining the predicted number of crashes using Model V with the observed number of crashes for the same period. Finally, the potential for safety improvements (PSI) was calculated for ranking of the roundabouts.

5.4.1 Identifying and Ranking High-Risk Roundabouts

Model V was applied to estimate the number of crashes at roundabouts. The estimation of the expected number of crashes for each roundabout was obtained by combining the observed crashes number with the predicted crashes number using the EB approach. This combination was applied based on the weighting adjustment factor which was estimated from the over-dispersion parameter related to Model V, i.e. K = 0.250, in this research. This approach helped to ensure unbiased estimates of the long-term expected number of crashes for each roundabout as described earlier in Chapter 3.

The EB approach identified the black spot sites based on their PSI value. Table 5.8 shows the identified and ranking of the black spot sites using the EB approach for a total of 49 roundabouts in the study area. As noted in the table, the first 19 roundabouts had positive values of PSI, which indicated that a potential for safety improvement existed. The remaining 30 roundabouts had zero and negative values of PSI, which indicated no, or limited, potential for safety improvement.

In general, the most dangerous roundabout identified in the analysis for safety improvement was R_NW7 (between Anzac Avenue, Hursley Road, and Holberton Street) with PSI = 2.870. The roundabout with least potential for improvement was R_NW8 (between West Street, Russell Street, and Anzac Avenue) with PSI = -1.007. Appendix A provides full details of all roundabouts.

Table 5.8 Ranking roundabouts for safety improvement

Roundabout ID	**observed (cr./3year)**	**Predicted (cr./3year)**	**Weighted adjustment(w)**	**Expected (cr./3year)**	**PSI [a]**	**Rank**
R_NW7	12	7.15	0.41	10.02	2.870	1
R_SE11	5	2.29	0.68	3.15	0.860	2
R_SW3	5	2.35	0.68	3.21	0.856	3
R_SW2	4	1.81	0.73	2.40	0.588	4
R_NE1	4	1.05	0.82	1.57	0.519	5
R_NE4	2	1.13	0.81	1.29	0.162	6
R_NE7	2	0.51	0.91	0.65	0.140	7
R_SE2	2	0.50	0.91	0.64	0.139	8
R_SE6	3	2.60	0.65	2.74	0.137	9
R_SE13	2	1.45	0.77	1.57	0.125	10
R_SE17	2	1.53	0.76	1.64	0.111	11
R_NW1	2	1.71	0.74	1.78	0.075	12
R_NE5	1	0.42	0.92	0.46	0.046	13
R_NE2	1	0.66	0.88	0.70	0.040	14
R_SE5	1	0.75	0.87	0.78	0.033	15
R_SE14	1	0.77	0.87	0.80	0.032	16
R_NE6	1	0.85	0.85	0.87	0.022	17
R_SE16	1	0.88	0.85	0.89	0.019	18
R_NE9	1	0.95	0.84	0.96	0.008	19
R_NE3	0	0.18	0.97	0.17	-0.006	20
R_SE1	0	0.20	0.96	0.20	-0.008	21
R_SW6	0	0.29	0.94	0.27	-0.016	22
R_NW5	0	0.30	0.94	0.28	-0.017	23
R_SE9	0	0.36	0.93	0.34	-0.025	24
R_SE23	0	0.38	0.93	0.35	-0.027	25
R_SE8	0	0.40	0.92	0.37	-0.030	26
R_SE19	1	1.18	0.81	1.14	-0.034	27
R_SW5	0	0.43	0.92	0.39	-0.034	28
R_SW8	1	1.26	0.80	1.21	-0.053	29
R_SW1	0	0.58	0.90	0.52	-0.060	30
R_NW4	1	1.34	0.79	1.26	-0.072	31
R_SE3	1	1.34	0.79	1.27	-0.072	32
R_SE18	1	1.37	0.78	1.29	-0.081	33
R_SE20	2	2.29	0.68	2.20	-0.093	34
R_SE22	0	0.82	0.86	0.70	-0.117	35
R_SE15	0	0.95	0.84	0.80	-0.153	36
R_SE24	0	0.96	0.84	0.80	-0.157	37
R_SE12	3	3.42	0.59	3.25	-0.171	38
R_SW4	1	1.78	0.73	1.57	-0.206	39

Table 5.8 Ranking roundabouts for safety improvement (continue)

Roundabout ID	**observed (cr./3year)**	**Predicted (cr./3year)**	**Weighted adjustment(w)**	**Expected (cr./3year)**	**PSI**	**Rank**
R_NW3	1	1.90	0.72	1.65	-0.252	40
R_SE7	3	3.64	0.57	3.37	-0.272	41
R_SE10	1	2.07	0.70	1.76	-0.319	42
R_NE8	1	2.29	0.68	1.88	-0.411	43
R_SW7	5	5.82	0.46	5.38	-0.445	44
R_SE21	0	1.92	0.72	1.38	-0.536	45
R_NW2	0	1.93	0.72	1.39	-0.544	46
R_NW6	1	2.61	0.65	2.05	-0.557	47
R_SE4	2	4.11	0.55	3.15	-0.960	48
R_NW8	9	10.48	0.32	9.47	-1.007	49

[a] PSI = (expected crashes number - predicted crashes number)

5.5 Crash Modification Factors for Roundabout Crashes

A crash modification factor (CMF) identifies the change in road safety (crash frequency) resulting from implementing a particular treatment. This treatment may be in the form of design modification, change in traffic operations, or any countermeasures. The recognition of any change in geometric design features or traffic operations will increase or decrease crash frequency. There are several methods available to estimate CMFs values. These methods vary from a before-and-after study with a comparison group to relatively more sophisticated methods such Empirical Bayes (EB) and Full Bayes (FB) methods (Mbatta 2011). Also, the cross-sectional method, proposed by Washington et al. (2005) has been usually used to estimate CMFs values because it is easier to collect data compared to other methods. As described in Chapter 3, this method is also known as a crash prediction model (CPM) or safety performance function (SPF), which relates crash number with geometric characteristics and traffic volume of a roadway. The CMF can be estimated directly from the coefficient of the variable associated with the proposed treatment. Part of the cross-sectional method to estimate the CMFs based on the coefficients of the CPMs is known as a crash modification function (CMFunction).

5.5.1 Description of Base Conditions

The CMFs were developed based on the base condition of the covariates i.e. $e^{\beta \times (X_i - X_{Base})}$. As discussed earlier, the base condition values in this study were adopted from previous studies as well as the mean values of an individual explanatory variable. For instance, the mean values for the number of roundabout legs and the number of lanes entering on the major approach were found to be 4 and 2, respectively (from Table 5.3). In the same way, for the entry or exit lane width the base values were derived directly from previous studies. By definition, the base condition can be defined as the condition associated with a CMF value of 1.0. Table 5.9 shows a list of base conditions adopted for each traffic or design element for roundabouts.

Table 5.9 Base conditions for different design elements of roundabout

Feature	**Base Values**
Number of roundabout legs	4 legs
Number of lanes Entering or exiting	2 lanes
Entry or exit lane width	4.2 metres
Entry or Exit Radius	60 metres
AADT on major approach	7,000 vehicle per day
AADT on minor approach	4,000 vehicle per day
Fixed object on central island	0 (No object)
Circulatory roadway width	7 metres
Weaving length	15 metres
Weaving width	7 metres
Central island diameter	15 metres
Speed Limit	60 km/hr

5.5.2 Crash Modification Function

The CMFunction method was used in this study to estimate the road safety effect for each independent variable that was used in the development of CPMs at roundabouts. A CMF value of 1.0 represents no effect on safety while a CMF above 1.0 indicates a treatment resulting in a higher number of crashes. In contrast, a CMF below 1.0 indicates a treatment resulting in lower crash numbers. After applying this method based on the parameters of the variables associated with the type of treatment, CMFs and standard errors (Std. Er.) for each treatment were estimated as follows.

Number of Roundabout Legs:

Model II was used to derive CMFs values associated with the number of roundabout legs. The 4-legged roundabout was adopted as a base condition to estimate CMFs. The result presented in Table 5.10 shows that the 5-legged roundabout was associated with more crashes than 3-legged and 4-legged roundabouts. When the roundabout changed from 4-legged to 3-legged the number of crashes reduced by 37%. When the number of legs increased from 4-legged to 5-legged the number of crashes increased by 60%. This result was expected because the traffic volume and vehicle interactions at roundabouts increase after adding more legs. A similar result has also been concluded in previous studies (Shadpour 2012; Kim & Choi 2013). It should be noted that the number of roundabout legs should preferably be limited to 4, as increased conflicts occur at multi-lane roundabout exits.

Table 5.10 CMFs based on the number of roundabout legs

CMFunction	Lg_i	CMF [a]	Std. Er.
$CMFunction = e^{0.467\times[Lg_i-4]}$	3	0.63	0.031
	4	1.00	0.050
(Base condition at 4-legs)	5	1.60	0.080

[a] Estimated using model II

Number of Entry Lanes

Table 5.11 shows the CMFs for the number of entry lanes on major and minor approaches were derived from Model IV and Model III, respectively. In order to estimate the CMFs for the number of entry lanes entering based on each entry approach, the relevant model parameters were divided by two for both major and minor approaches (Lord & Bonneson 2007; Li et al. 2010). The results indicate that the number of entry lanes was associated with more crashes for both major and minor approaches. For example, after adding one entry lane on a major approach or a minor approach, the probability of crashes increases by 25% and 1%, respectively. It can be noticed that the effect of the number of entry lanes at a major approach is found to be more significant than a minor approach and this is probably due to the difference in traffic volume. Turner et al. (2009) also concluded that the multiple entry lanes are associated with greater crash frequency. In general, the number of entry roundabout lanes provided on major or minor approaches should be limited to the minimum number that meets the required capacity and operating requirements for the traffic volumes.

Table 5.11 CMFs based on entry lanes

CMFunction	***LN***	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{0.282\times[LN_i-2]}$	1	0.75	0.016	0.99	0.014
$CMFunction_{minor} = e^{0.011\times[LN_i-2]}$	2	1.00	0.021	1.00	0.014
(Base condition at 2 lanes)	3	1.33	0.028	1.01	0.014

[a] Estimated using model IV
[b] Estimated using model III

Number of Exit Lanes

Similar to the number of entry lanes, the CMFs were estimated based on the exit for each road approach. Model I was used to estimate CMFs for major approaches and Model V for minor approaches. The results indicated that road crashes increased by 18% and 4% after adding one exit lane on a major approach and on a minor approach, respectively, as shown in Table 5.12. This result was expected because the number of conflict points increases at multi-lane entrances and exits when compared to single-lane conditions. The number of exit lanes should be limited to the number of circulating lanes to prevent the conflict between the merging and diverging vehicles.

Table 5.12 CMFs based on exit lanes

CMFunction	LE_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF [b]**	**Std. Er.**
$CMFunction_{major} = e^{0.169\times[LX_i-2]}$	1	0.84	0.024	0.96	0.235
$CMFunction_{minor} = e^{0.040\times[LX_i-2]}$	2	1.00	0.028	1.00	0.244
(Base condition at 2 lanes)	3	1.18	0.033	1.04	0.254

[a] Estimated using model I
[b] Estimated using model V

Entry Width

Table 5.13 shows the values of CMF for entry width for both major and minor approaches. Model V and Model III have been used to estimate the CMFs for major and minor approaches, respectively. The results show that wider entry width at major and minor approaches was associated with higher road crash numbers compared with narrow width. This result is possible because the wider entry width is associated with higher vehicle speed at the entry of the roundabout. Designers should therefore aim to make the entry lane widths no wider than necessary to be able to accommodate the path of entering design vehicles (Austroads 2015). Figure 5.5 represents the effect of entry width on road safety for both minor and major approaches. The value of CMF in this study is applicable to the entry width changing from 2.9 to 8.6 metres for major approaches and from 2.9 to 6.8 for minor approaches.

Table 5.13 CMFs based on entry width

CMFunction	***En$_i$***	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF** [b]	**Std. Er.**
$CMFunction_{major} = e^{0.307\times[En_i-4.2]}$	3.6	0.83	0.088	0.80	0.024
$CMFunction_{minor} = e^{0.367\times[En_i-4.2]}$	4.2	1.00	0.106	1.00	0.030
(Base condition at 4.2 m)	4.8	1.20	0.128	1.25	0.037

[a] Estimated using model V
[b] Estimated using model III

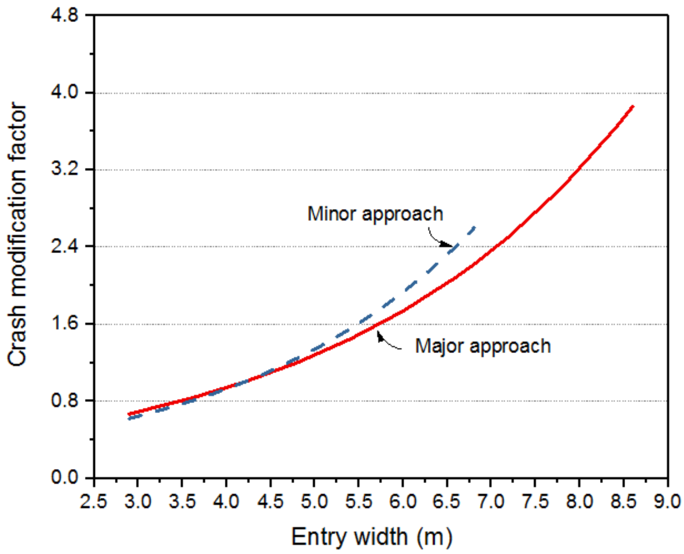

Figure 5.5 CMF for entry width

Exit Width

The study also examined the effect of exit width for major and minor approaches at the roundabouts using Model I and Model IV, respectively, as shown in Table 5.14. The results revealed that a wider exit width for both major and minor approaches increased road safety. This result is possibly because the wider exit width increases comfort for drivers to exit the roundabout safely and to ensure that the exit width accommodates the swept path of the design vehicle (Austroads 2015). In roundabout design it is usually desirable to reduce entry width and entry path radius to slow vehicles, but to allow for vehicles to accelerate on the exit. Thus, the width of the exit is usually wider than the entering width. Figure 5.6 shows the relationship between exit width and road safety, where the exit width on minor approaches appears to have less impact on road safety compared to the exit width on major approaches. The value of CMF in this study is applicable to the exit width changing from 3.2 to 8.0 metres for major approaches and from 3.1 to 7.2 for minor approaches.

Table 5.14 CMFs based on exit width

CMFunction	Ex_i	Major		Minor	
		CMF [a]	Std. Er.	CMF [b]	Std. Er.
$CMFunction_{major} = e^{-0.068\times[Ex_i-4.2]}$	3.6	1.04	0.005	1.00	0.065
$CMFunction_{minor} = e^{-0.005\times[Ex_i-4.2]}$	4.2	1.00	0.005	1.00	0.065
(Base condition at 4.2 m)	4.8	0.96	0.004	0.99	0.064

[a] Estimated using model I
[b] Estimated using model IV

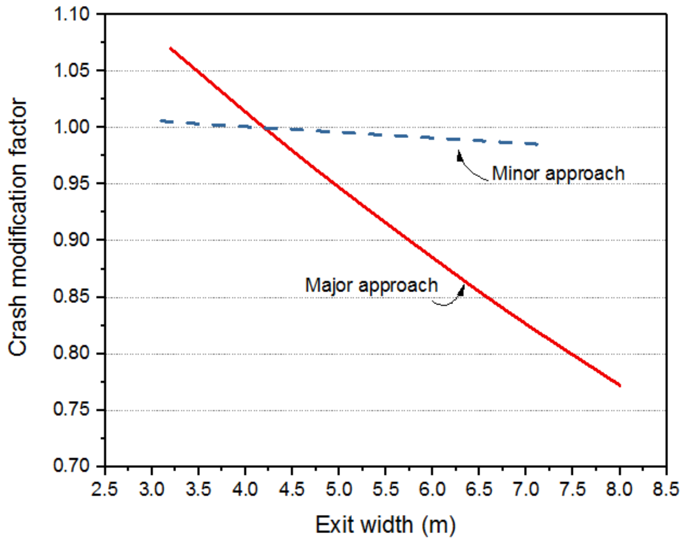

Figure 5.6 CMF for exit width

Entry Radius

The entry radius or entry path radius is one of the most important factors among geometric parameters at a roundabout, since it affects both safety and capacity (Montella et al. 2012). A large entry path radius usually results in faster entry speeds and results in additional road crashes. Table 5.15 shows the values of CMF for major and minor approaches using Model IV and Model II, respectively. The larger entry path radius for both minor and major approach are associated with more road crashes at a roundabout. Figure 5.7 illustrates the relationship between CMF values and entry path radius. It can be seen from the figure that the effect on CMF values of entry path radius for both minor and major approaches is roughly the same. The values of CMF in this study is applicable to the entry radius ranging from 31 to 101 metres for major approaches and from 28 to 105 metres for minor approaches.

Table 5.15 CMFs based on entry radius

CMFunction	Rn_i	Major		Minor	
		CMF [a]	Std. Er.	CMF [b]	Std. Er.
$CMFunction_{major} = e^{0.032\times[Rn_i-60]}$	50	0.73	0.007	0.71	0.006
$CMFunction_{minor} = e^{0.035\times[Rn_i-60]}$	60	1.00	0.010	1.00	0.009
(Base condition at 60 m)	70	1.38	0.014	1.42	0.013

[a] Estimated using model IV
[b] Estimated using model II

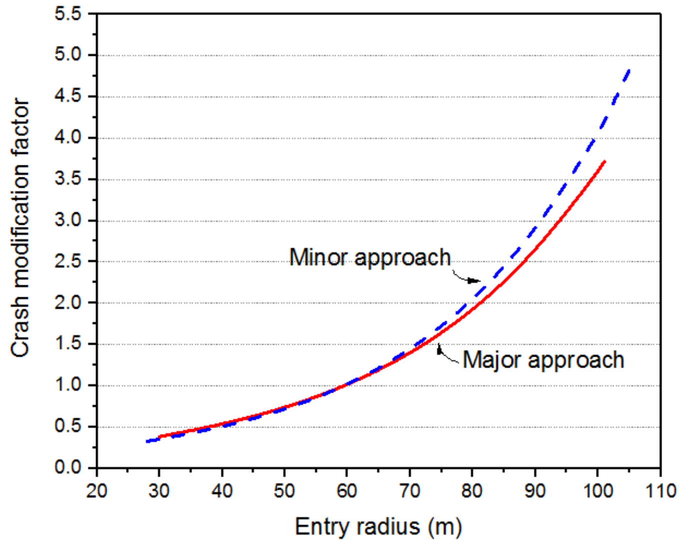

Figure 5.7 CMF for entry radius

Exit Radius

Model IV and Model III have been used to estimate the CMFs for major and minor approaches, respectively. A smaller exit radius results in increased safety risk for both major and minor approaches at roundabouts, as shown in Table 5.16. As mentioned previously, the exit from the roundabout must be as comfortable and easy for a driver as possible. Entries of roundabouts are designed to decrease vehicle speeds, whilst exits allow vehicles to increase speed out of the circulating roadway. Thus, the exit radius should generally be greater than entry radius for safety and operational issues at roundabouts. The study found that a higher exit radius is associated with less crash risk as shown in Figure 5.8. For instance, at the major approach, the percent of crash reduction after increasing the exit radius by 10 metres was 18%. This result agrees with a study undertaken by Anjana and Anjaneyulu (2014). The value of CMF in this study is applicable to the exit radius ranging from 34 to 98 metres for major approaches and from 30 to 119 metres for minor approaches.

Table 5.16 CMFs based on exit radius

CMFunction	***Rx***$_i$	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF** [b]	**Std. Er.**
$CMFunction_{major} = e^{0.020\times[Rx_i-60]}$	50	1.22	0.013	1.27	0.019
$CMFunction_{minor} = e^{0.024\times[Rx_i-60]}$	60	1.00	0.011	1.00	0.015
(Base condition at 60 m)	70	0.82	0.009	0.79	0.012

[a] Estimated using model IV
[b] Estimated using model III

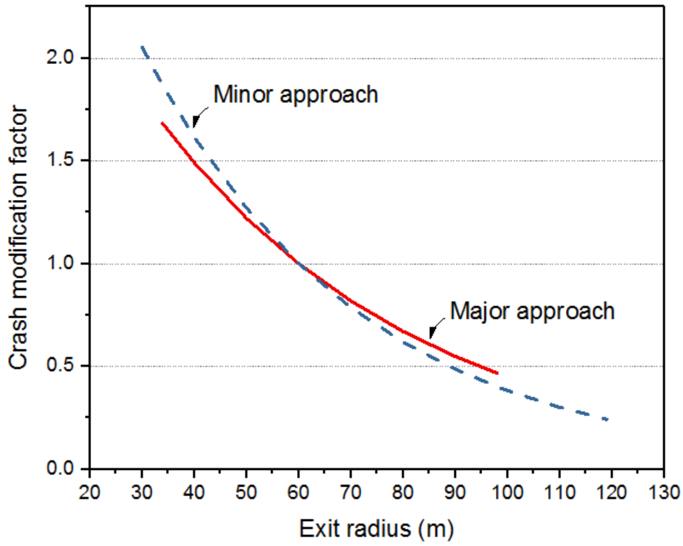

Figure 5.8 CMF for exit radius

Traffic Volume (AADT)

The Highway Safety Manual (AASHTO 2010) uses traffic volume as a significant predictor in studying road safety. In this study, Model V has been selected to estimate the CMFs for major and minor approaches, respectively based on the GOF test. The base condition for a major approach was adopted at 7,000 vehicles per day and for a minor approach at 4,000 vehicles per day. These values were adopted based on the mean values of traffic volumes in the dataset. Table 5.17 shows that the crash risk increases with increasing traffic volumes due to increased vehicle interactions. The results also show that the volume on the minor approach has a larger impact on safety than major approach at high traffic volumes. This may be due to the difference in geometric characteristics (i.e. lane width, number of lane, etc.) between minor and major approaches. Figure 5.9 illustrates the relationship between traffic volumes and road safety. The value of CMF in this study is applicable to the traffic volume ranging from 1,300 to 16,000 vehicles per day for major approaches and from 1,200 to 10,000 vehicles per day for minor approaches.

Table 5.17 CMFs based on traffic volumes

CMFunction	Q_i	**Major**		**Minor**	
		CMF [a]	**Std. Er.**	**CMF** [a]	**Std. Er.**
$CMFunction_{major} = (\frac{Q_{major}}{7{,}000})^{0.438}$	1,200	N/A	N/A	0.33	0.011
$CMFunction_{minor} = (\frac{Q_{minor}}{4{,}000})^{0.923}$	4,000	0.78	0.027	1.00	0.033
(Base condition at 7,000 veh/day	7,000	1.00	0.035	1.68	0.055
& 4,000 veh/day, respectively)	10,000	1.17	0.041	2.33	0.077

N/A, Non-Applicable based on the range of dataset
[a] Estimated using model V

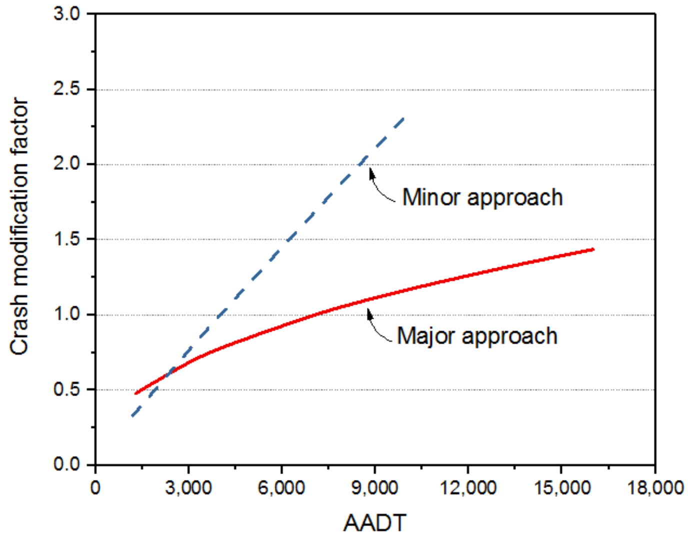

Figure 5.9 CMF for traffic volume

Fixed Objects on Central Islands

Fixed objects like trees may be placed within a central island area, provided the island is large enough to ensure that clear zone requirements are met and the sightlines for drivers are not obstructed. In most cases, these fixed objects are placed on the central island to help reduce the entry speed of the vehicles and focus the driver's attention approaching the roundabout. Table 5.18 shows the values of CMFs for roundabouts with and without fixed objects on the central island using Model II. The study found that roundabouts with fixed objects have about 5% fewer crashes than roundabouts without fixed objects.

Table 5.18 CMFs based on presence of fixed object on a central island

CMFunction	F_i	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{-0.052\times[F_i-0]}$	0.0	1.00	0.275
(Base condition at No object)	1.0	0.95	0.275

[a] Estimated using model II

Circulatory Roadway Width

The circulating roadway is the portion of roundabout between the inscribed circle and the central island used by vehicular traffic as shown in Figure 5.1. The circulating roadway width is recommended to be about 1.0 to 1.2 times the entry width to a roundabout (Montella et al. 2012). A wider circulatory roadway width should be avoided, especially at a single-lane roundabout, where drivers may then think that two vehicles are allowed to drive side by side within the roundabout. Model IV was used to derive the values of CMF as shown in Table 5.19. The result indicates that the wider circulatory roadway width is associated with greater crash risk at roundabouts. Figure 5.10 illustrates the relationship between circulatory roadway width and road safety. The value of CMF in this study is applicable to the circulatory roadway width ranging from 4.8 to 9.3 metres.

Table 5.19 CMFs based on circulatory roadway width

CMFunction	***CW***	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{0.063\times[CW_i - 7.0]}$	6.5	0.97	0.192
	7.0	1.00	0.198
(Base condition at 7.0 m)	7.5	1.03	0.205

[a] Estimated using model IV

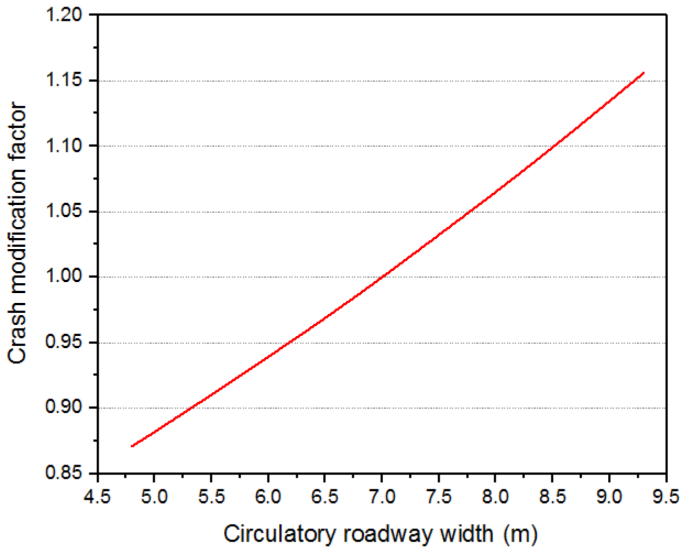

Figure 5.10 CMF for circulatory roadway width

Weaving Length

The weaving section is a dynamic portion in the roundabout, where vehicles carry out one or more lane changes to complete merging and diverging operations (see Figure 5.2). The two significant parameters in the analysis of weaving sections, based on road safety and capacity, are weaving length and weaving width (Golob et al. 2004). This study investigated the impact of weaving length on road safety using Model II to derive values of CMF as shown in Table 5.20. The result revealed that an increase in weaving length results in a decrease in crash risk. This result was reasonable because a long distance of weaving length decreases the probability of crashes as a result of sufficient space and time to complete merging or diverging operations. Figure 5.11 illustrates the relationship between weaving length and road safety. The value of CMF in this study is applicable to weaving length ranging from 9 to 36 metres.

Table 5.20 CMFs based on weaving length

CMFunction	***WL_i***	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{-0.010\times[WL_i - 15]}$	12	1.03	0.072
	15	1.00	0.070
(Base condition at 15 m)	18	0.97	0.068

[a] Estimated using model II

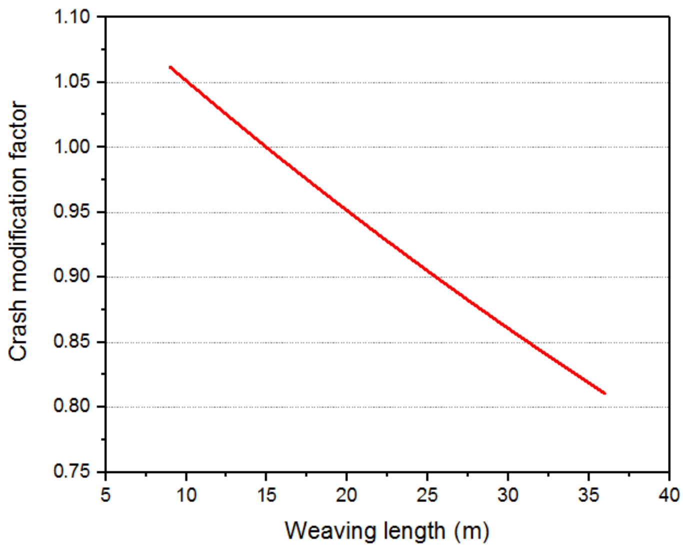

Figure 5.11 CMF for weaving length

Weaving Width

As mentioned previously, one source of vehicles conflicts at the roundabout is the weaving section, where the merge and diverge occur between vehicles. The impact of weaving width on road safety was investigated in this study using Model I. A wider weaving width results in an increase in crash risk, as shown in Table 5.21. The wider weaving width, as in the circulatory roadway width, can lead to attempts by vehicles to pass each other, resulting in high speed driving and therefore increased risk. Figure 5.12 illustrates the relationship between weaving width and road safety. The value of CMF in this study is applicable to a weaving width ranging from 5.8 to 10.7 metres.

Table 5.21 CMFs based on weaving width

CMFunction	WW_i	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{0.305\times[WW_i - 7.0]}$	6.5	0.86	0.123
	7	1.00	0.144
(Base condition at 7.0 m)	7.5	1.16	0.167

[a] Estimated using model I

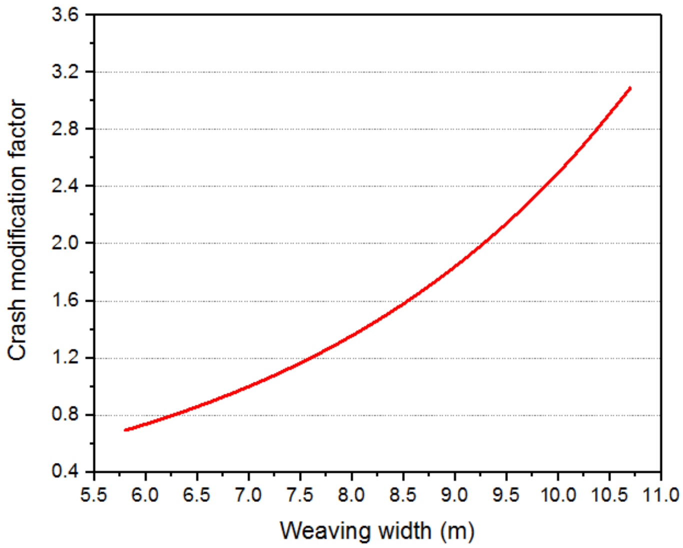

Figure 5.12 CMF for weaving width

Central Island Diameter

The geometry of a central island should be designed to reduce high entry speeds to the roundabout. The shape of central islands should preferably be circular because changes in curvature of the circulating carriageway lead to a variance in speeds and increase the complexity for drivers. Wider central island diameters are preferable, as they reduce the entry vehicle speeds because a reduction of the angle formed between the circulating and entering vehicle paths (Austroads 2015). Model III was selected to estimate the CMF values based on the Goodness of Fit test. The base condition in this study was an island diameter of 15 metres, adopted based on the mean values of the central island diameters in the dataset. Table 5.22 shows that the wider central island diameter roundabout was associated with lower crash risk. Figure 5.13illustrates the relationship between the central island diameter and road safety. A similar result has been concluded by Shadpour (2012) and Kim and Choi (2013). The value of CMF in this study is applicable to a central island diameter ranging from 5.8 to 90 metres.

Table 5.22 CMFs based on central island diameter

Central island diameter	***CD$_i$***	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{-0.02\times[CD_i - 15]}$	12	1.07	0.017
	15	1.00	0.016
(Base condition at 15 m)	18	0.94	0.015

[a] Estimated using model III

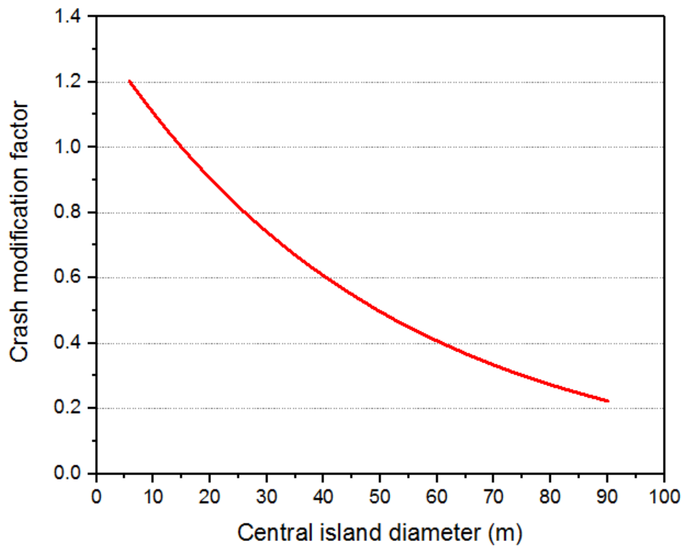

Figure 5.13 CMF for central island diameter

Speed Limit

Speed limit is one of the most important parameters that significantly affect road safety at roundabouts (Austroads 2015). Ideally, lower operating speeds at roundabouts are associated with a longer time for driver reaction and thus reduce the number and severity of road crashes that do occur. In this study, the speed limits on major approaches were analysed and the CMF values were estimated as shown in Table 5.23. Model II was selected to estimate the CMF values based on the GOF test. The results indicate that the crash risk increases as posted speed limit increases. For instance, a 10 km/hr increase in speed limit leads to a 26% increase in the expected number of crashes. Figure 5.14 illustrates the relationship between speed limit and road safety. The value of CMF in this study is applicable to the posted speed limit ranging from 40 to 70 km/hr.

Table 5.23 CMFs based on speed limit

CMFunction	V_i	CMF [a]	Std. Er.
$CMFunction_i = e^{0.023 \times [V_i - 60]}$	60	1.00	0.040
(Base condition at 6o km/hr)	70	1.26	0.050

[a] Estimated using model II

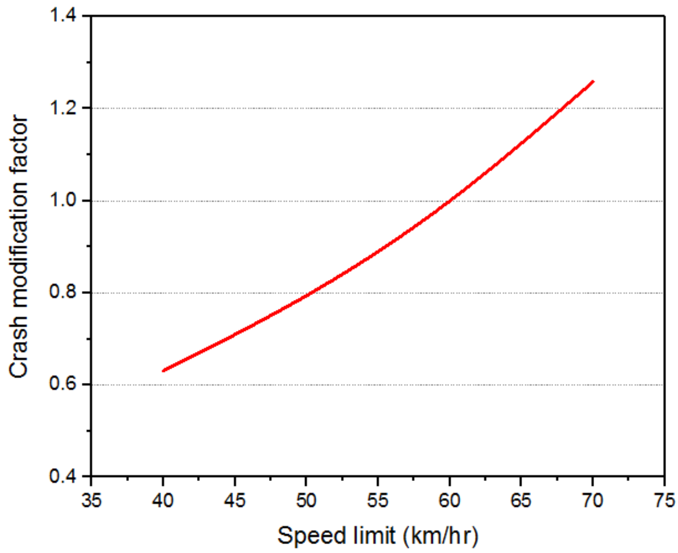

Figure 5.14 CMF for speed limit

Summary of the effects of Independent Variables

A summary of the effects of the variables on the safety performance of roundabouts is presented in Table 5.24. The extensive literature reviews previously undertaken found that the safety effects of exit lane width and exit radius have not been studied or evaluated. Both were considered to be significant enough to warrant inclusion in the research reported here. However, it is worth noting that the safety effects of all variables is associated with the study area conditions.

Table 5.24 Summary of the CMF results for roundabout

Explanatory variables	**Effect on safety performance**		**Comment**
	Positive Effect	**Negative Effect**	
No. of legs		✓	Significant
No. of lanes Entering			
Major-approach		✓	Significant
Minor-approach		✓	Insignificant
No. of lanes Exiting			
Major-approach		✓	Significant
Minor-approach		✓	Insignificant
Entry width			
Major-approach		✓	Significant
Minor-approach		✓	Significant
Exit width			
Major-approach	✓		Insignificant
Minor-approach	✓		Insignificant
Entry radius			
Major-approach		✓	Significant
Minor-approach		✓	Significant
Exit radius			
Major-approach	✓		Significant
Minor-approach	✓		Significant
AADT			
Major-approach		✓	Significant
Minor-approach		✓	Significant
Fixed object	✓		Insignificant
Circulatory roadway width		✓	Insignificant
Weaving length	✓		Insignificant
Weaving width		✓	Significant
Central island diameter	✓		Insignificant
Speed limit		✓	Significant

5.6 Combined CMFs for Roundabout Crashes

As described earlier, the Empirical Bayes (EB) approach was applied to determine the most hazardous roundabouts in Toowoomba city. The top 10 hazardous roundabouts were then selected to investigate the possible treatments using crash modification factors for single and combined treatments. Combined treatments can be defined as a technique where more than one single treatment is applied at the same time (Park et al. 2014). Four different techniques were used to estimate the effect of combined treatments on safety at roundabouts: (i) HSM technique; (ii) apply only the most effective CMF technique; (iii) systematic reduction of a subsequent CMFs technique; and (iv) Turner technique. These techniques were also discussed in detail in Chapter 2.

5.6.1 Roundabout Characteristics

Using CMFs to identify the effective safety treatments can help to determine the expected impact resulting from treatments such as changes in the geometric design and traffic operation parameters. The main characteristics for the top 10 hazardous roundabouts are provided in this section as the initial step in determining treatments.

1) Roundabout at Anzac Avenue, Hursley Road, and Holberton Street (R_NW7)

Figure 5.15 shows the 4-legged roundabout with give way signs on major and minor approaches. The roundabout is located between Anzac Avenue (major approach), Hursley Road, and Holberton Street. The two minor approaches are not on the same line, and therefore form a skewed roundabout. The red points represent the road crashes, i.e. fatal and serious injury crashes, which occurred between 2010 and 2015. It should be noted that some of these points refer to more than one crash due to the recorded of crash locations using the same coordinates. It can be seen that a larger number of crashes have occurred at the entry of the major approach. This may be due to the presence of two entry lanes with different movement patterns, which confuses the drivers. The traffic volume on the major and minor approaches was 15,700 and 7,400 vehicles per day, respectively.

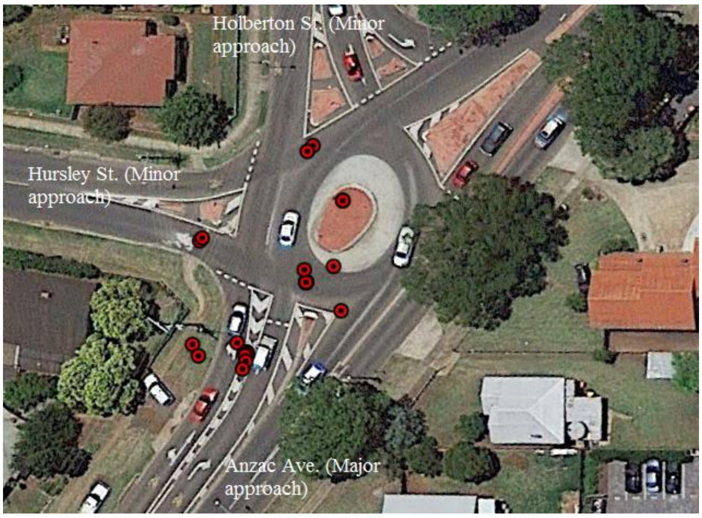

Figure 5.15 Roundabout R_NW7 between Anzac Avenue, Hursley, and Holberton Street
(Source: Aerial Image from Google Earth pro)

2) Roundabout at Ramsay Street and Alderley Street (R_SE11)

The second roundabout is located between Ramsay Street (major approach) and Alderley Street (minor approach) as shown in Figure 5.16. The major approaches have a total of two entry lanes and two exit lanes and the same goes for the minor approaches. During the study period, the average traffic volumes on the major and minor approaches were 8,700 and 7,700 vehicles per day, respectively.

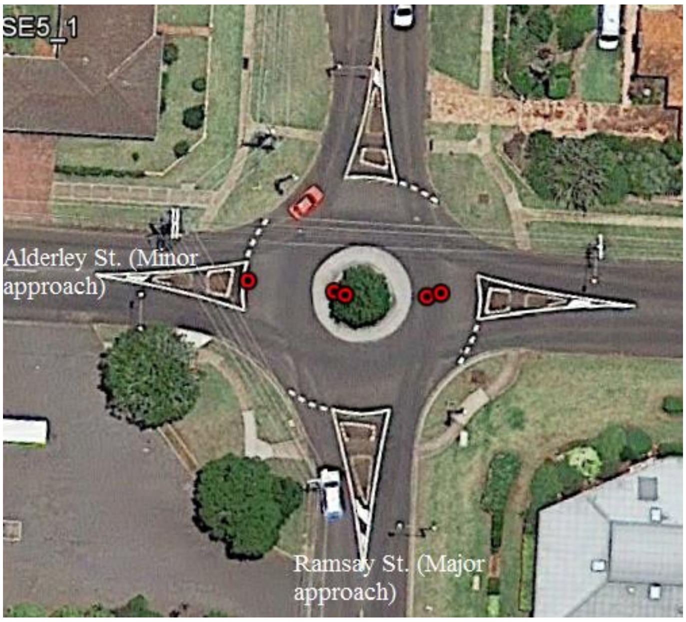

Figure 5.16 Roundabout R_SE11 between Ramsay Street and Alderley Street
(Source: Aerial Image from Google Earth pro)

3) Roundabout at Greenwattle Street and Glenvale Street (R_SW3)

Figure 5.17 shows a 4-legged roundabout, where the major approaches have a total of two entry lanes and two exit lanes and the same goes for the minor approaches. This roundabout is located between Greenwattle Street (major road) and Glenvale Street. The give way sign and splitter island are present on each entering approach and also there is a tree located on the central island. The traffic volumes on the major and minor approaches were 8,100 and 6,600 vehicles per day, respectively.

Figure 5.17 Roundabout R_SW3 between Greenwattle Street and Glenvale Street
(Source: Aerial Image from Google Earth pro)

4) Roundabout at Glenvale Street and McDougall Street (R_SW2)

The fourth roundabout is located between Glenvale Street (major approach) and McDougall Street as shown in Figure 5.18. The traffic volumes on the major and minor approaches were 5,400 and 4,200 vehicles per day, respectively. Although only a small number of severe crashes (fatal and serious injury) occurred on this roundabout, it was considered as a hazardous roundabout due to the predicted crashes using EB approach, which found less than the expected number crashes. More specifically, this approach depends not only on the number of crashes in identifying the hazardous roundabouts but also on the geometric and traffic volume characteristics.

Figure 5.18 Roundabout R_SW2 between Glenvale Street and McDougall Street
(Source: Aerial Image from Google Earth pro)

5) Roundabout at Curzon Street and Herries Street (R_NE1)

This roundabout is located between Curzon Street (major approach) and Herries Street (minor approach). Figure 5.19 shows a 4-legged roundabout, where the major approaches have a total of two entry lanes and two exit lanes and the same goes for the minor approaches. There is also a give way sign and splitter island present on each entering approach and there is no fixed object located on the central island. The traffic volumes on the major and minor approaches were 6,600 and 3,100 vehicles per day, respectively.

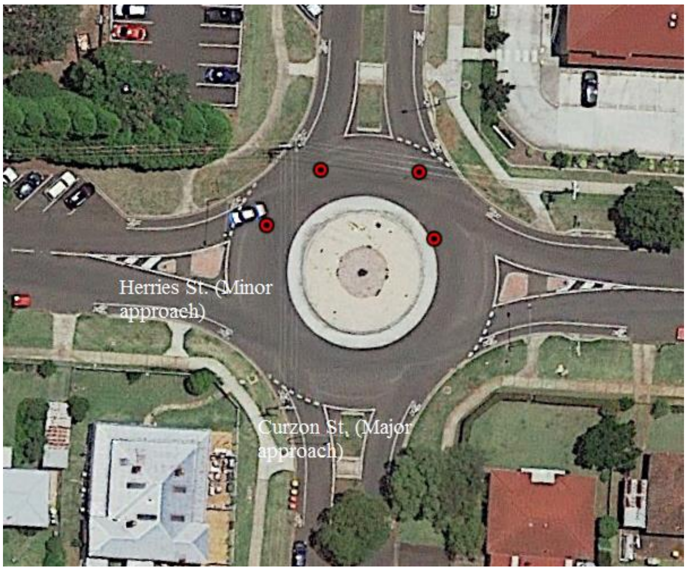

Figure 5.19 Roundabout R_NE1 between Curzon Street and Herries Street
(Source: Aerial Image from Google Earth pro)

6) Roundabout at Bridge Street and Mackenzie Street (R_NE4)

Figure 5.20 shows a 4-legged roundabout with a give way sign on each approach. This roundabout is located between Bridge Street (major approach) and Mackenzie Street. Both major and minor approaches have a splitter island and there is a tree on the central island. The traffic volumes on the major and minor approaches were 4,600 and 4,400 vehicles per day, respectively.

Figure 5.20 Roundabout R_NE4 between Bridge Street and Mackenzie Street
(Source: Aerial Image from Google Earth pro)

7) Roundabout at James Street and Burke Street (R_NE7)

The roundabout is located between James Street (major approach) and Burke Street. The traffic volumes on the major and minor approaches were 5,200 and 1,900 vehicles per day, respectively. A give way sign and splitter island are present on each entering approach and also there is a tree located on the central island as shown in Figure 5.21.

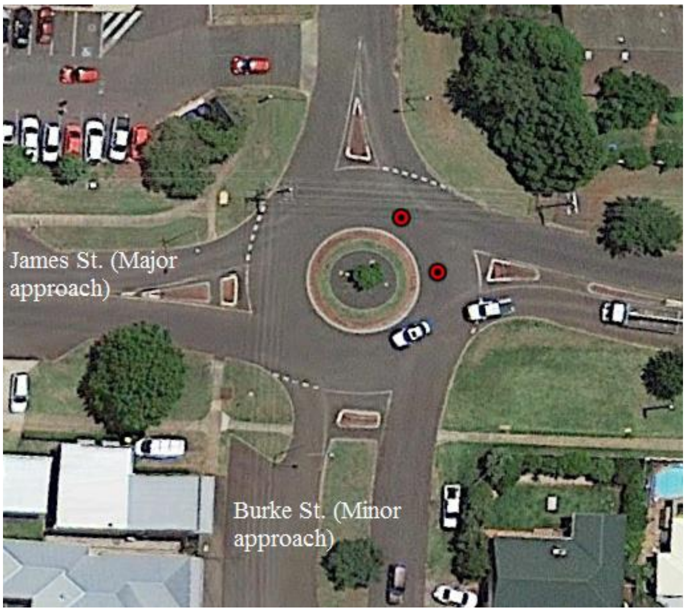

Figure 5.21 Roundabout R_NE7 between James Street and Burke Street
(Source: Aerial Image from Google Earth pro)

8) Roundabout at Spring Street and Mackenzie Street (R_SE2)

Figure 5.22 shows a 4-legged roundabout, where the major approaches have a total of two entry lanes and two exit lanes and the same goes for the minor approaches. This roundabout is located between Spring Street (major approach) and Mackenzie Street (minor approach). The traffic volumes on the major and minor approaches were 2,700 and 2,400 vehicles per day, respectively. This roundabout has only two crashes and has the lowest traffic volume compared to other hazardous roundabouts. As discussed earlier, the EB approach depends not only on the number of recorded crashes to estimate the predicted and expected crashes, but also on roundabout traffic and geometric characteristics.

Figure 5.22 Roundabout R_SE2 between Spring Street and Mackenzie Street
(Source: Aerial Image from Google Earth pro)

9) Roundabout at Ramsay Street and Stenner Street (R_SE6)

Figure 5.23 shows a 4-legged roundabout, where the major approaches have a total of two entry lanes and two exit lanes and the same goes for the minor approaches. This roundabout is located between Ramsay Street (major approach) and Stenner Street. A give way sign and splitter island are present on each entering approach and there is no fixed object located on the central island. The traffic volumes on the major and minor approaches were 7,900 and 7,400 vehicles per day, respectively.

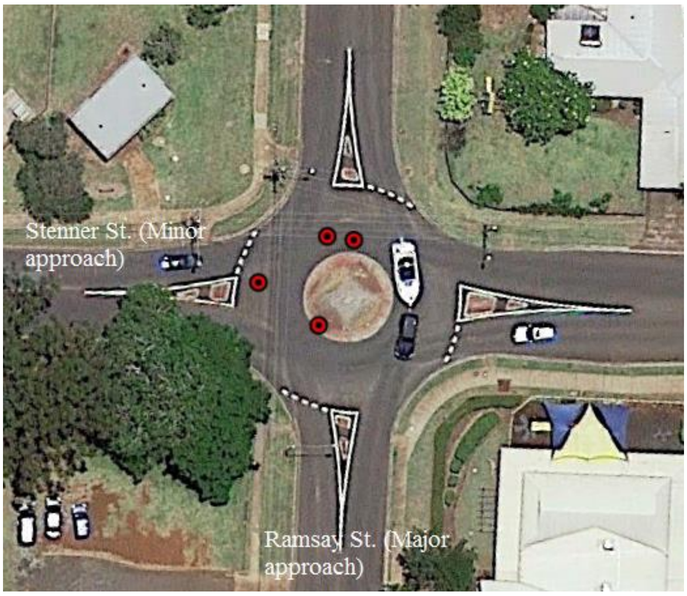

Figure 5.23 Roundabout R_SE6 between Ramsay Street and Stenner Street
(Source: Aerial Image from Google Earth pro)

10) Roundabout at Mackenzie Street and South Street (R_SE13)

The last roundabout is located between Mackenzie Street (major approach) and South Street (minor approach) as shown in Figure 5.24. The traffic volumes on the major and minor approaches were 8,800 and 3,700 vehicles per day, respectively. A give way sign and splitter island are present on each entering approach and also there is no fixed object on the central island.

Figure 5.24 Roundabout R_SE13 between Mackenzie Street and South Street
(Source: Aerial Image from Google Earth pro)

5.6.2 Roundabout Treatment Identification

After identifying the top 10 roundabouts and their characteristics, the possible road safety treatments were determined for each one. The treatments or countermeasures were evaluated using values of CMF for single and combined treatments to determine the expected reduction in road crashes. As discussed earlier, the average values from four different techniques have been adopted to estimate the impact of combined treatments on road safety. Table 5.25 shows the values of CMF for all suggested treatments. The highlighted row identify the most effective single treatment.

In the first step, the CMFs were estimated for each single treatment type and then ranked, starting with the most effective treatment. Thereafter, the combined CMFs were estimated gradually, starting with two suggested treatments and then adding one treatment each time, using four different techniques as shown in Table 5.26. This method, i.e. gradual estimation, was adopted to identify the effect of each single treatment on the total expected crash reduction.

The analysis using ten treatments for roundabout R_NW7, resulted in an expected road crash reduction after applying all of these treatments together was 68% (i.e., CRF=100-CMF%). From Tables 5.26, it is seen that the same expected crash reduction of 68% was achieved after introducing the first seven treatments, which means that there is no significant impact on safety due to the last three treatments. Likewise, six treatments were suggested for roundabout R_SE11 with the crash reduction 69%. Ten treatments were also suggested for the roundabout R_SW3 with the crash reduction 73%. The safety of this roundabout was not affected after introducing the last two treatments. For roundabout R_SW2 there were seven suggested treatments with an expected crash reduction of 75%. Although most roundabouts have more suggested treatments, this roundabout has a higher crash reduction. This means that crash reduction not only depends on the number of treatments, but also on the type of those treatments. Nine treatments were suggested for each of the roundabouts R_NE1 and R_NE7 with crash reductions of 58% and 65%, respectively. The same values of crash reduction were achieved for both R_NE1 and R_NE7 after introducing the eighth treatment and sixth treatment, respectively. Seven treatments were suggested for the roundabout R_NE4 with road crash reduction 71% and this value was also achieved after introducing the fifth treatment. There are also only five treatments suggested for roundabout R_SE2 with crash reduction 51%. Eleven treatments were suggested for roundabout R_SE6 with crash reduction 73%, and this value of crash reduction was achieved after the eighth treatment. Finally, ten treatments were suggested for roundabout R_SE13 with a crash reduction 72% and this value was also achieved after the eighth treatment. The results indicate that although maximum benefit is gained with a reasonable large number of treatments (e.g., between 6 and 10 treatments), the application of more than three treatments usually results in only a minor improvement in crash reduction (see Table 5.26).

Table 5.25 Estimated CMFs for single treatment at roundabouts

Suggested treatments	**Labelling**	**CMF**	**Std. Er.**	**Suitable for Roundabout**
Reduce entry width on minor by 0.6 m*	0.6_REn_{minors}	0.53	0.039	R_NW7, R_SW3, R_SE6, R_SE13
Reduce entry width on minor by 0.6 m	0.6_REn_{minor}	0.73	0.024	R_NE7
Reduce entry width on major by 0.6 m*	0.6_REn_{majors}	0.69	0.148	R_NW7, R_SW3, R_SE6, R_SE13
Reduce entry width on major by 0.6 m	0.6_REn_{major}	0.83	0.088	R_NE1
Reduce entry width on major by 1.2 m*	1.2_REn_{majors}	0.48	0.102	R_SW2
Increase exit width on major by 0.6 m*	0.6_IEx_{majors}	0.92	0.009	R_NW7, R_SW3, R_SE6, R_SE13
Increase exit width on major by 0.6 m	0.6_IEx_{major}	0.96	0.005	R_NE1
Increase exit width on minor by 0.6 m	0.6_IEx_{minor}	0.99	0.065	R_NE7
Increase exit width on minor by 0.6 m*	0.6_IEx_{minors}	0.99	0.130	R_NW7, R_SW3, R_SE6, R_SE13
Increase exit width on major by 1.2 m*	1.2_IEx_{majors}	0.85	0.008	R_SW2
Reduce entry path radius on major by 10 m*	10_REnR_{majors}	0.53	0.011	R_SE11
Reduce entry path radius on major by 10 m	10_REnR_{major}	0.73	0.007	R_NW7, R_SW3, R_NE1, R_SE6
Reduce entry path radius on major by 20 m	20_REnR_{major}	0.53	0.005	R_NE4
Reduce entry path radius on minor by 10 m*	10_REnR_{minors}	0.50	0.004	R_SE11, R_SW2, R_NE4
Reduce entry path radius on minor by 10 m	10_REnR_{minor}	0.70	0.006	R_SW3, R_NE1, R_NE7, R_SE6
Increase exit path radius on minor by 10 m*	10_IExR_{minors}	0.62	0.009	R_NE7
Increase exit path radius on minor by 10 m	10_IExR_{minor}	0.79	0.012	R_NE4
Increase exit path radius on major by 10 m*	10_IExR_{majors}	0.67	0.015	R_NE7
Increase exit path radius on major by 10 m	10_IExR_{major}	0.82	0.009	R_SE13
Increase exit path radius on major by 20 m	20_IExR_{major}	0.67	0.007	R_SE2
Reduce weaving width by 0.6 m	0.6_RW	0.83	0.120	R_NW7, R_SE11, R_NE4, R_NE7,
Reduce weaving width by 1.2 m	1.2_RW	0.69	0.100	R_SW2, R_SW3, R_NE1, R_SE2, R_SE6,
Reduce weaving width by 1.8 m	1.8_RW	0.58	0.083	R_SE13
Reduce circulatory roadway width by 0.6	0.6_RCr	0.96	0.191	R_NW7, R_SE11, R_NE4, R_NE7,
Reduce circulatory roadway width by 1.2	1.2_RCr	0.93	0.184	R_SW2, R_SW3, R_NE1, R_SE2, R_SE6
Reduce circulatory roadway width by 1.8	1.8_RCr	0.89	0.177	R_SE13
Increase central island diameter by 1.2 m	1.2_ICi	0.98	0.015	R_NW7, R_SE11, R_NE4, R_NE7
Increase central island diameter by 2.4 m	2.4_ICi	0.95	0.015	R_SW2, R_SW3, R_NE1, R_SE2, R_SE6,
Increase central island diameter by 3.6 m	3.6_ICi	0.93	0.014	R_SE13
Add fixed object on central island(e.g. tree)	A_Fixed	0.95	0.275	R_NW7, R_NE1, R_SE6, R_SE13
Reduce speed limit on major approaches from 60 to50 km/hr	$R\_V_{60\text{-}50}$	0.80	0.032	R_NW7, R_SE11, R_SW2, R_SW3, R_NE1, R_NE4, R_NE7, R_SE2, R_SE6, R_SE13

Table 5.26 Estimated CMFs for combined treatments at roundabouts

ID	Suggested Treatments	Combined CMFs				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
R_NW7	0.6_REn_{minors} + 0.6_REn_{majors}	0.37	0.58	0.38	0.53	0.46
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major}	0.26	0.51	0.28	0.53	0.40
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$	0.21	1.39	0.23	0.53	0.36
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW	0.17	0.45	0.20	0.53	0.34
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_IEx_{majors}	0.16	0.44	0.18	0.53	0.33
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_IEx_{majors} + A_Fixed	0.15	0.44	0.18	0.53	0.32
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr	0.15	0.43	0.17	0.53	0.32
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr + 1.2_ICi	0.14	0.43	0.17	0.53	0.32
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr + 1.2_ICi + 0.6_IEx_{minors}	0.14	0.43	0.17	0.53	**0.32**
R_SE11	10_REnR_{minors} + 10_REnR_{majors}	0.27	0.51	0.27	0.50	0.39
	10_REnR_{minors} + 10_REnR_{majors} + $R\_V_{60\text{-}50}$	0.21	0.47	0.20	0.50	0.35
	10_REnR_{minors} + 10_REnR_{majors} + $R\_V_{60\text{-}50}$ + 0.6_RW	0.18	0.45	0.16	0.50	0.32
	10_REnR_{minors} + 10_REnR_{majors} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_RCr	0.17	0.45	0.15	0.50	0.32
	10_REnR_{minors} + 10_REnR_{majors} + $R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi	0.17	0.44	0.14	0.50	**0.31**
R_SW3	0.6_REn_{minors} + 1.2_RW	0.37	0.58	0.38	0.53	0.46
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors}	0.25	0.50	0.27	0.53	0.39
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor}	0.18	0.45	0.20	0.53	0.34
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major}	0.13	0.42	0.14	0.53	0.31
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$	0.10	0.40	0.11	0.53	0.29
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors}	0.09	0.40	0.10	0.53	0.28

Table 5.26 Estimated CMFs for combined treatments at roundabouts (continue)

ID	Suggested Treatments	Combined CMFs				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
R_SW3	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr	0.09	0.39	0.09	0.53	0.27
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr + 2.4_ICi	0.08	0.39	0.08	0.53	0.27
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr + 2.4_ICi + 0.6_IEx$_{minors}$	0.08	0.39	0.08	0.53	**0.27**
R_SW2	1.2_REn$_{majors}$ + 10_REnR$_{minors}$	0.24	0.49	0.23	0.48	0.36
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW	0.17	0.44	0.13	0.48	0.30
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$	0.13	0.42	0.08	0.48	0.28
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$ + 1.2_IEx$_{majors}$	0.11	0.41	0.05	0.48	0.26
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$ + 1.2_IEx$_{majors}$ + 1.2_RCr	0.10	0.40	0.04	0.48	0.26
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$ + 1.2_IEx$_{majors}$ + 1.2_RCr + 2.4_ICi	0.10	0.40	0.03	0.48	**0.25**
R_NE1	1.2_RW + 10_REnR$_{minor}$	0.48	0.66	0.54	0.69	0.59
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$	0.35	0.57	0.45	0.69	0.52
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$	0.28	0.52	0.40	0.69	0.47
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$	0.23	0.49	0.37	0.69	0.44
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr	0.22	0.48	0.35	0.69	0.44
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi	0.21	0.47	0.35	0.69	0.43
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi + A_Fixed	0.20	0.46	0.34	0.69	0.42
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi + A_Fixed + 0.6_IEx$_{major}$	0.19	0.46	0.34	0.69	**0.42**

Table 5.26 Estimated CMFs for combined treatments at roundabouts (continue)

ID	**Suggested Treatments**	**Combined CMFs**				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
R_NE4	10_REnR$_{minors}$ + 20_REnR$_{major}$	0.27	0.51	0.27	0.50	0.39
	10_REnR$_{minors}$ + 20_REnR$_{major}$ + 10_IExR$_{minor}$	0.21	0.47	0.20	0.50	0.34
	10_REnR$_{minors}$ + 20_REnR$_{major}$ + 10_IExR$_{minor}$ + R_V$_{60\text{-}50}$	0.17	0.44	0.15	0.50	0.31
	10_REnR$_{minors}$ + 20_REnR$_{major}$ + 10_IExR$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW	0.14	0.43	0.11	0.50	0.29
	10_REnR$_{minors}$ + 20_REnR$_{major}$ + 10_IExR$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW + 0.6_RCr	0.13	0.42	0.10	0.50	0.29
	10_REnR$_{minors}$ + 20_REnR$_{major}$ + 10_IExR$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi	0.13	0.42	0.10	0.50	**0.29**
R_NE7	10_IExR$_{minors}$ + 10_IExR$_{majors}$	0.42	0.61	0.46	0.62	0.53
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$	0.29	0.53	0.36	0.62	0.45
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$	0.21	0.47	0.29	0.62	0.40
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$ + R_V$_{60\text{-}50}$	0.17	0.45	0.25	0.62	0.37
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW	0.14	0.43	0.22	0.62	0.35
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW + 0.6_RCr	0.14	0.42	0.21	0.62	0.35
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi	0.13	0.42	0.21	0.62	0.35
	10_IExR$_{minors}$ + 10_IExR$_{majors}$ + 10_REnR$_{minor}$ + 0.6_REn$_{minor}$ + R_V$_{60\text{-}50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi + 0.6_IEx$_{minor}$	0.13	0.42	0.21	0.62	**0.35**
R_SE2	20_IExR$_{major}$ + 1.2_RW	0.46	0.64	0.52	0.67	0.57
	20_IExR$_{major}$ + 1.2_RW + R_V$_{60\text{-}50}$	0.37	0.58	0.45	0.67	0.52
	20_IExR$_{major}$ + 1.2_RW + R_V$_{60\text{-}50}$ + 1.2_RCr	0.34	0.56	0.43	0.67	0.50
	20_IExR$_{major}$ + 1.2_RW + R_V$_{60\text{-}50}$ + 1.2_RCr + 2.4_ICi	0.33	0.55	0.42	0.67	**0.49**

Table 5.26 Estimated CMFs for combined treatments at roundabouts (continue)

ID	Suggested Treatments	Combined CMFs				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
R_SE6	0.6_REn_{minors} + 1.2_RW	0.37	0.58	0.38	0.53	0.46
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors}	0.25	0.50	0.27	0.53	0.39
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor}	0.18	0.45	0.20	0.53	0.34
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major}	0.13	0.42	0.14	0.53	0.31
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$	0.10	0.40	0.11	0.53	0.29
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors}	0.09	0.40	0.10	0.53	0.28
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors} + 1.2_RCr	0.09	0.39	0.09	0.53	0.27
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors} + 1.2_RCr + 2.4_ICi	0.08	0.39	0.08	0.53	0.27
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors} + 1.2_RCr + 2.4_ICi + A_Fixed	0.08	0.39	0.08	0.53	0.27
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor} + 10_REnR_{major} + $R\_V_{60\text{-}50}$ + 0.6_IEx_{majors} + 1.2_RCr + 2.4_ICi + A_Fixed + 0.6_IEx_{minors}	0.08	0.39	0.08	0.53	**0.27**
R_SE13	0.6_REn_{minors} + 1.8_RW	0.31	0.54	0.32	0.53	0.42
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors}	0.21	0.47	0.22	0.53	0.36
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$	0.17	0.45	0.17	0.53	0.33
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major}	0.14	0.43	0.13	0.53	0.31
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr	0.12	0.42	0.11	0.53	0.30

Table 5.26 Estimated CMFs for combined treatments at roundabouts (continue)

ID	**Suggested Treatments**	**Combined CMFs**				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
R_SE13	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$ + R_V$_{60-50}$ + 10_IExR$_{major}$ + 1.8_RCr + 0.6_IEx$_{majors}$	0.11	0.41	0.10	0.53	0.29
	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$ + R_V$_{60-50}$ + 10_IExR$_{major}$ + 1.8_RCr + 0.6_IEx$_{majors}$ + 3.6_ICi	0.11	0.40	0.09	0.53	0.28
	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$ + R_V$_{60-50}$ + 10_IExR$_{major}$ + 1.8_RCr + 0.6_IEx$_{majors}$ + 3.6_ICi + A_Fixed	0.10	0.40	0.09	0.53	0.28
	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$ + R_V$_{60-50}$ + 10_IExR$_{major}$ + 1.8_RCr + 0.6_IEx$_{majors}$ + 3.6_ICi + A_Fixed + 0.6_IEx$_{minors}$	0.10	0.40	0.09	0.53	**0.28**

[a] Highway Safety Manual (HSM) technique
[b] Turner technique
[c] systematic reduction of subsequent CMFs technique
[d] apply only the most effective CMF technique

It can be also noticed that the most effective single treatment for the roundabouts R_NW7, R_SW3, R_SE6, and R_SE13 is reducing entry width on minor approaches by 0.6 metres with a crash reduction of 47% whereas, the most effective treatment for the roundabouts R_SE11 and R_NE4 is reducing entry path radius on minor approaches by 10 metres with crash reduction of 50%. The effective treatment for the roundabouts R_SW2, R_NE1, R_NE7, and R_SE2 is reducing entry width on one major approach by 1.2 metres, reducing weaving width by 1.2 metres, increasing exit path radius on minor approaches by 10 metres, and increasing exit path radius on one major approach by 20 metres, respectively. In addition, the crash reduction value for these roundabouts is 62%, 31%, 38%, and 33%, respectively.

In general, the study able to estimate crash modification factors (CMFs) for different treatments at the hazardous roundabouts in Toowoomba city using the cross-sectional method. These values of CMFs will help the council and its engineers in the decision-making process to select the best treatments for safety improvement. In the second stage of this study, the hazardous roundabouts were modelled using VISSIM software to ensure that the suggested treatments will not subsequently impact on the conditions of the traffic operation. Section 5.6 shows the results of the simulation analysis.

5.7 Simulation of Traffic Operations at Treated Roundabouts

Currently, the traffic simulation models have become the most important and useful tools in intelligent transportation system (ITS) related studies. In this study to determine the effect of road safety treatments on traffic operation, the traffic simulation software PTV VISSIM 9.0 was utilized. In the first step the roundabout geometric characteristics and measurements (number of legs, number of entry and exit lanes, lane width, shoulder width, etc.) have been collected using Google Earth Pro and site visits. The traffic volume, vehicle compositions, and speed limit information were also collected. After collecting the required data, PTV VISSIM 9.0 was used to construct the roundabout models based on the existing conditions. Finally, the simulation models were validated to ensure that the models provided realistic simulations. These steps have been applied to the 10 hazardous roundabouts previously identified.

For the validation purposes the study used the two roundabouts, R_SE11 and R_SE6 and the results are shown in Table 5.27. From the table, it can be noticed that the values of traffic delays for both roundabout using the observed data were close to the simulation results. At the same time, the relative error between the observed and simulation results was found to be within ±10 % and considered acceptable (Leng et al. 2008). The simulation parameters for the roundabouts are well validated and can simulate the real situation.

Table 5.27 Validation results of the roundabouts R_SE11 and R_SE6

Roundabout ID	**Observed [a]**		**Simulated**		**Error [b]**
	Delay (sec)	**LOS**	**Delay (sec)**	**LOS**	**%**
R_SE11	18.50	C	17.44	C	-5.7
R_SE6	14.80	B	16.21	C	9.7

[a] obtained from Toowoomba regional council
[b] Error = [Sim. Delay-Obs. Delay)/ Obs. Delay] x 100%

After model construction and validation, the roundabouts were modified based on the suggested treatments to identify the traffic operation conditions before and after implementation of treatments. Table 5.28 shows the values of delay and level of service (LOS) before and after the treatments implementation. The results have been adopted after 10 simulation runs with random seed values to further confirm the simulation results as discussed previously in Chapter 3. Moreover, the simulation time for each run was a total of 3600 seconds with an interval period 600 seconds.

Table 5.28 Comparison of delay and LOS between before and after treatments

Roundabout ID	**Before treatments**		**After treatments**	
	Delay	**LOS**	**Delay**	**LOS**
R_NW7	15.15	C	11.12	B
R_SE11	17.44	C	10.87	B
R_SW3	16.24	C	10.28	B
R_SW2	6.46	A	6.80	A
R_NE1	6.92	A	7.68	A
R_NE4	11.08	B	7.84	A
R_NE7	7.71	A	6.50	A
R_SE2	5.85	A	7.28	A
R_SE6	16.21	C	12.97	B
R_SE13	12.36	B	11.15	B

[b] This result was based on 10-simulation runs with random seed values (see Appendix C)

The results in Table 5.28 indicate that there is no significant change in the values of delay and LOS and the traffic operation was improved at the most of treated roundabouts. For instance, for the roundabout R_NE4, the LOS was changed from B to A and there is no negative impact on traffic operation after applying the suggested treatments. Figure 5.25 and Figure 5.26 show the typical simulation process using PTV VISSIM 9.0 for the roundabout R_SE6 between Ramsay Street and Stenner Street. The figures also display the geometric characteristics before and after treatments implementation. For instance, the central island diameter was increased by 2.4 metres and a tree added to the central island.

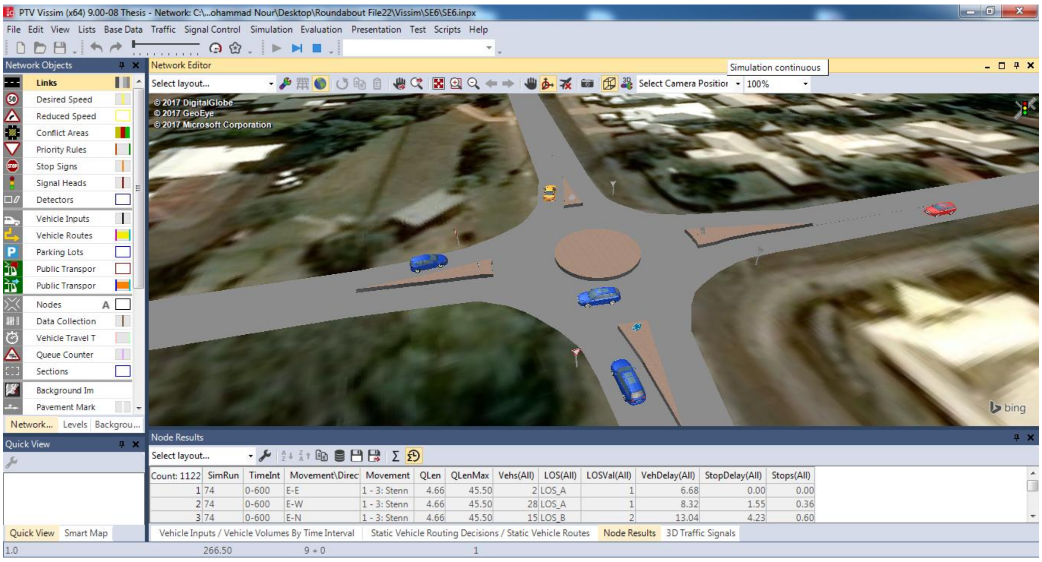

Figure 5.25 Roundabout R_SE6 before treatment implementation

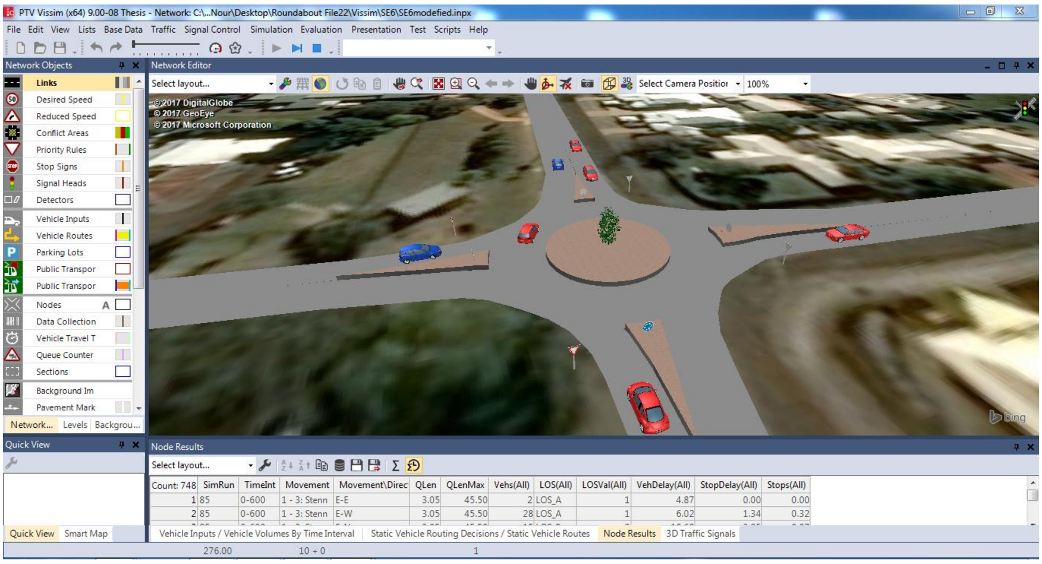

Figure 5.26 Roundabout R_SE6 after treatment implementation

5.8 Benefits and costs of treatments

5.8.1 Benefits

This section discusses the effectiveness of safety improvement treatments on roundabouts based on the total discounted benefits. As has been discussed in Chapter 4, the percentages of road fatalities and road injuries that occurred in the study area were 2.2 % and 97.8 %, respectively. The cost for each type of crash was adopted using the estimation for the year 2006 by BITRE (2009). In the first step of the analysis, the average number of road crashes (per 3-year) before the treatment implementation was determined using the study period 2010-2015. Whereas, the expected number of road crashes after treatment implementation was determined using the values of combined CMFs estimated for all suggested treatments. Table 5.29 shows the total cost savings after applying the treatments or countermeasures for each roundabout. The difference between the crash cost before and after treatments was also calculated to determine the cost saved (benefit) based on 2006 estimates. Ideally, these costs have been adjusted to reflect the cost in 2017 instead of 2006, using an inflation rate of 2.5%. A more detailed discussion was provided in Chapter 3.

Table 5.29 Total cost of roundabout crashes before and after treatments implementation

Roundabout ID	**CMF**	**Ave. crash/ 3year [a]**		**Crash cost [b]/ 3year**		**Saved /3year (2006)**
		Before	**After**	**Before**	**After**	
R_NW7	0.32	8.50	2.70	2,709,675	867,096	1,842,579
R_SE11	0.31	2.50	0.78	796,963	247,059	549,904
R_SW3	0.26	4.50	1.22	1,434,534	387,324	1,047,210
R_SW2	0.25	2.00	0.50	637,571	159,393	478,178
R_NE1	0.42	2.00	0.84	637,571	267,780	369,791
R_NE4	0.29	1.50	0.43	478,178	138,672	339,506
R_NE7	0.35	1.00	0.35	318,785	111,575	207,210
R_SE2	0.49	1.00	0.49	318,785	156,205	162,580
R_SE6	0.27	2.00	0.54	637,571	172,144	465,427
R_SE13	0.28	2.00	0.56	637,571	178,520	459,051

[a] based on the study period 2010-2015
[b] Crash costs are in Australian Dollar (AUD)

5.8.2 Benefit-Cost Analysis

To estimate the present values (PVs) the study has adopted the future benefits discounted rate at a lower value, i.e. 4 %. Previous studies have recommended using a lower or zero value, because this discount rate is inappropriate for evaluating human risk (Litman 2009). Table 5.30 shows the PVs after applying treatments for each roundabout. It should be pointed out that these values have been estimated based on a 10-year treatment life. A detailed explanation of the PVs for combined treatments is provided in Appendix D**.**

Table 5.30 Present values for roundabouts

Roundabout ID	Cost saved /year (2006)	Cost saved/year (2017)[a]	PV [b] ($AUD)
R_NW7	614,193	805,875	6,536,364
R_SE11	183,302	240,508	1,950,732
R_SW3	349,070	458,010	3,714,871
R_SW2	159,393	209,137	1,696,288
R_NE1	123,264	161,733	1,311,796
R_NE4	113,169	148,487	1,204,365
R_NE7	69,070	90,626	735,058
R_SE2	54,194	71,107	576,738
R_SE6	155,142	203,560	1,651,054
R_SE13	153,017	200,772	1,628,437

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%

Using the crash costs, the benefit values can be measured based on the safety treatment type. These values can be also used to estimate the Benefit-Cost ratios (B/C), once the costs (i.e., construction and maintenance costs) associated with each treatment type and location have been evaluated. For the best economic worth of treatments, the discounted benefits should be significantly higher than the costs of treatment implementation and maintenance, i.e. Benefit/Cost ≥1. These benefit-cost ratios help road engineers to make better-informed decisions regarding the choice of appropriate safety treatment for roundabouts.

Table 5.31 shows an example of benefit-cost ratio values for seven proposed treatment types at roundabout R_NW7. It can be noted that applying some of the treatments simultaneously, such as reducing entry width and increasing exit width on major approaches, is more cost-effective than applying only one treatment. For example, the impact of increasing exit width on both minor approaches is not as significant as reducing entry width on both minor approaches, but applying these treatments simultaneously will help in reducing the treatment costs. This table also shows that the optimum cost-effective treatments would be reducing entry widths on both the minor and major approaches of the roundabout with B/C value of 90.35 and 59.60, respectively, noting that costs may increase slightly depending on road conditions.

Table 5.31 Example of the economic feasibility assessment at roundabout R_NW7

Description	**CMF**	**Cost saved / year (2017)**	**PV [c]**	**Treatment Cost [b]**	**B/C**
Reduce entry width on major by 0.6 m	0.69 [a]	367,384	2,979,813	50,000	59.60
Increase exit width on major by 0.6 m	0.92 [a]	94,809	768,984		15.38
Reduce entry width on minor by 0.6 m	0.53 [a]	557,002	4,517,781	50,000	90.35
Increase exit width on minor by 0.6 m	0.99 [a]	11,851	96,123		1.92
Reduce weaving width by 0.6 m	0.83	201,469	1,634,091	100,000	16.34
Reduce circulatory roadway width by 0.6	0.96	47,404	384,492		3.84
Increase central island diameter by 1.2 m	0.98	23,702	192,246		1.92

[a] Estimated for both road approaches, see table 5.25
[b] Source: Toowoomba Regional Council
[c] Based on 10-year treatment life and the discount rate (r) equal 4%

5.9 Overview of Roundabout-Related Treatments

The following treatment plans are recommended for the identified top 10 hazardous roundabouts to achieve the highest crash reductions on the basis of the traffic operational performance and economic benefits:

- The study resulted in ten treatments for roundabout R_NW7 between Anzac Avenue and Hursley Road and Holberton Street (see Figure 5.15). They were: reducing entry width on minor approaches by 0.6 m; reducing entry width on major approaches by 0.6 m; reducing entry path radius on one major approach by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing weaving width by 0.6 m; increasing exit width on major approaches by 0.6 m; adding a fixed object on central island (e.g. tree); reducing circulatory roadway width by 0.6 m; increasing central island diameter by 1.2 m; and increasing exit width on minor approaches by 0.6 m. The estimated road crash reduction after applying the suggested treatments was 68%. It should be noted that crash reduction was not improved by applying the last three treatments; however, these treatments were recommended to reduce the overall cost of implementation. For instance, the first, second, sixth, and tenth treatments can be applied by moving the splitter island 0.6 m towards entry lanes on major and minor approaches. Similarly, the fifth, seventh, and ninth treatments can be achieved by increasing the central island diameter by 1.2 m. The expected crash cost reduction associated with the ten treatments was approximately $AUD 6.5 million. The LOS at this roundabout is also expected to improve from C to B.

- Six treatments were the outcome for roundabout R_SE11 between Ramsay Street and Alderley Street (see Figure 5.16). They were: reducing entry path radius on minor approaches by 10 m; reducing entry path radius on major approaches by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing weaving width by 0.6 m; reducing circulatory roadway width by 0.6 m; and increasing central island diameter by 1.2 m. The last three treatments are interdependent, as increasing central island diameter will increase the weaving and circulatory roadway width. The estimated road crash reduction after applying the treatments was 69%. The expected crash cost reduction associated with the suggested treatments was approximately $AUD 1.9 million. The LOS at this roundabout is expected to improve from C to B.
- Ten treatments were recommended for roundabout R_SW3 between Greenwattle Street and Glenvale Road (see Figure 5.17). They were: reducing entry width on minor approaches by 0.6 m; reducing weaving width by 1.2 m; reducing entry width on major approaches by 0.6 m; reducing entry path radius on one minor approach by 10 m, reducing entry path radius on one major approach by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; increasing exit width on major approaches by 0.6 m; reducing circulatory roadway width by 1.2 m; increasing central island diameter by 2.4 m; and increasing exit width on minor approaches by 0.6 m. In a similar way to the previous roundabouts, most of the suggested treatments are dependent on each other. For instance, the application of the last two treatments did not affect the total crash reduction, but their application was expected to reduce the implementation costs. The estimated road crash reduction after applying the suggested treatments was 73%, and the expected crash cost reduction associated with the treatments was approximately $AUD 3.7 million. The LOS at this roundabout was expected to improve from C to B.
- Seven treatments were the outcome for roundabout R_SW2 between Glenvale Road and McDougall Street (see Figure 5.18). They were: reducing the entry width on major approaches by 1.2 m; reducing the entry path radius on minor approaches by 10 m; reducing weaving width by 1.2 m; reducing posted speed limit on major

approaches from 60 to 50 km/hr; increasing exit width on major approaches by 1.2 m; reducing circulatory roadway width by 1.2 m; and increasing the central island diameter by 2.4 m. The estimated road crash reduction after applying the suggested treatments was 75%, and the expected crash cost reduction associated with the identified treatments was approximately $AUD 1.7 million. The LOS at this roundabout remained unchanged at A.

- Nine treatments were suggested for roundabout R_NE1 between Curzon Street and Herries Road (see Figure 5.19). They were: reducing the weaving width by 1.2 m; reducing the entry path radius on one minor approach by 10 m; reducing entry path radius on one major approach by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing entry width on one major approach by 0.6 m; reducing circulatory roadway width by 1.2 m; increasing central island diameter by 2.4 m; adding a fixed object on the central island (e.g. tree); and increasing the exit width on one major approach by 0.6 m. The highest estimated crash reduction was obtained after applying the first eight treatments. The last treatment is only used to reduce the cost associated with the recommended treatments. The estimated road crash reduction after applying the treatments was 58%. The expected crash cost reduction associated with the suggested treatments was approximately $AUD 1.3 million. The LOS at this roundabout remained unchanged at A.
- Seven treatments were the outcome for roundabout R_NE4 between Bridge Street and Mackenzie Street (see Figure 5.20). They were: reducing entry path radius on the minor approaches by 10 m; reducing entry path radius on one major approach by 20 m; increasing exit path radius on one minor approach by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing weaving width by 0.6 m; reducing circulatory roadway width by 0.6 m; and increasing central island diameter by 1.2 m. The highest estimated crash reduction was obtained after applying the first five treatments for a crash reduction of 71%. The expected crash cost reduction associated with the identified treatments was approximately $AUD 1.2 million. The LOS at this roundabout is expected to improve from B to A.
- Nine treatments were recommended for roundabout R_NE7 between James Street and Burke Street (see Figure 5.21). They were: increasing exit path radius on the

minor approaches by 10 m; increasing exit path radius on major approaches by 10 m; reducing entry path radius on one minor approach by 10 m; reducing entry width on one minor approach by 0.6 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing weaving width by 0.6 m; reducing circulatory roadway width by 0.6 m; increasing central island diameter by 1.2 m; and increasing exit width on one minor approach by 0.6 m. The highest estimated crash reduction was obtained after applying the first six treatments for an estimated crash reduction of 65%. The expected crash cost reduction associated with the identified treatments was approximately $AUD 0.7 million. The expected LOS at this roundabout remained the same at A.

- Five treatments were suggested for roundabout R_SE2 between Spring Street and Mackenzie Street (see Figure 5.22). They were: increasing the exit path radius on one major approach by 20 m; reducing weaving width by 1.2 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; reducing circulatory roadway width by 1.2 m; and increasing central island diameter by 2.4 m. The estimated road crash reduction after applying the suggested treatments was 51%, and the expected crash cost reduction associated with the suggested treatments was approximately $AUD 0.6 million. The expected LOS at this roundabout before and after the identified treatments remained at A.
- Eleven treatments were the outcome for roundabout R_SE6 between Ramsay Street and Stenner Street (see Figure 5.23). They were: reducing the entry width on minor approaches by 0.6 m; reducing weaving width by 1.2 m; reducing entry width on major approaches by 0.6 m; reducing entry path radius on one minor approach by 10 m; reducing entry path radius on one major approach by 10 m; reducing posted speed limit on major approaches from 60 to 50 km/hr; increasing exit width on major approaches by 0.6 m; reducing circulatory roadway width by 1.2 m; increasing central island diameter by 2.4 m; adding a fixed object on central island (e.g. tree); and increasing the exit width on the minor approaches by 0.6 m. The estimated crash reduction after applying the first eight treatments was 73%. The ninth and eleventh treatments were suggested to reduce the implementation costs of other treatments whilst the tenth treatment can be ignored as it does not affect the total crash reduction. The expected crash cost reduction associated with the

identified treatments was approximately $AUD 1.6 million. The LOS at this roundabout is expected to improve from C to B.

- Ten treatments were suggested for roundabout R_SE13 between Mackenzie Street and South Street (see Figure 5.24). They were: reducing the entry width on minor approaches by 0.6 metre; reducing weaving width by 1.8 m; reducing entry width on major approaches by 0.6 metre; reducing posted speed limit on major approaches from 60 to 50 km/hr; increasing exit path radius on one major approach by 10 m; reducing circulatory roadway width by 1.8 m; increasing exit width on major approaches by 0.6 m; increasing central island diameter by 3.6 m; adding a fixed object on central island (e.g. tree); and increasing exit width on minor approaches by 0.6 m. The highest estimated crash reduction was obtained after applying the first eight treatments for an estimated crash reduction of 72%. In addition, the expected crash cost reduction associated with the identified treatments was approximately $AUD 1.6 million. The expected LOS at this roundabout before and after the suggested treatments remained at B.

5.10 Summary

This chapter provides details of the research undertaken on road safety at roundabouts using crash prediction models based on 6 years of crash data i.e., 3 years for model development and 3 years for model validation. Fatal and serious injury crashes were selected for the purpose of analysis and assessment, because the property damage relating only to crash data was incomplete (not reported after 31 December 2010).

The fitted crash models showed that several significant variables affected safety at roundabouts. These variables included traffic volumes on both major and minor approaches, number of entry and exit lanes on major approaches, entry and exit width on major approaches, entry width on minor approaches, entry and exit path radius on both major and minor approaches, weaving length, weaving width, central island diameter, and speed limit. These variables were identified based on a 90 % confidence level.

The Empirical Bayes (EB) method was applied to identify the hazardous roundabouts and rank the roundabouts. This method was used to overcome the problem of regression-to-mean (RTM) bias that is often associated with crash data. The most ten hazardous roundabouts were subsequently investigated. Safety treatments or countermeasures were determined for each of those roundabouts. The treatments were evaluated using crash modification factors (CMFs).

The CMFs were used to identify and select the most appropriate treatments that had positive impacts on road safety at the roundabouts. The effect of combined treatments on road safety was also evaluated using four techniques: highway safety manual (HSM) technique, Turner technique, systematic reduction of subsequent CMFs technique, and apply only the most effective CMF technique.

The crash reduction values were identified after applying single and combined treatments. The highest crash reduction factor (i.e., CRF = 52%) calculated for a single treatment was obtained by reducing entry width on major approaches by 1.2 m. Likewise, the highest crash reduction (i.e., CRF = 75%) for combined treatments was obtained at roundabout R_SW2 (located at the intersection of Glenvale Street and McDougall Street). The combined treatments for this roundabout included a reduced entry width on major approaches of 1.2 m, reduced entry path radius on minor approaches by 10 m, reduced weaving width by 1.2 m, increased exit width on major roads by 1.2 m, reduced speed limit on major approaches from 60 to50 km/hr, reduced circulatory roadway width by 1.2 m, and an increased central island diameter by 2.4 m.

Using PTV VISSIM 9.0, traffic simulation models were developed to investigate the impact of the proposed road safety treatments on traffic operation. The level of service (LOS) and traffic delays were identified before and after implementation of treatments at the hazardous roundabouts. It was found that there was no significant impact on traffic operation (LOS and traffic delay) after the implementation of the proposed treatments. On the other hand, the traffic operations at some of the other treated roundabouts improved (e.g., R_SW3, R_NE4, and R_SE6).

Finally, a benefit-cost analysis was conducted to estimate the total cost that would be saved during the next 10 years after application of treatments. CRFs have been used to estimate these benefits after application of the single and combined treatments, based on the number of road crashes before and after treatment implementation. These estimated costs can help the road authorities to select appropriate treatment types by determining the ratio between the expected benefits and the cost of treatments (i.e., benefit-cost ratio). It was found that the highest cost saving for a roundabouts was around $AUD 6.5 million after application of all suggested treatments at roundabout R_NW7 that is located at the intersection of Anzac Avenue, Hursley Road, and Holberton Street.

Chapter 6
Road Segment Safety Analysis

6.1 Introduction

Road crashes are associated with numerous contributing factors including human factors, geometric features, weather conditions, operational elements or a combination of all. All roads have some level of crash risk, but some road sites (e.g., road segments) are considered to be more dangerous than others. Identifying roadway segments with high crash risk and determining appropriate treatments will improve road safety at those locations. Statistically, the total number of severe-crashes (i.e., fatalities and hospitalised injuries) that occurred in Toowoomba City for the 6 years between 2010 and 2015 was approximately 1650 crashes on roadways, excluding intersection related crashes (Queensland Government 2016).

The success of safety improvement projects in reducing road crashes is founded on the availability of techniques that provide reliable estimates of the road safety level that are associated with current road situations or future situations (i.e., after treatment implementation). This chapter provides details on how geometric and operational elements impact on road crashes and to identify the most appropriate treatments on road segments using single and combined crash modification factor (CMF) techniques. Firstly, the crash prediction models were developed and the Empirical Bayes (EB) approach was applied to identify the hazardous road segments. Subsequently, the impact of all contributing variables to road safety was estimated using CMFs. These safety estimates were also used to identify the appropriate treatments for identified hazardous road segments. Finally, the suggested treatments were evaluated using traffic simulation (PTV VISSIM version 9.0) and the benefits of crash reduction were estimated.

6.2 Data Preparation

Crash data were collected from 84 road segments in Toowoomba city from the Department of Transport and Main Roads, Queensland in Excel spreadsheet format. The data consisted of information about each crash including crash date, severity level, persons involved, location, speed limit and traffic control type. In addition, traffic volume data were obtained from the jurisdiction road authorities of Toowoomba Regional Council and Department of Transport and Main Roads, Queensland. The data related to geometric characteristics of road segments were collected from historical design records, site visits, and Google Earth Pro. A total of 315 police records of crashes were used to accomplish this study and the total length of road segments was 44.7 km. Two criteria were adopted in the road segmentation process. In the first criterion, the definition of road segment was introduced as that part of the road between two main intersections, excluding the intersection boundary that was identified in Chapter 4. In the second criterion, the road segment was defined as a homogeneous segment in which the values of all explanatory variables (i.e., traffic volume, lane width, shoulder width, etc.) to be used in the model are constant, and therefore the risk is relatively uniform. Overall, the presence of an intersection, or the change in the value of any variable, results in the start of a new segment. Figure 6.1 shows schematically how road segment boundaries were adopted.

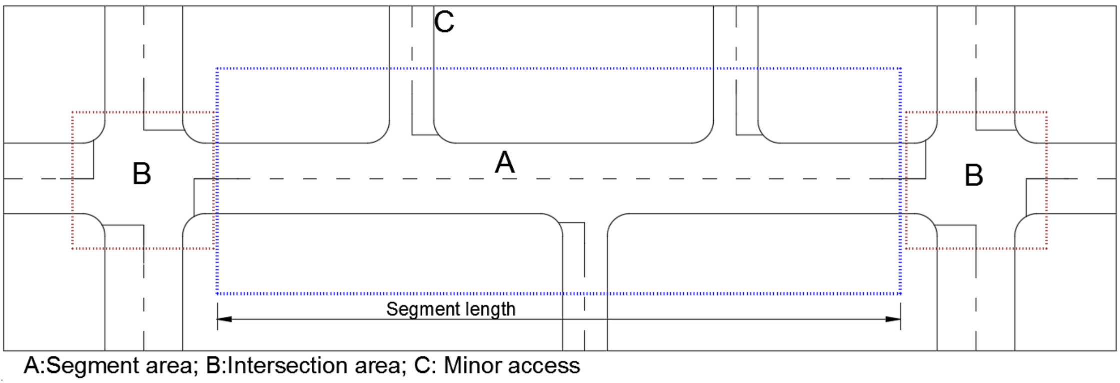

Figure 6.1 Road segment as defined in this study

The road segments were selected based on the geographic location in the study area to prevent bias towards high or low crash frequency locations as described in Chapter 4. The study area was divided into four quadrants (i.e., NE, NW, SE, and SW) to provide a uniform distribution for data selection as shown earlier in Figure 4.1. The segments have been defined using the quadrant symbol with numbers as indicated below. Details of all segments are provided in Appendix B.

Segment ID	On road name	From	To
S_NE4	James Street	Ruthven Street	Fitzgibbon Street
S_NW4	West Street	Campbell Street	Bridge Street
S_SE4	Spring Street	Hume Street	Ramsay Street
S_SW4	West Street	Alderley Street	Peak Street

The study period covered 6 years from 2010 to 2015, where the first 3-year period was used for model development and the second 3-year period was used for model validation. Ten explanatory variables describing traffic and road geometry were used as the most common factors that have been associated with road crashes at road segments. The following is a detailed description of these variables:

1. <u>Road segment length:</u> This is the length of a portion of a road with uniform traffic and geometric characteristics.
2. <u>Road AADT:</u> This variable is the traffic volume as Annual Average Daily Traffic (AADT) on a specific segment.
3. <u>Number of lanes per direction:</u> This variable is the total number of lanes for each direction of traffic movement.
4. <u>Lane width:</u> This is the lateral dimension of a lane, perpendicular to the traffic direction, measured from the faces of curbs and the central lane marking.
5. <u>Shoulder width:</u> This is the width of a portion of the road contiguous with the vehicular way that is used by bicycles, stopped vehicles, and for emergency use.
6. <u>Median island:</u> This variable is the presence or absence of a raised median island on the roadway.
7. <u>Road marking:</u> This variable is the presence or absence of a road marking in the edge line of the roadway as well as in the centre line of the roadway.
8. <u>Grade (%):</u> This variable measures the road segment's steepness as it falls and rises along the road, and is often expressed as a percent.
9. <u>Speed limit:</u> This variable is the speed limit in kilometres per hour on the road segment.

10. Number of access points: is the number of minor crossing and exit roads along the road segment, used for vehicles entering and departing.

6.3 Developing Crash Prediction Models for Road Segments

The models relate the observed number of crashes to traffic volume, traffic control and road design. A generalised linear modelling (GLM) technique was used to fit the models, and the distributions of crash counts were initially assumed to follow a negative binomial (NB) distribution. The NB distribution is appropriate for crash modelling when the observed variance is larger than the mean of the dataset; this phenomenon is often called “over-dispersion”. The study used two tests to investigate whether the dataset is over-dispersed or not. These tests were (i) the value of the deviance divided by degree of freedom *(df)* and (ii) the Pearson Chi-square (x^2) divided by degree of freedom *(df)*. More detailed discussion of these tests is provided in Chapter 3. The regression analyses were carried out using the SPSS software version 22.

6.3.1 Identifying Possible Models using Correlation Matrix

The Pearson’s correlation analysis was assessed to identify the correlation values between contributing variables. This analysis gives the degree of linear relationship between any pair of variables. In the case where the predictor variables are strongly correlated (i.e., the correlate between 0.5 and 1.0 or -0.5 and -1.0), the standard error of the regression parameters increases, meaning that the estimates are not accurate (Navidi 2008). The correlation values for all predictor variables were identified and the correlate was adopted between -0.49 and +0.49. Table 6.1 shows the correlation matrix of the variables used in the safety models. Notation for each variable is provided in Table 6.2. The variable parameter is considered to be statistically significant at 0.1 significance level (using 90% confidence). Based on the correlation matrix and 0.1 significance level, four road safety models were identified after several trials of a different combination of variables as shown in Table 6.2.

Table 6.1 Pearson's correlation matrix for road segments

Variable		***SL***	***Q***	***NL***	***LW***	***SW***	***MI***	***EL***	***CL***	***G***	***V_s***	***AP***
SL	Pearson Correlation [a]	1.00										
	Sig. (2-tailed)											
Q	Pearson Correlation [a]	-.296	1.00									
	Sig. (2-tailed)	.006										
NL	Pearson Correlation [a]	-.415	.404	1.00								
	Sig. (2-tailed)	.000	.000									
LW	Pearson Correlation [a]	.134	-.461	-.606	1.00							
	Sig. (2-tailed)	.224	.000	.000								
SW	Pearson Correlation [a]	.018	.194	.217	-.364	1.00						
	Sig. (2-tailed)	.873	.077	.048	.001							
MI	Pearson Correlation [a]	-.241	.147	.320	-.096	.250	1.00					
	Sig. (2-tailed)	.027	.182	.003	.384	.022						
EL	Pearson Correlation [a]	.029	.242	.197	-.455	.707	.290	1.00				
	Sig. (2-tailed)	.794	.026	.072	.000	.000	.007					
CL	Pearson Correlation [a]	.020	.411	.231	-.224	-.023	-.402	.014	1.00			
	Sig. (2-tailed)	.854	.000	.035	.040	.839	.000	.896				
G	Pearson Correlation [a]	.113	-.057	-.148	.081	-.223	-.123	-.172	.156	1.00		
	Sig. (2-tailed)	.307	.607	.179	.464	.042	.266	.117	.157			
V_s	Pearson Correlation [a]	.179	.376	.205	-.006	-.104	-.196	-.109	.288	.084	1.00	
	Sig. (2-tailed)	.103	.000	.061	.960	.349	.074	.324	.008	.448		
AP	Pearson Correlation [a]	.436	-.144	-.145	.117	-.143	-.199	-.107	-.037	.013	.185	1.00
	Sig. (2-tailed)	.000	.191	.188	.289	.195	.069	.334	.740	.908	.908	

[a] Listwise N=84

Table 6.2 Variables included in the final road segment models

Variable	**SPSS labelling**	**Model I**	**Model II**	**Model III**	**Model IV**
Road segment length	*SL*	✓	✓	✓	✓
AADT	*Q*	✓	✓	✓	✓
Number of lanes per direction	*NL*		✓		
Lane width	*LW*			✓	
Shoulder width	*SW*			✓	✓
Presence of median island	*MI*	✓			
Presence of road marking					
Edge line	*EL*		✓		
Centre line	*CL*		✓		
Grade (%)	*G*	✓			
Speed Limit (km/hr)	V_s			✓	
Number of access points	*AP*				✓

Descriptions of the independent variables used in the Modelling procedure are provided in Table 6.3. Included in the table is the variable description, SPSS labelling, and variable type. The table also provides the summary statistics of the variables. The road segments considered in this study have larger variations in traffic and geometric characteristics. Thus, the relationship between road crashes and explanatory variables could be analysed with a good degree of accuracy. The developing and testing of the crash models is presented in section 6.3.2.

Table 6.3 Statistical summary of road segment dataset

Variable Description	**N**	**Min.**	**Max.**	**Mean**	**Std. Deviation**	**SPSS labelling**	**Variable Type**
Road Segment Length	84	200.0	1400.0	532.5	232.08	*SL*	Continuous
AADT (ln AADT)	84	2500 (7.82)	21784 (9.99)	10964.6 (9.18)	4874.55 (0.522)	*Q*	Continuous
Number of lanes per direction	84	1.0	2.0	1.37	0.485	*NL*	Count
Lane Width (m)	84	2.9	4.8	3.96	0.853	*LW*	Continuous
Shoulder width (m)[1]	84	0	5.0	1.06	1.684	*SW*	Continuous
Presence of median	84	0	1	0.19	0.395	*MI*	Categorical
Road marking							
Edge line	84	0	1	0.45	0.501	*EL*	Categorical
Centre line	84	0	1	0.92	0.278	*CL*	Categorical
Grade (%)	84	0.43	8.67	3.60	1.824	*G*	Continuous
Speed Limit (km/hr)	84	40	60	58.6	3.847	V_s	Continuous
Access points	84	0	10	2.79	1.770	*AP*	Continuous

[1] combined width

6.3.2 Modelling and Measuring Goodness-of-Fit

Several crash models have been developed to represent road safety on Toowoomba city roads. Out of the various crash models developed, the study has narrowed down four models as shown in Table 6.4. These crash models were selected based on a statistical significance of less than 0.1 and a correlation value between 0.49 and -0.49. The parameters shown in Table 6.4 were substituted into equations in Table 6.5 to estimate the road crashes at road segments. As previously mentioned, the negative binomial (NB) distribution was initially used in an attempt to generate suitable models. The NB distribution was accepted to analyse road segment data as the variance was larger than the mean of the dependent variables, indicating the existence of over-dispersion in the data. This conclusion was verified after applying the two tests to determine if there was over-dispersion in the data. Table 6.6 presents the values of Deviance and Pearson Chi-square (x^2) statistics divided by its degrees of freedom (*df*). It can be seen that all values are within the accepted range of 0.80-1.20 (Bauer & Harwood 2000; Abdul Manan et al. 2013), which means that the NB distribution assumption is accepted to analyse the data. The values of dispersion coefficient (K) shown in Table 6.4 are positive, indicating over-dispersion (Couto & Ferreira 2011).

Table 6.4 Negative binomial parameter estimates for selected road segment models

	Model *I*		**Model *II***		**Model *III***		**Model *IV***	
Parameter	**β**	***P*-Value**[b]	**β**	***P* - Value**[b]	**β**	***P* - Value**[b]	**β**	***P* - Value**[b]
Intercept	-6.380	.001	-8.284	.000	-6.943	.000	-6.719	.004
Segment length (ln SL)	.340	.008	.282	.060	.401	.002	.391	.000
AADT (ln *Q*)	.535	.000	.878	.000	.367	.001	.536	.000
Number of lanes per direction (*NL*)	-	-	-.541	.000	-	-	-	-
Lane width (*LW*)	-	-	-	-	-.135	.000		
Shoulder width (*SW*)	-	-	-	-	-.065	.177	-.062	.174
Presence of a median island (MI)	-.390	.001	-	-	-	-	-	-
Presence of road markings								
Edge line (*EL*)	-	-	-.130	.106	-	-	-	-
Centre line (*CL*)	-	-	-.088	.204	-	-	-	-
Grade (*G*)	.025	.320	-	-	-	-	-	-
Speed limit (km/hr) (V_s)	-	-	-	-	.040	.197	-	-
Access points *(AP)*	-	-	-	-	-	-	.038	.382
Dispersion (K)	.550 [a]		.490 [a]		.610 [a]		.520 [a]	

[a] Computed based on the Pearson Chi-square
[b] significance at 0.1 level

Table 6.5 Summary of the selected models to estimate segment crashes

Model No.	**Model Form**
I	$N_{pre.i} = \ SL_i^{.340}.\ Q_i^{.535}\ .\ e^{(-6.380-\ .390\ MI+\ .025\ G)}$
II	$N_{pre.i} = SL_i^{.282}.\ Q_i^{.878}\ .\ e^{(-8.284-\ .541\ NL-\ .130\ EL-\ .088\ CL)}$
III	$N_{pre.i} = SL_i^{.401}.Q_i^{.367}\ .\ e^{(-6.943-\ .135\ LW-\ .065\ SW+\ .040\ V_S)}$
IV	$N_{pre.i} \ = SL_i^{.391}.\ Q_i^{.536}\ .\ e^{(-6.719-\ .062\ SW\ +\ .038\ CR)}$

$N_{pre,i}$ = predicted crashes along i[th] roadway segment for 3 years

The goodness of fit (GOF) for the selected models was measured in term of Akaike Information Criterion (AIC), Bayesian Information Criterion (BIC), cumulative residual (CURE), and residual plot. As discussed previously, the models with smaller AIC and BIC values are considered better than the other models with high values (Cafiso et al. 2010; Abdul Manan et al. 2013; Young & Park 2013). Based on the values of AIC and BIC presented in Table 6.6, the predicted models were ranked starting with the best model as follows: Model I, Model III, Model II, and Model IV.

Table 6.6 Goodness of fit tests for road segments models

Model	**Parameter**	**Value**	***df*** [a]	**Value/*df***
I	Deviance	84.060	79	1.064
	Pearson Chi-Square (x^2)	69.931		0.885
	Akaike's Info. Criterion (AIC)	283.941		.
	Bayesian Info. Criterion (BIC)	296.095		.
II	Deviance	82.160	77	1.067
	Pearson Chi-Square (x^2)	68.311		0.887
	Akaike's Info. Criterion (AIC)	285.814		.
	Bayesian Info. Criterion (BIC)	302.830		.
III	Deviance	80.307	78	1.030
	Pearson Chi-Square (x^2)	67.002		0.859
	Akaike's Info. Criterion (AIC)	284.519		.
	Bayesian Info. Criterion (BIC)	299.104		.
IV	Deviance	84.926	78	1.089
	Pearson Chi-Square (x^2)	70.491		0.904
	Akaike's Info. Criterion (AIC)	286.638		.
	Bayesian Info. Criterion (BIC)	301.222		.

[a] *df*: degree of freedom

The cumulative residual (CURE) plot for each crash model was also generated as shown in Figure 6.2. It can be seen that the data fits all models along the entire range of values for a selected variable. These CURE plots are based on the traffic volume (AADT) variable due to the fact that all models share this predictor variable. As mentioned earlier in Chapter 3, a good CURE plot is one where the curve fluctuates around the zero-axis and moves up and down without crossing the standard deviation boundaries (±2σ) (Hauer et al. 2004; Abdul Manan et al. 2013). A comparison of all models shows that Model I has closer fluctuation around the zero-axis, which indicates a better fit than other models.

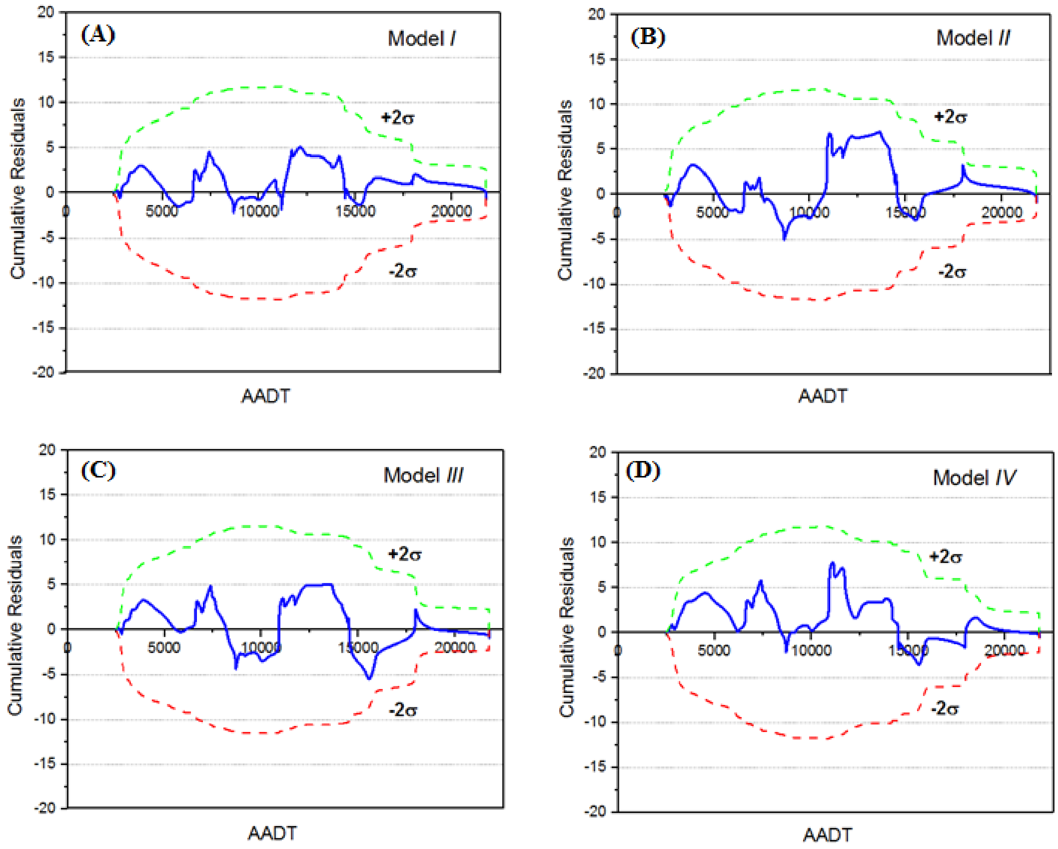

Figure 6.2 Cumulative residual (CURE) plots for road segment models. (A) Model *I*. (B) Model *II*. (C) Model *III*. (D) Model *IV*

The GOF of the models was also examined using the residuals plot method, where the residual values were ranked in increasing order for the natural logarithm of AADT (Log-AADT) variable. The plot exhibits a well-fitted model, when the residual values are located close to but randomly about the zero axis. In contrast, wide horizontal spread represents large residual values. Figure 6.3 shows the plot of the residuals against the Log-AADT for all models. From this plot, it is noticed that Model I has the least spread of all models, i.e., the residual values for Model I range from -2.51 to 3.30. The average spread of the residuals for the Model I was 0.993, while for Model II, Model III, and Model IV it was 1.021, 1.007, and 1.015, respectively. Overall, the GOF measures used in this study show that the Model I is statistically better than other models, but these other models can also be accepted.

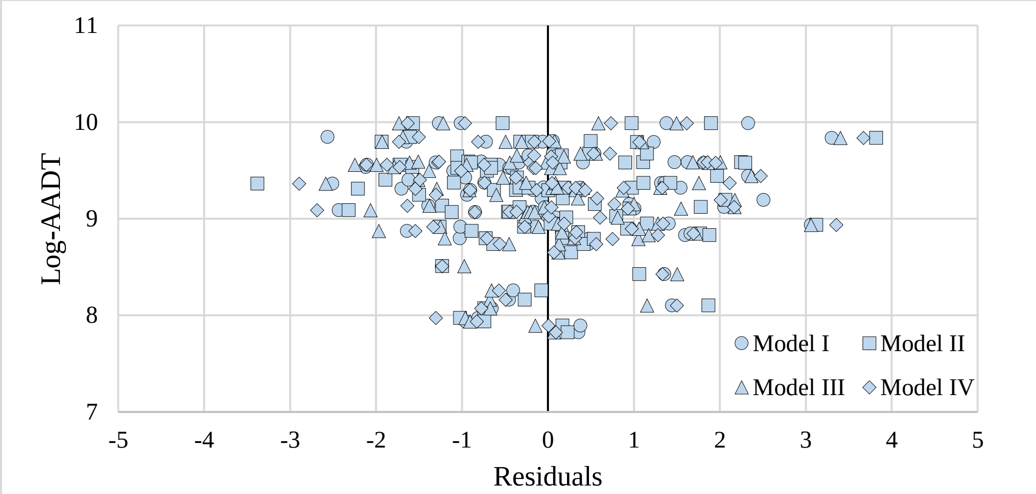

Figure 6.3 Plot of the residuals with Log-AADT at road segments

6.3.3 Model Validation

Several GOF measures were employed to test the validity of the models as no single measure can achieve a completely reliable answer. The data used in this study have been divided into two groups, estimation dataset years (2010-2012) and validation dataset years (2013-2015). The four performance measures were applied to the validation and estimation data including the mean squared prediction error (MSPE), mean absolute deviation (MAD), mean squared error (MSE), and Freeman-Tukey R-Squared coefficient (R^2_{FT}). These measures have been defined previously in Chapter 3. Table 6.7 shows the results of the GOF measures for the estimation and validation dataset.

Table 6.7 Performance measures for all crash prediction models

Performance measures	Model I		Model II		Model III		Model IV	
	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]	2010-12[a]	2013-15[b]
MSPE	-	2.161	-	2.469	-	2.313	-	2.318
MSE	1.759	-	1.755	-	1.695	-	1.748	-
MAD	1.015	1.167	1.021	1.230	0.993	1.206	1.015	1.203
R^2_{FT} %	24.0	17.0	11.9	7.0	19.4	10.9	15.2	8.4

[a] Calculated based on estimation dataset 2010-2012
[b] Calculated based on validation dataset 2013-2015

The values of MSPE using the validation dataset are slightly higher than the values of MSE using the estimation dataset. This indicates that the selected models are slightly over-fitted. The values of MAD using both estimation and validation datasets are slightly similar for all developed models, which indicates a high level of transferability of the models. The values of R^2_{FT} were lower for the validation dataset than that for the estimation dataset, but overall the difference was not significant. These results indicate that the models are performing fairly well for the additional years of data.

6.4 High-Risk Road Segments

The Empirical Bayes (EB) approach was applied to refine the estimate of the expected number of crashes at a site by combining the number of observed crashes with the number of predicted crashes obtained from the safety prediction model, to provide a more accurate result in the safety estimation process at any site. Model I was selected to estimate the predicted number of crashes for each road segment based on the GOF results from the previous section. The weighting adjustment factor (ω) was then calculated using the over-dispersion parameter (K = 0.550, for Model I), road segment length in kilometres, and predicted number of crashes for the study period (2010-2012). The expected number of crashes was then estimated by combining the predicted number of crashes from Model I with the observed number of crashes. Finally, the potential for safety improvement (PSI) values were calculated for ranking the road segments.

6.4.1 Identifying and Ranking High-Risk Road Segments

As described previously, crash prediction models (CPMs) can be used to estimate the average expected crash number for a site. Model I was applied with the EB approach to estimate the expected crash frequency at road segments by considering both the predicted and the observed crash number. The combination between the predicted and the observed crashes number was applied using the weighted adjustment factor (ω). This approach helped to provide unbiased estimates of the long-term expected crashes number for each road segment. In other words, the EB approach reduces the potential bias resulting from the regression-to-the-mean (RTM) effect. The RTM phenomenon reflects the tendency of sites (e.g., roadway segments) that have a higher crash frequency in a particular year to regress to a lower crash frequency in the following year without any safety actions (AASHTO 2010; Persaud et al. 2010; Elvik et al. 2017). This phenomenon was discussed in Chapter 2.

The ranking of black spots is important when road agencies have limited funds to treat a limited number of sites. One of the most logical criteria for ranking of the black spot sites is the potential for safety improvement (PSI) value. This value can be calculated as the difference between the expected number of crashes using the EB approach and the predicted number of crashes for a particular site. The higher the PSI (PSI > 0.0) value, the higher the risk of crash involvement and vice-versa. Table 6.8 shows the predicted and expected crashes number and the PSI values for all road segments. It can be seen that the first 38 road segments have the potential for safety improvement while the remaining 46 road segments have little or no safety improvement potential since the PSI values are negative, i.e. PSI < 0.0. The most dangerous road segment for safety improvement was S_NW22 (Tor Street between Hursley Road and Gatfield Street) with PSI = 3.027.The segment with the least potential for improvement was S_NE4 (James Street between Ruthven Street and Fitzgibbon Street) with PSI = -1.795. Appendix B provides the details of all road segments including road name, location, and crashes number.

Table 6.8 Ranking of road segments using EB approach

Segment ID	Observed [a] (cr./3year)	Predicted (cr./3year)	Segment length (km)	Weighted Adjustment (w)	Expected (cr./3year)	PSI	Rank
S_NW22	6	2.270	0.290	0.188	5.297	3.027	1
S_NE8	5	1.941	0.410	0.277	4.151	2.210	2
S_NW21	4	1.672	0.250	0.214	3.502	1.831	3
S_SW4	5	2.671	0.736	0.334	4.223	1.552	4
S_SW16	5	2.493	0.900	0.396	4.007	1.513	5
S_SE9	4	2.185	0.270	0.183	3.667	1.482	6
S_SE5	4	1.951	0.420	0.281	3.424	1.473	7
S_SW8	4	2.374	0.463	0.262	3.574	1.200	8
S_SW19	4	2.529	0.400	0.223	3.671	1.143	9
S_NW1	3	1.621	0.220	0.198	2.727	1.106	10
S_SW21	4	2.458	0.778	0.365	3.437	0.979	11
S_SW6	4	2.342	0.995	0.436	3.277	0.935	12
S_NE12	3	1.770	0.360	0.270	2.668	0.898	13
S_NW11	4	2.644	0.780	0.349	3.526	0.883	14
S_NE11	2	0.647	0.200	0.360	1.513	0.866	15
S_NW20	3	1.686	0.620	0.401	2.473	0.788	16
S_SW15	4	2.403	1.400	0.514	3.179	0.775	17
S_NW19	3	1.557	0.920	0.518	2.253	0.696	18
S_NE10	3	1.992	0.530	0.326	2.671	0.679	19
S_SW12	3	2.052	0.544	0.325	2.692	0.640	20
S_NE20	3	2.040	0.670	0.374	2.641	0.601	21
S_NE1	3	2.459	0.420	0.237	2.872	0.413	22
S_SE13	2	1.529	0.420	0.333	1.843	0.314	23
S_NE13	2	1.528	0.430	0.338	1.840	0.312	24
S_NW5	3	2.594	0.560	0.282	2.886	0.292	25
S_SE3	3	2.603	0.710	0.332	2.868	0.265	26
S_SW2	3	2.625	0.820	0.362	2.864	0.239	27
S_NW10	2	1.696	0.580	0.383	1.883	0.188	28
S_SE11	1	0.623	0.430	0.557	0.790	0.167	29
S_NW13	1	0.643	0.410	0.537	0.808	0.165	30
S_NW16	2	1.841	0.469	0.317	1.950	0.109	31
S_NW17	2	1.840	0.520	0.339	1.946	0.106	32
S_SE12	2	1.865	0.750	0.422	1.943	0.078	33
S_SW1	2	1.905	0.359	0.255	1.976	0.071	34
S_NE9	2	1.909	0.430	0.291	1.973	0.065	35
S_NW3	3	2.917	0.700	0.304	2.975	0.058	36
S_NE2	2	1.945	0.210	0.164	1.991	0.046	37
S_SE8	3	2.943	0.520	0.243	2.986	0.043	38
S_NW12	2	2.002	0.440	0.286	2.000	-0.001	39
S_SW13	2	2.037	0.500	0.309	2.011	-0.025	40
S_SW14	2	2.051	0.420	0.271	2.014	-0.037	41
S_NE16	2	2.063	0.523	0.316	2.020	-0.043	42
S_NW15	3	3.074	1.180	0.411	3.030	-0.044	43

[a] The total of the observed crash frequency for 3 years (2010-2012)

Table 6.8 Ranking of road segments using EB approach (continue)

Segment ID	Observed [a] (cr./3year)	Predicted (cr./3year)	Segment length (km)	Weighted Adjustment (w)	Expected (cr./3year)	PSI	Rank
S_SE17	2	2.150	0.870	0.424	2.064	-0.087	44
S_SE7	2	2.170	0.700	0.370	2.063	-0.107	45
S_SE10	2	2.176	0.440	0.269	2.047	-0.128	46
S_NW14	2	2.225	0.280	0.186	2.042	-0.183	47
S_SE14	1	1.406	0.760	0.496	1.201	-0.205	48
S_SW7	1	1.452	0.840	0.513	1.232	-0.220	49
S_SW5	2	2.338	0.360	0.219	2.074	-0.264	50
S_SE18	2	2.440	0.710	0.346	2.152	-0.288	51
S_SE15	0	0.811	0.700	0.611	0.495	-0.316	52
S_SE1	2	2.419	0.360	0.213	2.089	-0.329	53
S_SW20	2	2.450	0.370	0.215	2.097	-0.353	54
S_NW23	1	1.518	0.374	0.309	1.160	-0.358	55
S_SE16	0	0.654	0.260	0.420	0.274	-0.379	56
S_NW8	1	1.569	0.420	0.327	1.186	-0.382	57
S_NE17	1	1.620	0.470	0.345	1.214	-0.406	58
S_NW18	1	1.777	0.630	0.392	1.304	-0.472	59
S_SW10	2	2.740	0.620	0.291	2.216	-0.524	60
S_NE19	2	2.723	0.410	0.215	2.155	-0.568	61
S_SE2	1	2.019	0.850	0.434	1.442	-0.577	62
S_SW18	0	0.956	0.340	0.393	0.375	-0.581	63
S_NE18	1	1.847	0.420	0.292	1.248	-0.600	64
S_SW17	1	2.026	0.750	0.402	1.413	-0.613	65
S_SE20	1	1.941	0.430	0.287	1.270	-0.671	66
S_NE3	1	1.904	0.300	0.223	1.201	-0.703	67
S_NE15	1	1.963	0.310	0.223	1.215	-0.748	68
S_NW2	1	2.016	0.322	0.225	1.229	-0.788	69
S_SE4	0	1.234	0.350	0.340	0.420	-0.814	70
S_SE6	0	1.391	0.540	0.414	0.576	-0.816	71
S_NW4	1	2.100	0.350	0.233	1.256	-0.844	72
S_NW6	1	2.307	0.390	0.235	1.307	-1.000	73
S_NE6	2	3.270	0.480	0.211	2.267	-1.002	74
S_SW3	1	2.622	0.705	0.328	1.533	-1.090	75
S_NW9	0	1.695	0.460	0.330	0.560	-1.135	76
S_NE7	0	1.641	0.380	0.296	0.486	-1.155	77
S_NE14	0	1.648	0.340	0.273	0.450	-1.198	78
S_SE19	0	1.703	0.230	0.197	0.336	-1.367	79
S_NE5	0	2.434	0.860	0.391	0.952	-1.482	80
S_SW9	0	2.123	0.370	0.241	0.511	-1.612	81
S_NW7	0	2.108	0.300	0.206	0.433	-1.675	82
S_SW11	0	2.777	0.880	0.366	1.015	-1.762	83
S_NE4	1	3.564	0.839	0.300	1.768	-1.795	84

[a] The total of the observed crash frequency during 3 years (2010-2012)

6.5 Crash Modification Factors for Road Segment Crashes

As outlined earlier, crash modification factors are used to estimate the impacts of safety improvements. Typically, CMFs are estimated using two methods: before and after comparison, and the cross-sectional method. Since before and after data was not generally available for road segments, the cross-sectional method was adopted. In this method the CMF can be derived for a specific treatment from the road safety models as crash modification functions (CMFunctions). In this section the CMFs were estimated for each variable based on the models described in section 6.3.2 and the base conditions.

6.5.1 Description of Base Conditions

The base condition can be defined as the condition associated with a CMF value 1.0 and reflects the current road condition without any safety improvement actions. Base condition values were adopted from previous studies and from the mean values of an individual explanatory variable. For instance, the mean value of traffic volume (AADT) was about 11,000 vehicles per day as shown in Table 6.3 and this value was adopted as a base condition for traffic volume. This issue is further discussed in Chapter 3. Table 6.9 provides details of the base conditions adopted for road segment variables.

Table 6.9 Base conditions for road segments variables

Feature	**Base Values**
Road segment length	500 metres
Traffic volume (AADT)	11,000 vehicle per day
Number of lanes (per direction)	1 lane
Lane width	3.6 metres
Shoulder width	1.0 metres
Presence of median	0 (No median)
Presence of edge marking	0 (No marking)
Presence of centre marking	0 (No marking)
Grade	3%
Speed limit	60 km/hr
Number of minor crossing roads	3 roads

6.5.2 Crash Modification Function

The cross-sectional method was adopted to estimate CMFs based on the crash prediction models (CPMs). In this approach each parameter of the CPM is associated with the one road feature in order to estimate CMF as a function, i.e. $CMF_i = e^{\beta\times(X_i - X_{Base})}$. This function can be used to estimate the reduction (or increase) in road crashes as a result of a treatment implementation. In general, a CMF value greater than 1.0 denotes a situation where the treatment is associated with more road crashes while a CMF less than 1.0 indicates that the treatment is associated with fewer road crashes. CMFs values and standard error (Std. Er.) for each treatment were estimated. When the value of standard error equals 0.1 or less, it indicates that a CMF is more reliable.

Road Segment Length

The road segment length adopted was homogeneous with respect to traffic operation, traffic volume, and geometric design, resulting in variable lengths. Based on the Goodness-of-Fit test, Model I was selected to estimate CMFs at various lengths of road segment. Table 6.10 indicates that the longer segments were associated with more crash risks based on a 500 m segment length as a base condition. This result may be due to the longer homogeneous segment (i.e. constant speed limit, constant number of lanes, constant lane width) which may reduce the driver's attention while driving. Figure 6.4 provides the relationship between the homogeneous segment length and road safety.

Table 6.10 CMFs based on segment length

CMFunction	SL_i	**CMF** [a]	**Std. Er.**
$CMFunction_i = (SL_i/500)^{0.340}$	200	0.73	0.142
(Base condition at 500 metres)	500	1.00	0.193
	750	1.15	0.222

[a] Estimated using model I

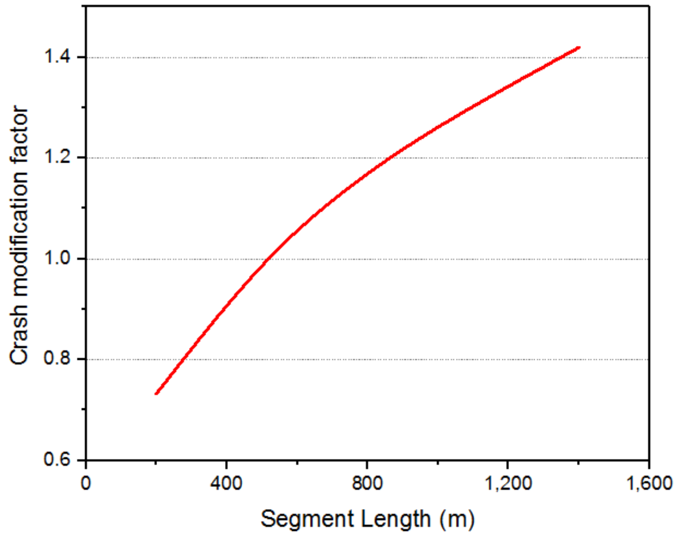

Figure 6.4 CMF for segment lengths

Traffic Volume (AADT)

Traffic volume, in terms of AADT, was used as a key variable for road crash analysis for all road segments in the study area. This variable has been used in previous studies as a significant factor in road segment crashes (Lord & Bonneson 2007; AASHTO 2010). Model I was used to estimate the values of CMF based on the Goodness of Fit test. The base condition for AADT was 11,000 vehicles per day based on its mean value in the datasets. The results indicate that an increase in traffic volume results in an increase in road segment crashes as shown in Table 6.11. This result may be due to the high-speed variability among vehicles in the presence of high traffic volume. Figure 6.5 illustrates the relationship between traffic volumes and road safety based on the range of traffic volume in the dataset. The value of CMF in this research is applicable to traffic volumes ranging from 2,500 to 22,000 vehicles per day.

Table 6.11 CMFs based on traffic volume

CMFunction	***Q_i***	**CMF [a]**	**Std. Er.**
$CMFunction_i = (Q_i/11{,}000)^{0.535}$	6,000	0.72	0.122
	11,000	1.00	0.169
(Base condition at 11,000 veh/day)	16,000	1.22	0.207

[a] Estimated using model I

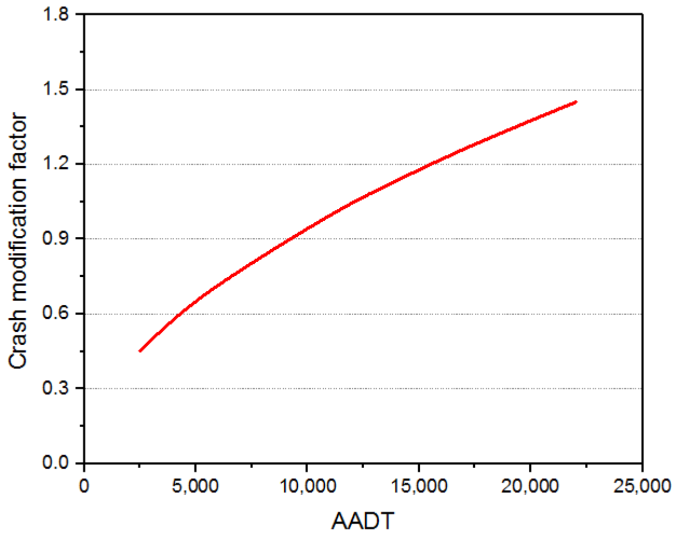

Figure 6.5 CMF for traffic volume

Number of Lanes

The CMFs were estimated for the number of lanes within the road segments using Model II. The base condition was adopted at 1-lane per direction. The results indicate that road crashes were reduced by 42% by adding one lane to a road segment as shown in Table 6.12. This result confirms that adding one lane will increase the level of service for the road segment and reduce the crash risk (Park, Abdel-Aty & Wang et al. 2015). The value of CMF in this research is applicable to the number of lanes changing from 1 to 2 lanes per direction.

Table 6.12 CMFs based on the number of lanes

CMFunction	NL_i	CMF [a]	Std. Er.
$CMFunction_i = e^{-0.541\times[NL_i-1]}$	1	1.00	0.221
(Base condition at 1 lanes)	2	0.582	0.129

[a] Estimated using model II

Lane Width

The impact of lane width on safety performance was estimated for road segments using Model III and a base condition of 3.6-metre lane width as shown in Table 6.9. Table 6.13 shows the values of CMF for various lane widths. The results revealed that as the lane width increases, the number of crashes decreases, which is largely related to driver behaviour and reduced risk of vehicle interactions. For instance, on an undivided road, a reduced lane width resulted in a greater oncoming traffic problem. More specifically, with narrow lane width, drivers tend to drive closer to the centreline and at the same time, the oncoming vehicles tend to move toward the left side of their lanes. Therefore, the wider lane width increases the separation between vehicles travelling in opposing directions. Figure 6.6 illustrates the relationship between lane width and crash risk based on the range of lane width of 2.9 to 4.8 metres in the dataset.

Table 6.13 CMFs based on lane width

CMFunction	LW_i	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{-0.135 \times [LW_i - 3.6]}$	3.0	1.08	0.119
	3.6	1.00	0.110
(Base condition at 3.6 metres)	4.2	0.92	0.101

[a] Estimated using model III

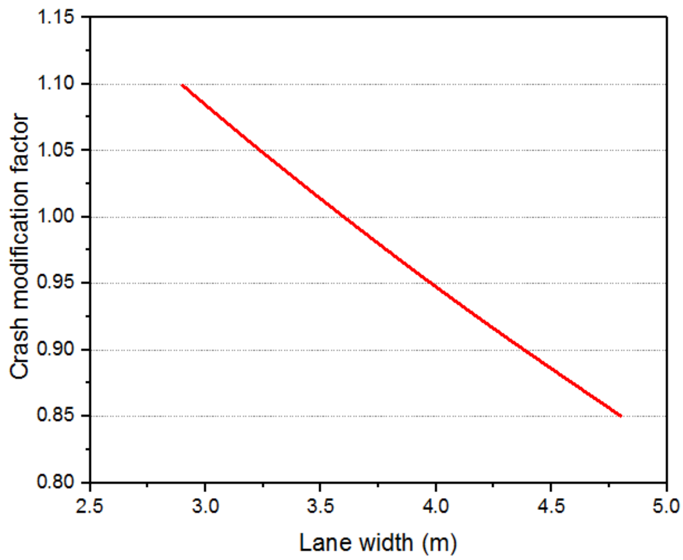

Figure 6.6 CMF for lane width

Shoulder Width

The study examined the effect of shoulder width for road segments using Model III and the findings are shown in Table 6.14. It was found that a wider shoulder width was associated with the lower crash occurrence, likely due to the wider shoulder width providing more lateral clearance for drivers. It should be noted that the impact of shoulder width on road safety was not significant. For instance, a 0.5-metre increase in shoulder width (i.e., on one roadside) decreased the number of crashes by 2.0 %. In general, the shoulder width should not be more than 3.0-metre because some drivers may elect to use this shoulder as another lane, which leads to unsafe driving (Austroads 2005). Figure 6.7 illustrates the relationship between shoulder width and CMF based on the range of shoulder width in the dataset. The value of CMF in this research is applicable to shoulder widths ranging from 0.0 to 2.5 metre. It can be observed that shoulder width has a lower effect on road crashes than lane width.

Table 6.14 CMFs based on average shoulder width (each side)

CMFunction	***SW$_i$***	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{-0.032\times[SW_i-1]}$	0.5	1.02	0.024
	1.0	1.00	0.024
(Base condition at 1.0 metre)	1.5	0.98	0.024

[a] Estimated using model III

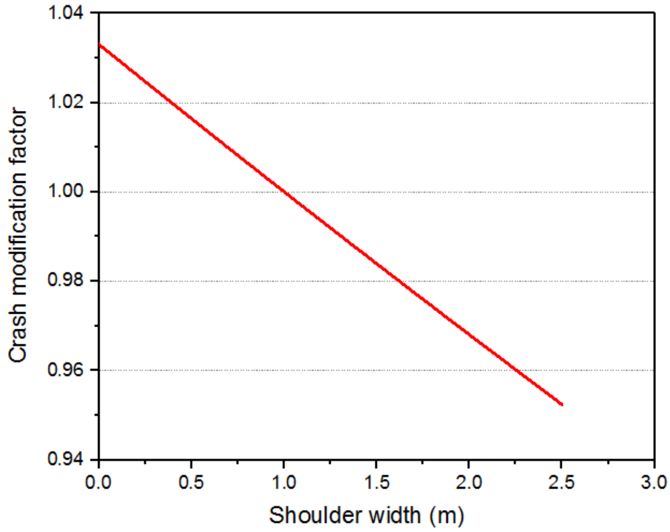

Figure 6.7 CMF for shoulder width

Presence of Median Island

The effect of a median island (raised median) at road segments was investigated using Model I. It was found that adding a median island is associated with lower road crash occurrence. Compared to road segments without a median, segments with a median had a reduction in crashes of 32% for a CMF value of 0.68 as shown in Table 6.15. This result is expected because the separation of opposing vehicles on the roadway using a raised median helps prevent crossover of vehicles into oncoming traffic.

Table 6.15 CMFs based on the presence of median

CMFunction	***MI$_i$***	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{-0.390\times[MI_i-0]}$	0	1.00	0.218
(Base condition at NO median island)	1	0.68	0.147

[a] Estimated using model I

Presence of Road Markings

The values of CMFs have been determined to identify the impact of the presence of road markings on road safety using Model II and a base condition of no road marking. The findings reveal that the presence of centre line and edge line markings have a positive impact on safety performance. In particular, segment related crashes reduced by 12% and 8% after added edge line marking (both directions) and centre line marking respectively, as seen in Table 6.16.

Table 6.16 CMFs based on road marking

CMFunction	X_i	**Edge line**		**Centre line**	
		CMF [a]	**Std. Er.**	**CMF [a]**	**Std. Er.**
$CMFunction_{Edge} = e^{-0.130\times[EL_i-0]}$	0	1.00	0.179	1.00	0.362
$CMFunction_{Centre} = e^{-0.088\times[CL_i-0]}$	1	0.88	0.157	0.92	0.331
(Base condition at NO road marking)					

[a] Estimated using model II

Grade Percentage

Table 6.17 provides values of CMFs for road grades using Model I and a base condition at 3% grade. The study found that higher grades (both upgrade and downgrade) are associated with higher road crashes, of around 2 % increase in crashes per 1 % increase in grade. The result reflects the likelihood that a higher grade percentage may reduce driving visibility (Ratanavaraha & Suangka 2014). Compared with other geometric features, the grade percentages have only a minor impact on road segment crashes. Figure 6.8 illustrates the relationship between grade percentage and road safety based on the range of the grades in the dataset. The value of CMF in this study is applicable to grades ranging from 0.4 to 8.8 %.

Table 6.17 CMFs based on the grade percentages

CMFunction	GL_i	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{0.025\times[GL_i-3.0]}$	2.0	0.97	0.024
	3.0	1.00	0.031
(Base condition at grade 3.0 %)	4.0	1.02	0.039

[a] Estimated using model I

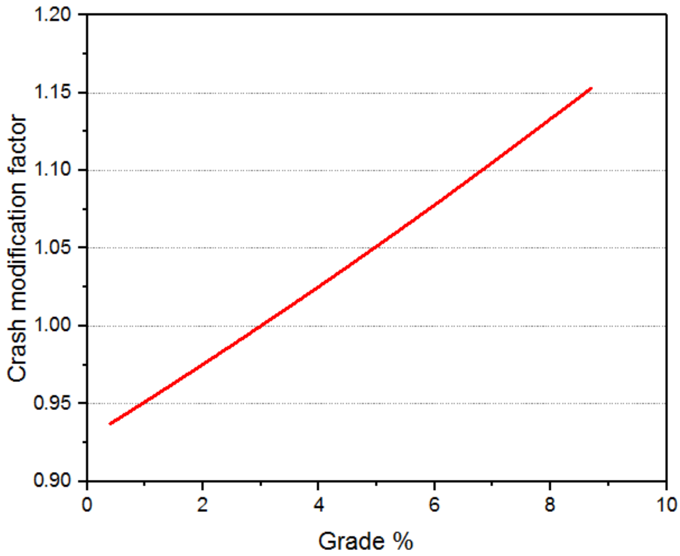

Figure 6.8 CMF for grade percentages

Speed Limit

Previous studies have concluded that the posted speed limit has a direct impact on crash occurrences on any particular road segment (Gargoum & El-Basyouny 2016; Gitelman et al. 2017). In this research, Model III was used to estimate the effect of speed limit on road safety using 60 km/hr as a base condition. It can be seen from Table 6.18 that a 10 km/hr reduce in speed limit from 60 to 50 km/hr would reduce road crashes by around 33%. Figure 6.9 illustrates the relationship between speed limit and road safety based on the range of the speed limit in the dataset. The value of CMF in this research is applicable to the posted speed limit changing between 40 and 60 km/hr.

Table 6.18 CMFs based on Speed limit

CMFunction	V_{si}	**CMF** [a]	**Std. Er.**
$CMFunction_i = e^{0.04\times[V_{si}-60]}$ (Base condition at 60 km/hr)	50	0.67	0.021
	60	1.00	0.031

[a] Estimated using model III

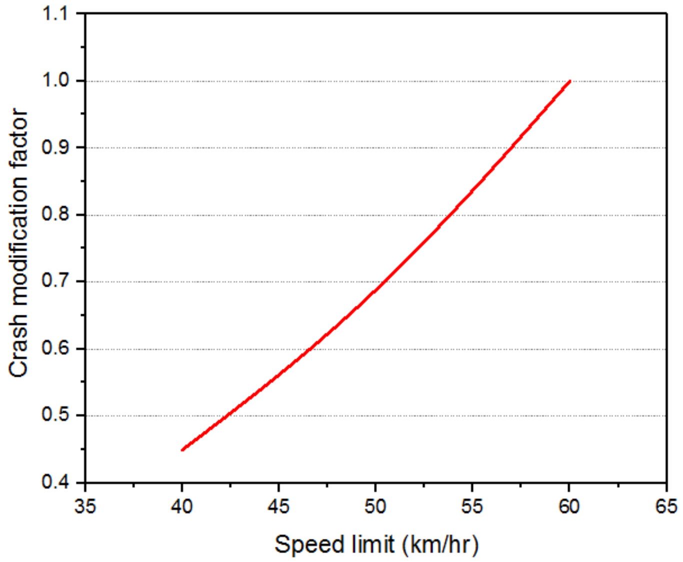

Figure 6.9 CMF for speed limit

Access Points

The effect of access points (i.e., minor crossing roads) along the road segments on crash occurrences was investigated using Model IV and a base condition of 3 access points (Figure 6.1). Table 6.19 shows that more access points were associated with more crash risk. The result was anticipated since an increase in access points increases the number of potential conflict points (i.e., merging and diverging) and thus increases crash probability. Figure 6.10 illustrates the relationship between access points and road safety based on the range of the access points in the dataset. The value of CMF in this research is applicable to the access points ranging from 0 to 10 access points.

Table 6.19 CMFs based on number of access points

CMFunction	***AP***	**CMF [a]**	**Std. Er.**
$CMFunction_i = e^{0.038\times[AP_i-3]}$	2	0.96	0.042
	3	1.00	0.044
(Base condition at 3 roads)	4	1.04	0.045

[a] Estimated using model IV

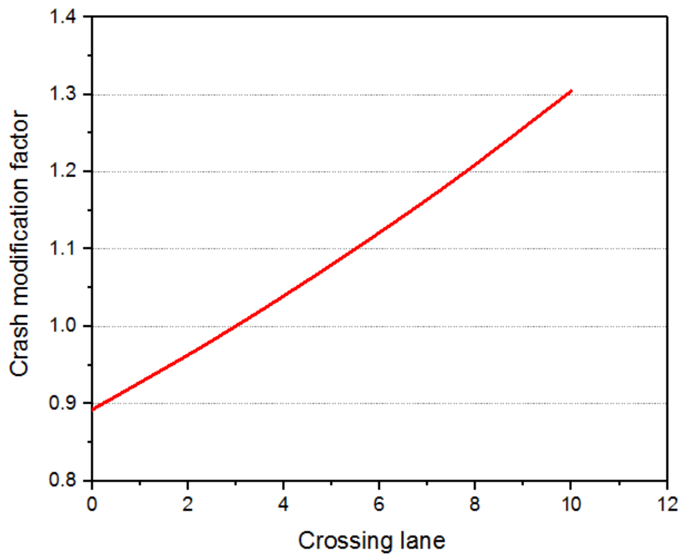

Figure 6.10 CMF for access points

Summary of the effects of Independent Variables

Table 6.20 shows a summary of the effect of individual variables on the safety performance of roadway segments in a study area. The table also shows which variables have significant or insignificant effect on the safety performance based on CMF results.

Table 6.20 Summary of the CMF results for roadway segments

Explanatory variables	**Effect on safety performance**		**Comment**
	Positive Effect	**Negative Effect**	
Segment length		√	Significant
Traffic volume		√	Significant
Number of lanes	√		Significant
Lane width	√		Insignificant
Shoulder width	√		Insignificant
Presence of median island	√		Significant
Presence of edge line marking	√		Significant
Presence of centre line marking	√		Insignificant
Grade (%)		√	Insignificant
Speed limit		√	Significant
Access points		√	Insignificant

It is obvious from Table 6.20 that the combination of individual treatments will likely result in overall increased safety. Cost effective treatments such as reduced speed limits combined with edge line marking stand out. The impact of combined CMFs are investigated in the next section.

6.6 Combined CMFs for Road Segment Crashes

The research used a cross-sectional method to assess the effectiveness of safety improvements using CMFunctions developed to estimate potential changes in road safety after particular treatments. In this section, four techniques of combined CMFs were employed to identify the expected changes in road safety as a result of implementing more than one treatment on a road segment. The techniques were HSM technique, apply only the most effective CMF technique, systematic reduction of a subsequent CMFs technique, and Turner technique. The techniques were reviewed and any related issues identified in Chapter 2. It is important to note that no previous research has been able to identify the most accurate technique in estimating the combined effect of multiple treatments through a comparison with actual safety improvements in a study area. The average value from all four techniques has been adopted to best estimate the effect of multiple treatments at a particular road segment.

6.6.1 Road Segments Characteristics

This section describes the main characteristics for the top 10 most hazardous road segments that were identified earlier using the EB approach. Ideally, this description helped to identify the effective safety treatments for each road segment such as changes in the geometric design and traffic operational features. The road segments identified below are listed starting from the most hazardous segments. In general, the main characteristics of all road segments used in this study are provided in Appendix B.

1) Road segment on Tor Street (S_NW22)

The S_NW22 segment on Tor Street is located between Hursley Road and Gatfield Street. It is a four-lane undivided road with two lanes for each direction, has a segment length equal to 290 metres and has no road shoulders as shown in Figure 6.11. The posted speed limit was 60 km/hr and the grade percentage was about 0.43%. The red points represent the severe road crashes (fatal and serious injury crashes), which occurred between 2010 and 2015. The traffic volume (AADT) on the road segment was 18,600 vehicles per day.

Figure 6.11 Segment S_NW22 on Tor Street
(Source: Aerial Image from Google Earth pro)

2) Road segment on Margaret Street (S_NE8)

The road segment is located on Margaret Street between Clifford Street and West Street as shown in Figure 6.12. It is a two-lane undivided road with one lane for each direction and a segment length of 410 metres. The posted speed limit was 60 km/hr and the gradient was about 8.67 %. The traffic volume on this road segment was 7,600 vehicles per day.

Figure 6.12 Segment S_NE8 on Margaret Street
(Source: Aerial Image from Google Earth pro)

3) Road segment on James Street (S_NW21)

Figure 6.13 shows the third hazardous road segment S_NW21, located on James Street between Mirle Street and Anzac Avenue. It is a four-lane divided road with two lanes for each direction has a segment length of 250 metres, speed limit 60 km/hr, and gradient 2.5 %. The traffic volume on this road segment was 21,800 vehicles per day.

Figure 6.13 Segment S_NW21 on James Street
(Source: Aerial Image from Google Earth pro)

4) Road segment on James Street (S_SW4)

The road segment S_SW4, is located on West Street between Alderley Street and Peak Street as shown in Figure 6.14. It is a two-lane undivided road with one lane for each direction. The segment length equal 736 metres, speed limit 60 km/hr, and gradient 2.65 %. The traffic volume on this road segment was 12,600 vehicles per day.

Figure 6.14 Segment S_SW4 on West Street
(Source: Aerial Image from Google Earth pro)

5) Road segment on Stenner Street (S_SW16)

The fifth hazardous segment in the study area was S_SW16 on Stenner Street, located between West Street and Drayton Road. The segment length was 900 metres with a two-lane undivided road, one lane for each direction as shown in Figure 6.15. The posted speed limit was 60 km/hr and the gradient was about 2.50 %. The traffic volume on this road segment was 9,800 vehicles per day.

Figure 6.15 Segment S_SW16 on Stenner Street
(Source: Aerial Image from Google Earth pro)

6) Road segment on Ruthven Street (S_SE9)

Figure 6.16 shows the segment S_SE9, located on Ruthven Street between South Street and Long Street. It is a four-lane undivided road with two lanes in each direction and has a segment length of 270 metres, a speed limit 60 km/hr, and a gradient 5.37 %. The traffic volume on this road segment was 14,400 vehicles per day.

Figure 6.16 Segment S_SE9 on Ruthven Street
(Source: Aerial Image from Google Earth pro)

7) Road segment on Alderley Street (S_SE5)

Figure 6.17 shows the seventh hazardous segment S_SE5, located on Alderley Street between Ramsay Street and Geddes Street. It is a two-lane undivided road with one lane for each direction has a segment length of 420 metres, speed limit 60 km/hr, and gradient 4.58 %. The traffic volume on this road segment was 9,100 vehicles per day.

Figure 6.17 Segment S_SE5 on Alderley Street
(Source: Aerial Image from Google Earth pro)

8) Road segment on Anzac Avenue (S_SW8)

The eighth hazardous segment in the study area was S_SW8 on Anzac Avenue, located between South Street and Stephen Street. The segment length was 463 metres with a two-lane undivided road, one lane for each direction as shown in Figure 6.18. The posted speed limit was 60 km/hr and the gradient was about 1.20 %. The traffic volume on this road segment was 14,500 vehicles per day.

Figure 6.18 Segment S_SW8 on Anzac Avenue
(Source: Aerial Image from Google Earth pro)

9) Road segment on Anzac Avenue (S_SW19)

The ninth hazardous segment in the study area was S_SW19 on Anzac Avenue, located between Ball Street and Parker Street. The segment length was 400 metres with a two-lane undivided road, one lane for each direction as shown in Figure 6.19. The posted speed limit was 60 km/hr and the gradient was around 5.71 %. The traffic volume on this road segment was 14,500 vehicles per day.

Figure 6.19 Segment S_SW19 on Anzac Avenue
(Source: Aerial Image from Google Earth pro)

10) Road segment on James Street (S_NW1)

The last hazardous segment in the study area was S_NW1 on James Street, located between Ruthven Street and Helen Street. The segment length was 220 metres with a two-lane divided road, one lane for each direction, as shown in Figure 6.20. The posted speed limit was 60 km/hr and the gradient was around 3.00 %. The traffic volume on this road segment was 21,700 vehicles per day.

Figure 6.20 Segment S_NW1 on James Street
(Source: Aerial Image from Google Earth pro)

6.6.2 Segment Treatment Identification

The geometric design and operational characteristics of the top ten hazardous segments were utilized to investigate the appropriate safety treatments. The CMFs were estimated for each type of treatment and ranked starting with the most effective treatment as shown in Table 6.21. The highlighted row identify the most effective single treatment. The CMFs for single treatments were also employed in estimating the combined effects of safety treatments.

In general, the implementation of several safety treatments was seen to be more effective than implementing a single treatment. It is improbable that the full impact of each treatment would be obtained if they were all implemented at the same time (Gross et al. 2010). Therefore, the study has adopted four different techniques to estimate the effects of multiple treatments on road safety. It can be seen from Table 6.22 that the combined CMFs have been estimated starting with two suggested treatments to indicate the effect of each single treatment on road safety using the four techniques.

As seen in Table 6.22, the study has proposed four treatments for each of S_NW22, S_SE9 and S_SW19 which resulted in crash reductions of 52%, 48% and 75%, respectively. It should be noted that segment S_SW19 was not affected by adding the last treatment (i.e. increase shoulder width by 0.5 metres on both sides of the road) and this last treatment at this roadway segment can be ignored. Three treatments were proposed for each of S_NE8, S_SW16, S_SE5, and S_NW1 with road crash reduction 36%, 36%, 36% and 43%, respectively. Two treatments were proposed for each of S_NW21, S_SW4, and S_SW8 with road crash reduction 40%, 34% and 43%, respectively. The most effective single treatment for the segments S_NW22, S_NE8, S_NW21, S_SW4, S_SW16, S_SE9, S_SE5, S_SW8, and S_NW1 was reducing the posted speed limit from 60 km/hr to 50 km/hr whereas, for the segment S_SW19 the most effective treatment was adding one lane for each direction.

It can be noticed from Table 6.22 that the higher expected crash reduction was obtained from segment S_SW19, although the S_NW22 and S_SE9 had the same number of treatments. This means that the value of crash reduction depended not only on the number of treatments but also on the type of treatments. The values of combined CMFs from the four techniques are different from each other and to best estimate combined CMFs, the average value of these techniques (adjustment approaches) was adopted for further investigation and analysis of safety impact and benefit-costs.

Table 6.21 Estimated CMFs for single treatment at road segments

Proposed treatments	**Labelling**	**CMF**	**Std. Er.**	**Suitable for Segment**
Increase lane width by 0.6 m (4-lane)	0.6_ILW_4	0.72	0.079	S_NW22, S_NW1
Increase lane width by 0.5 m (4-lane)	0.5_ILW_4	0.76	0.084	S_NW21
Increase lane width by 0.4 m (2-lane)	0.4_ILW_2	0.90	0.098	S_SW4
Increase shoulder width by 1.5m[a]	1.5_ISW	0.91	0.021	S_SE9, S_NW1
Increase shoulder width by 1.0 m[a]	1.0_ISW	0.94	0.022	S_NE8, S_SW16, S_SE5
Increase shoulder width by 0.5m[a]	0.5_ISW	0.97	0.023	S_SW19
Add median island	AMI	0.68	0.147	S_NW22, S_SE9, S_SW8, S_SW19
Reduce speed limit from 60 to 50 km/hr	$R\_V_{60\text{-}50}$	0.67	0.021	S_NW22, S_NE8, S_NW21, S_SW4, S_SW16, S_SE9, S_SE5, S_SW8, S_SW19, S_NW1
Add edge line [a]	AEL	0.92	0.331	S_NW22, S_NE8, S_SW16, S_SE9, S_SE5
Add one lane on each direction	$1\_L_{dire.}$	0.34	0.652	S_SW19

[a] CMF was estimated for both road direction

Table 6.22 Estimated CMFs for combined treatments at road segments

ID	Suggested Treatments	Combined CMFs				
		Technique 1[a]	Technique 2[b]	Technique 3[c]	Technique 4[d]	Average value
S_NW22	R_V_{60-50} + AMI	0.46	0.64	0.51	0.67	0.57
	R_V_{60-50} + AMI + 0.6_ILW_4	0.33	0.55	0.42	0.67	0.49
	R_V_{60-50} + AMI + 0.6_ILW_4 + AEL	0.30	0.53	0.40	0.67	**0.48**
S_NE8	R_V_{60-50} + AEL	0.62	0.74	0.63	0.67	0.67
	R_V_{60-50} + AEL + 1.0_ISW	0.58	0.72	0.61	0.67	**0.64**
S_NW21	R_V_{60-50} + 0.5_ILW_4	0.51	0.67	0.55	0.67	**0.60**
S_SW4	R_V_{60-50} + 0.4_ILW_2	0.60	0.74	0.62	0.67	**0.66**
S_SW16	R_V_{60-50} + AEL	0.62	0.74	0.63	0.67	0.67
	R_V_{60-50} + AEL + 1.0_ISW	0.58	0.72	0.61	0.67	**0.64**
S_SE9	R_V_{60-50} + AMI	0.46	0.64	0.51	0.67	0.57
	R_V_{60-50} + AMI + 1.5_ISW	0.41	0.61	0.48	0.67	0.54
	R_V_{60-50} + AMI + 1.5_ISW + AEL	0.38	0.59	0.46	0.67	**0.52**
S_SE5	R_V_{60-50} + AEL	0.62	0.74	0.63	0.67	0.67
	R_V_{60-50} + AEL + 1.0_ISW	0.58	0.72	0.61	0.67	**0.64**
S_SW8	R_V_{60-50} + AMI	0.46	0.64	0.51	0.67	**0.57**
S_SW19	$1\_L_{dire.}$ + R_V_{60-50}	0.23	0.49	0.18	0.34	0.31
	$1\_L_{dire.}$ + R_V_{60-50} + AMI	0.15	0.44	0.07	0.34	0.25
	$1\_L_{dire.}$ + R_V_{60-50} + AMI + 0.5_ISW	0.15	0.43	0.06	0.34	**0.25**
S_NW1	R_V_{60-50} + 0.6_ILW_4	0.48	0.65	0.58	0.67	0.60
	R_V_{60-50} + 0.6_ILW_4 + 1.5_ISW	0.44	0.63	0.55	0.67	**0.57**

[a] Highway Safety Manual (HSM) technique
[b] Turner technique
[c] systematic reduction of subsequent CMFs technique
[d] apply only the most effective CMF technique

Overall, the research determined estimates of CMF values for different types of treatments at the hazardous road segments in the study area using a cross-sectional method. These values of CMFs can help road authority planners and transportation safety practitioners to select the most appropriate treatments for safety improvement. In the second stage of this study, the hazardous road segments were simulated using

PTV VISSIM software to investigate the impact of the suggested treatments on the traffic operation. The next section discusses the results of the simulation analysis.

6.7 Simulation of Traffic Operations at Treated Road Segments

As outlined earlier, simulation modelling is considered to be a useful tool to study the effect of improvements to roadway systems. In particular, a simulation model enables a road engineer to predict the effects of different alternative scenarios on the roadway network before implementation and to evaluate the merits of alternative designs. In order to correctly predict the system response, the simulation model needs to reproduce the existing operational conditions. The procedure by which the model parameters are modified so that the simulated response matches with the observed field conditions is known as model calibration.

This section presents the steps that have been followed in the model construction for road segments using the traffic simulation package PTV VISSIM 9.0. In the first step the geometric characteristics and measurements (number of lanes, lane width, shoulder width, grade percentages, etc.) were collected using Google Earth pro and site visits. The traffic volume, vehicle compositions, and speed limit information were obtained from Toowoomba Regional Council and the Department of Transport and Main Roads, Queensland (DTMR). This enabled a detailed and complete description of the site as inputs to produce a realistic outputs. In the second step, the collected data was coded into VISSIM software to model the road segments based on the existing conditions. The last step of model construction involved model validation to ensure that the model provided a realistic simulation. The steps have been applied to the top ten hazardous road segments that were previously identified.

The research used two road segments to verify that the simulation models produced results within acceptable error limits by comparison with observed measurements. Table 6.23 shows the validation results for road segments S_SW4 and S_NW1. As can be seen in this table, travel time in seconds per vehicle was adopted as a performance measure in this stage. The results demonstrated that the relative error between simulation and observed results was found to be within an acceptable range of ±10%, indicating that simulation modelling using VISSIM was capable of simulating real situations for road segments.

Table 6.23 Validation results of the segments S_SW4 and S_NW1

Segment ID	**Travel time**		**Error [b] %**
	Observed [a]	**Simulated**	
S_SW4	38.5	41.59	8.0
S_NW1	12.90	13.89	7.7

[a] obtained from site visiting (using floating car technique)
[b] Error = [Sim. Travel time - Obs. Travel time)/ Obs. Travel time] x 100%

Once the validation was completed, the road segment features were modified according to the identified treatments described in the previous section (Tables 6.21, 6.22), to examine the traffic operation conditions before and after implementation of treatments. Ten simulation runs with random seed values were made for each model. The total simulation time for each run was 3600 seconds with an interval period of 600 seconds. The simulation results based on the average of ten runs for treated and untreated road segments are presented in Table 6.24. The results show that the travel time for all treated segments was slightly higher compared to untreated segments, with increases ranging between 2 and 10 seconds. This was mainly due to the effect of reducing the posted speed limit from 60 to 50 km/hr as one of the suggested treatments. It should be noted that the VISSIM does not directly output average speed. Therefore, the values of average speed in the road segments during the analysis period were calculated using the distance travelled by a particular vehicle in a road segment and the time spent by the vehicle to traverse the segment during the analysis period. More details of the travel time and the average speed on road segments are provided in Appendix C.

Table 6.24 Comparison of travel time and speed between before and after treatments

Segment ID	Before Treatments		After Treatments	
	Travel time	**Ave. speed [a]**	**Travel time**	**Ave. speed [a]**
S_NW22	16.24	58.35	19.24	49.23
S_NE8	19.50	57.67	23.02	48.83
S_NW21	12.08	58.69	14.96	49.35
S_SW4	41.59	58.49	49.05	49.59
S_SW16	53.32	58.62	63.45	49.26
S_SE9	12.83	57.61	15.08	49.29
S_SE5	25.01	57.58	29.02	49.63
S_SW8	22.57	58.70	26.87	49.31
S_SW19	22.26	57.47	25.74	49.66
S_NW1	13.89	59.10	16.53	49.70

[a] Average speed = total distance travelled by vehicle i in the road segment divided by total time spent by vehicle i in a road segment [$v = \frac{\Sigma d_i}{\Sigma t_i}$]

Figure 6.21 and Figure 6.22 provide the geometrical outlines of the segment S_NW1 on James Street as it is modelled by PTV VISSIM and display the geometric characteristics before and after treatments implementation. For instance, the width of the lanes was increased by 0.6 m and 1.5 m shoulders were added to both sides. The simulation models were able to provide the required comparative information to assist making a cost-effective decision about the type of treatment.

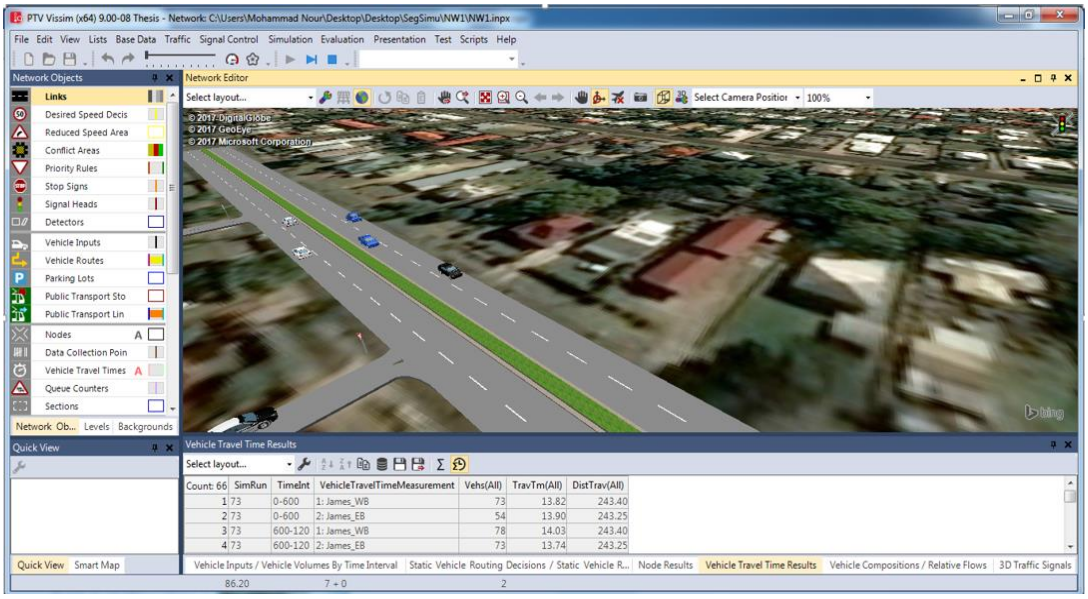

Figure 6.21 Road segment S_NW1 before treatment implementation

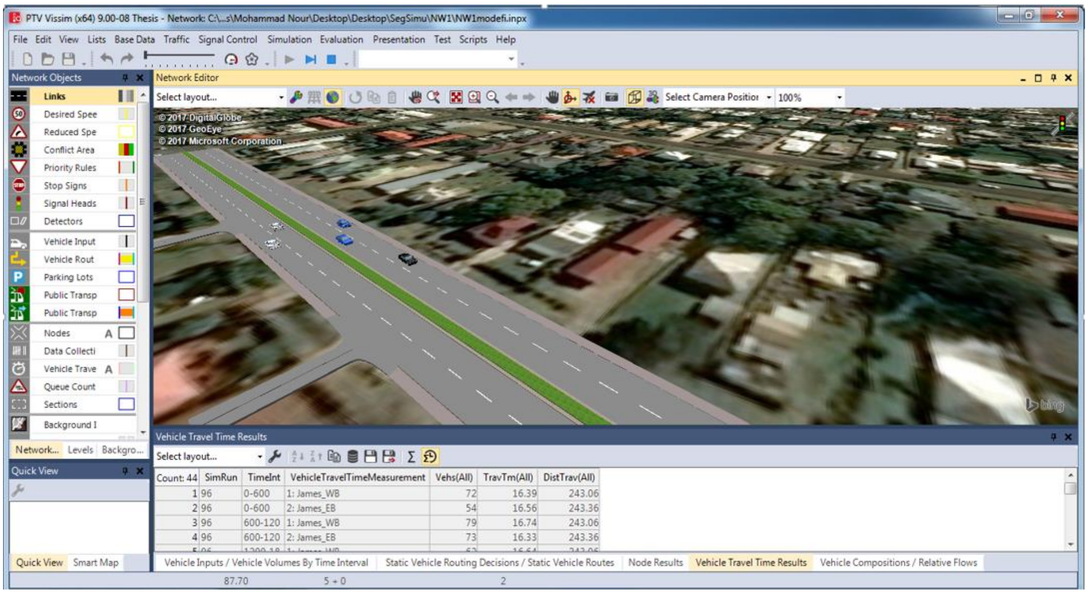

Figure 6.22 Road segment S_NW1 after treatment implementation

6.8 Benefits and Costs of Treatments

6.8.1 Benefits

The total discounted benefits were utilised as an economic criterion to estimate the effects of safety improvements on crash cost reduction. To estimate the crash costs reduction after safety improvements, the percentages of both fatal and serious injury crashes were determined, based on the total crashes that occurred in the study area over a period of 8 years (2008-2015) as detailed in the Table 4.25, Chapter 4. Using these percentages (i.e., 2.2% fatal crashes and 97.8% serious injury crashes), the number of fatal and serious injury crashes can be estimated directly from the total crashes. The cost of crashes was based on the estimation of year 2006 crashes by BITRE (2009) which were then adjusted for inflation. The average value of Australia's inflation rate was used to adjust the crash costs from 2006 to 2017. A more detailed discussion of the inflation rate was provided in Chapter 3. Table 6.25 shows the average crash number over 3 year periods (2010-2015) before and after treatment implementation. The expected number of road crashes after treatment implementations was determined based on the values of estimated combined CMFs, as shown in Table 6.22.

Table 6.25 Total cost of segment crashes before and after treatments implementation

Segment ID	CMF	Ave. crash / 3year [a]		Crash cost [b] / 3year		Saved /3year (2006)
		Before	After	Before	After	
S_NW22	0.47	5.00	2.38	1,593,927	759,898	834,029
S_NE8	0.64	3.00	1.93	956,356	616,617	339,739
S_NW21	0.60	3.00	1.80	956,356	574,292	382,064
S_SW4	0.66	4.50	2.96	1,434,534	942,608	491,926
S_SW16	0.64	3.00	1.93	956,356	616,617	339,739
S_SE9	0.52	3.00	1.57	956,356	501,859	454,497
S_SE5	0.64	2.50	1.61	796,963	513,847	283,116
S_SW8	0.57	3.00	1.70	956,356	543,370	412,986
S_SW19	0.25	3.00	0.74	956,356	235,406	720,950
S_NW1	0.57	3.00	1.72	956,356	547,308	409,048

[a] Based on the study period 2010-2015
[b] Crash costs are in Australian Dollar (AUD)

6.8.2 Benefit-Cost Analysis

Present values (PVs), also known as total discounted benefits, were utilised to calculate the total crash costs reduction (i.e., benefits) for treated roadway segments based on a 10-year treatment life. Table 6.26 shows the PVs of crash costs after implementation of combined treatments for each roadway segment. It can be seen in Table 6.26 that PVs ranged between around \$1.0 to \$3.0 million. These values can be used to assist in the identification of project prioritisation. Using the crash costs, the benefits can be quantified based on the reduction in the expected crashes after a particular type of treatment. The most appropriate safety treatment options should be the treatments that produce the highest benefit for every dollar invested. Full details on the PVs for combined treatments are provided in Appendix D**.**

Table 6.26 Present values (PVs) for road segments

Segment ID	**Cost saved /year (2006)**	**Cost saved/year (2017)[a]**	**PV [b] ($AUD)**
S_NW22	281,593	369,475	2,958,634
S_NE8	114,763	150,579	1,205,190
S_NW21	127,514	167,310	1,355,334
S_SW4	162,581	213,320	1,745,057
S_SW16	114,763	150,579	1,205,190
S_SE9	153,017	200,772	1,612,282
S_SE5	95,636	125,482	1,004,325
S_SW8	137,078	179,858	1,465,028
S_SW19	239,089	313,705	2,557,498
S_NW1	137,078	179,858	1,451,056

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%

The direct costs associated with the implementation of each proposed treatment must also be considered. The Benefit/Cost ratio can then be used to evaluate the cost-effectiveness of proposed safety treatment in terms of crash cost reduction at the treated site and cost of treatment implementation. Table 6.27 provides an example of B/C ratio calculated for two types of proposed treatments at roadway segment S_NW22. The calculated B/C ratios are indicative only as the exact direct costs associated with the all proposed treatments were unavailable and the estimated treatment costs were obtained from the Toowoomba Regional Council. The study estimated the values of PV that can be used by road authorities, Councils, and practitioners to identify the expected B/C ratio for a treated site.

Table 6.27 Example of the B/C ratio at roadway segment S_NW22

Description	**CMF**	**Cost saved / year (2017)**	**PV [a]**	**Treatment Cost [b]**	**B/C**
Add median island	0.68	669,238	5,428,123	50,000	108.56
Add edge line	0.92	167,310	1,357,031	5,000	271.41

[a] Based on 10-year treatment life and the discount rate (r) equal 4%
[b] Source: Toowoomba Regional Council

6.9 Overview of Segment-Related Treatments

A summary of the proposed safety treatments for the top 10 hazardous roadway segments is provided below. The expected values of travel time at these segments after applying the treatment plans increased by an average of 4.5 seconds due to one of the proposed treatments being reduced posted speed from 60 to 50 km/hr.

- The research identified four treatments for segment S_NW22 located on Tor Street between Hursley Road and Gatfield Street (see Figure 6.10). They were: reducing the posted speed limit from 60 to 50 km/hr, adding a median island, increasing lane width by 0.6 m for 4-lane, and adding an edge line on each direction. This segment has enough space to apply the second and third suggested treatments. The estimated crash reduction after applying these treatments was 52%. The expected crash cost reduction associated with the treatments was approximately $AUD 3.0 million. A sample of the benefit-cost ratios for this road segment was provided in Table 6.27. It can be seen that the benefit-cost ratio after adding a median island and adding edge lines was 108.6 and 271.4, respectively.
- Three treatments were identified for segment S_NE8 located on Margaret Street between Clifford Street and West Street (see Figure 6.11). They were: reducing posted speed limit from 60 to 50 km/hr, adding edge line on both directions, and increasing shoulder width by 1.0 metre on both roadsides. The estimated crash reduction after applying the suggested treatments was 36%. The expected crash cost reduction associated with the treatments was approximately $AUD 1.2 million.
- Two treatments were identified for segment S_NW21 located on James Street between Mirle Street and Anzac Avenue (see Figure 6.12). They were: reducing posted speed limit from 60 to 50 km/hr and increasing lane width by 0.5 m for 4-lane. The second treatment can be applied by reducing the median island to an average width of 3.5 m. The estimated crash reduction after applying these treatments was 40%. The expected crash cost reduction associated with the treatments was approximately $AUD 1.4 million.
- Two treatments were suggested for segment S_SW4 located on West Street between Alderley Street and Peak Street (see Figure 6.13). They were: reducing posted speed limit from 60 to 50 km/hr and increasing lane width by 0.4 m for 2-lane. The estimated crash reduction after applying these treatments was 34%. The

expected crash cost reduction associated with the treatments was approximately $AUD 1.7 million.

- Three treatments were identified for segment S_SW16 located on Stenner Street between West Street and Drayton Road (see Figure 6.14). They were: reducing posted speed limit from 60 to 50 km/hr, adding edge line on both directions, and increasing shoulder width by 1.0 metre on both roadsides. The estimated crash reduction after applying these treatments was 36%. It should be pointed out that the latter treatment did not significantly affect the total crash reduction. Thus, the application of this treatment can be related to the available budget. The expected crash cost reduction associated with the treatments was approximately $AUD 1.2 million.
- Four treatments were identified for segment S_SE9 located on Ruthven Street between South Street and Long Street (see Figure 6.15). They were: reducing posted speed limit from 60 to 50 km/hr, adding a median island, increasing shoulder width by 1.5 m on both roadsides, and adding edge line on both directions. The estimated crash reduction after applying the suggested treatments was 48%. The latter two treatments did not significantly affect the total crash reduction, so they will be optional based on the available budget. The expected crash cost reduction associated with the treatments was approximately $AUD 1.6 million.
- Three treatments were identified for segment S_SE5 located on Alderley Street between Ramsay Street and Geddes Street (see Figure 6.16). They were: reducing posted speed limit from 60 to 50 km/hr, adding edge lines on both directions, and increasing shoulder width by 1.0 metre on both roadsides. The estimated crash reduction after applying these treatments was 36%. The expected crash cost reduction associated with the treatments was approximately $AUD 1.0 million.
- Two treatments were identified for segment S_SW8 on Anzac Avenue between South Street and Stephen Street (see Figure 6.17). They were: reducing posted speed limit from 60 to 50 km/hr and adding a median island. The estimated crash reduction after applying these treatments was 43%. The expected crash cost reduction associated with the treatments was approximately $AUD 1.5 million.
- Four treatments were identified for segment S_SW19 located on Anzac Avenue between Ball Street and Parker Street (see Figure 6.18). They were: adding one lane on each direction, reducing posted speed limit from 60 to 50 km/hr, adding a median

island, and increasing shoulder width by 0.5 m on both roadsides. The estimated crash reduction after applying the suggested treatments was 75%. The most effective treatment was adding one lane (i.e., crash reduction was 66%). In contrast, the latter treatment can be ignored as it did not affect total crash reduction. The expected crash cost reduction associated with the identified treatments was approximately $AUD 2.6 million.

- Three treatments were identified for segment S_NW1 located on James Street between Ruthven Street and Helen Street (see Figure 6.19). They were: reducing posted speed limit from 60 to 50 km/hr, increasing lane width by 0.6 m for 4-lane, and increasing shoulder width by 1.5 m on both roadsides. The estimated crash reduction after applying the suggested treatments was 43%. Moreover, the expected crash cost reduction associated with the suggested treatments was approximately $AUD 1.4 million.

6.10 Summary

This chapter identified the most appropriate road safety treatments for hazardous road segments in Toowoomba city. The impact of the identified treatments on the traffic operations using simulation modelling was also investigated. The safety performance models were developed using a generalised linear model with Negative Binomial (NB) distribution to estimate the model parameters. Four safety models were developed to predict segment related crashes. Using the safety models, the Empirical Bayes (EB) approach was employed to identify the most hazardous road segments. This approach increases the accuracy of safety estimation by calculating the weighted combination of the observed with the predicted crash numbers to overcome the phenomenon of regression to the mean. The study has identified segment S_NW22 (i.e., located on Tor Street between Hursley road and Gatfield Street) as the most hazardous segment in the study area with the highest PSI value of 3.027. The segment S_NE4 (i.e., located on James Street between Ruthven Street and Fitzgibbon Street) was identified as the safest segment in the study area with a PSI value of -1.795.

Crash modification functions (CMFunctions) were derived from safety models to estimate the values of crash modification factor for different types of treatments. More specifically, the CMFs can be used to identify the effects of suggested treatments on road safety. The results of the CMFs showed that overall adding one lane is the most effective way to reduce segment related crashes with a crash reduction of 41%. After estimating the CMFs for individual treatments, the average of four different techniques were employed to estimate the effects of multiple treatments on road safety for the top ten hazardous segments. The highest crash reduction factor (i.e., CRF = 75%) for multiple treatments was obtained at segment S_SW19 (i.e., located on Anzac Avenue between Ball Street and Parker Street). The treatments for this segment included: adding one lane in each direction, reducing posted speed from 60 to 50 km/hr, adding a median island, and increasing shoulder width on both side by 0.5 m.

A traffic simulation model using VISSIM software was employed to investigate the effects of suggested treatments on the traffic operation conditions. Two performance measures were adopted in this study: travel time and average speed at road segments. The traffic conditions were simulated before and after implementation of suggested treatments. The results revealed that the expected values of travel time and average speed for all treated segments would be slightly higher due to a reduction in the posted speed from 60 to 50 km/hr. For instance, the values of travel time and average speed for road segment S_NE8 (i.e., located on Margaret Street between Clifford Street and West Street) before treatments were 19.50 seconds and 57.67 km/hr, respectively. The values of travel time and average speed for the same segment after treatments were 23.02 seconds and 48.83 km/hr, respectively.

The study offered the safest treatment options to improve the safety of road segments and considered the crash costs reduction associated with each safety treatment option. In particular, the segment related crashes are expected to decrease after implementation of the safety treatments. Therefore, the crash costs were estimated before and after treatment implementation using CRFs to determine the saved costs. These costs were also used to calculate Present values (PVs) based on a 10-year treatment life. The results showed that between $1 and $3 million will be saved after treatment implementation. Ideally, the benefit-cost ratios can be accurately calculated by knowing the costs of the identifying treatments. A sample of benefit-cost ratios was estimated based on data from Toowoomba Regional Council to provide some comparative ratios to illustrate how such information may be utilised by road authorities, Councils, and practitioners to better address issues within their road networks.

Chapter 7

Summary, Conclusions, and Future Research

7.1 Summary and Conclusions

Road authorities and road safety experts are interested in estimating the expected outcomes originating from multiple road safety treatments. Information emanating from proposed treatments enables planners to make a comparison between the expected savings from crash reductions and associated treatment costs. Importantly the information also allows prioritisation of safety improvement projects, which will provide wider benefits to the community. This research study outlines how road safety models can be developed and used to identify hazardous road locations (HRLs). It also demonstrates methodologies of estimating individual and combined crash modification factors for various treatment plans for HRLs. Moreover, by using traffic simulation models, the impact of the proposed safety treatments on the current traffic operation conditions can be investigated. Lastly, the crash cost reductions associated with safety improvement plans can be estimated to help practitioners in identifying the treatment plans with high investment return.

Initially an extensive review of the international research literature regarding crash prediction studies was carried out to identify the appropriate modelling techniques and statistical methods that could be used in the modelling stage. The generalised linear model (GLM) with negative binomial (NB) error structure using log link function was adopted as the research dataset showed over-dispersion. Once the model form and analysis technique had been defined the crash history, traffic volume, and geometric attributes were collected for the case study area, from 106 intersections, 59 roundabouts, and 89 roadway segments. The developed models were evaluated using following goodness-of-fit measures: Akaike Information Criterion (AIC), Bayesian Information Criterion (BIC), Pearson Chi-square (x^2), residual values, and Cumulative Residuals (CUREs) plot. The models' ability to predict road crashes for additional years was tested using the Mean Squared Prediction Error (MSPE), Mean Absolute Deviation (MAD), Mean Squared Error (MSE), and Freeman-Tukey R-Squared coefficient (R^2_{FT}).

The fitted CPMs showed several statistically significant explanatory variables (P<0.10) affecting safety at road intersections, roundabouts, and roadway segments, as summarised in Table 7.1.

Table 7.1 Significant explanatory variables affecting safety

Intersections	**Roundabouts**	**Roadway segments**
Number of intersection legs	Traffic volume on major and minor approaches	Segment length
Traffic volume on minor approaches	Number of entry and exit lanes on major approaches	Traffic volume
Number of through lanes entering on major and minor approaches	Entry and exit width on major approaches	Number of lanes per direction
Number of through lanes exiting on major and minor approaches	Entry width on minor approaches	Lane width
Number of left turn lanes on major and minor approaches	Entry and exit path radius on major and minor approaches	Presence of a median island
Number of right turn lanes on major approaches	Weaving length	-
Number of slip lanes on minor approaches	Weaving width	-
Presence of a median island on major and minor approaches	Central island diameter	-
Speed limit	Speed limit	-

An accurate identification of HRLs prevents wasted resources that may result if such locations are identified with less precision. The HRLs in the study area were identified using the Empirical Bayes (EB) approach which increases the accuracy of safety estimation by accounting for the regression-to-the-mean bias usually associated with road crash data. Using this approach, the expected crash frequencies were estimated by calculating the weighted combination of the observed and the predicted crash frequencies. The HRLs were ranked in descending order based on the potential for safety improvement (PSI), which is calculated as the difference between the expected and predicted crashes. The study identified 44 intersections, 19 roundabouts, and 38 roadway segments that had potential for safety improvement. The most hazardous intersection, needing safety improvement, was I_NW9 at Bridge Street and Tor Street with an average of 6.67 observed severe crashes per year and PSI value of 3.02. The most hazardous roundabout was R_NW7, located at Anzac Avenue, Hursley Road, and Holberton Street with an average of 4.0 severe crashes per year with a PSI value of 2.87. The most hazardous roadway segment was S_NW22, located on Tor Street between Hursley Road and Gatfield Street with an average of 2.0 severe crashes per year with a PSI value of 3.03.

The crash modification factor (CMF) is a value representing the change in road safety after modifying the geometric design or operation of the facility. Most previous studies have ignored the variation of CMF values among treated sites by estimating CMF as fixed or single value. This study developed a crash modification function (CMFunction) formulae to estimate the variation in the values of CMF with different sites characteristics, rather than using a single value. The CMF values were estimated for different treatment types at the top 10 HRLs using CMFunctions. The geometric features of HRLs and recent operational conditions were incorporated to determine the possible treatments for each location. The most effective single treatment for top 10 hazardous intersections, roundabouts, and roadway segments was as follows:

- The most effective single treatment for 6 intersections (I_NE5, I_SE12, I_NW15, I_NE6, I_NW6, and I_NE4) was adding a raised median island on major approaches with an expected crash reduction of 42%. For the remaining 4 intersections (I_NW9, I_SW19, I_NW5, and I_NE28), the most effective single treatment was changing the posted speed limit on major approaches from 60 to 50 km/hr, with an expected crash reduction of 32%.
- The most effective single treatment for 4 roundabouts (R_NW7, R_SW3, R_SE6, and R_SE13) was reducing entry width on minor approaches by 0.6 m, with an expected crash reduction of 47%. The most effective treatment for 2 roundabouts (R_SE11 and R_NE4) was reducing entry path radius on minor approaches by 10 m, with an expected crash reduction of 50%. The effective treatment for 4 roundabouts (R_SW2, R_NE1, R_NE7, and R_SE2) was reducing entry width on one major approach by 1.2 m, reducing weaving width by 1.2 m, increasing exit path radius on minor approaches by 10 m, and increasing exit path radius on one major approach by 20 m, respectively. The expected crash reduction after applying these treatments was 62%, 31%, 38%, and 33%, respectively.
- The most effective single treatment for 9 roadway segments (S_NW22, S_NE8, S_NW21, S_SW4, S_SW16, S_SE9, S_SE5, S_SW8, and S_NW1) was reducing the posted speed limit from 60 to 50 km/hr, with an expected crash reduction of 33%, whereas, for the other segment (S_SW19), adding one lane for each direction was most effective.

The Highway Safety Manual (HSM), Part D, suggests that CMF values should be multiplied to estimate the combined safety impacts of multiple treatments. This suggestion is based on the assumption that the road safety effect of each treatment is independent. Therefore, the HSM warns that the multiplication of the CMF values may result in over-estimating or under-estimating the combined effects of multiple treatments. In order to more reliably estimate a combined value of CMF, an adjustment approach (i.e., average values) of the existing techniques was used as an effective and simple approach. The combined values of CMF were estimated using four existing techniques (HSM, Turner, systematic reduction of subsequent CMFs, and applying only the most effective CMF technique). It was found that there were variations in the estimation of combined CMFs using the applied techniques. The results demonstrated that multiple treatments have higher safety effects than a single treatment. The highest expected crash reduction (i.e., CRF = 66%) for multiple treatments was obtained at intersection I_NE4 (between James Street and Neil Street) after applying seven proposed treatments. For roundabouts, the highest expected crash reduction (i.e., CRF = 75%) for multiple treatments was obtained at roundabout R_SW2 (between Glenvale Street and McDougall Street) after applying seven proposed treatments. The same expected crash reduction (i.e., CRF = 75%) was obtained at segment S_SW19 (located on Anzac Avenue between Ball Street and Parker Street) after applying four proposed treatments.

In previous researches, the focus was on developing CMFs and applying these factors to identify the appropriate treatments on the basis of the expected crash reduction achieved. In this research, in order to investigate the effect of proposed safety treatments on traffic conditions, the microscopic traffic simulation software PTV VISSIM 9.0 has been utilised. The top 10 hazardous intersections and roundabouts have been evaluated under different scenarios in terms of level of service (LOS) and traffic delay performance measures, whereas roadway segments have been evaluated in terms of travel time and average speed performance measures. The simulation results based on the average of 10 runs with random seed values showed that there was no significant impact on traffic conditions after the implementation of proposed treatments. It was found that two intersections (i.e., I_SE12 and I_NE28) had a slight negative impact on the delay time, which may have been due to installing signals at these non-signalised intersections. For roadway segments, the travel time for treated segments increased by 2-10 seconds and was slightly higher than for untreated segments, due to the effect of reducing the posted speed limit from 60 to 50 km/hr as one of the proposed treatments.

Quantifying the safety impacts of using CMFs supports the safety improvement process by providing the information required to make a comparison between the reduction in crash costs and the treatment costs to fulfil the greatest return on road safety investments. Therefore, CMFs have been used in the economic analysis to help identify the most beneficial treatments for safety improvements and allow prioritization of safety improvement projects. The crash costs were estimated before and after treatments implementation using single and combined CMFs to determine the saved costs. It is worth mentioning that the detailed expected treatment costs (i.e., construction and maintenance costs) associated with each proposed treatment type are not available, as the expected cost of treatments varied according to the particular location and annual maintenance cost. Regardless of treatment cost, the findings of this analysis provide an important first step in estimating the relative benefit-cost ratios associated with different safety treatments. Through extensive analysis efforts, the total discounted benefits have been estimated for all proposed treatments. The results showed that the expected total discounted benefits for the top 10 hazardous intersections after 10 years of treatments ranged between $2.2 and $8.2 million (AUD). Likewise, the total discounted benefits ranged between $0.6 and $6.5 million for roundabouts and between $1.0 and $3.0 million for roadway segments. The highest expected crash cost reduction would be likely at intersection I_NE5, roundabout R_NW7, and segment S_NW22 with $8.2, $6.5, and $3.0 million respectively after applying all proposed treatments for each one. Overall, better knowledge about the effectiveness of safety treatments will result in more accurate risk assessment and thus a more effective investment in road safety.

The original hypothesis of the research was that a better understanding of the main contributing factors to the road crashes could help to identify effective crash reduction measures at critical locations. The research has successfully demonstrated, through crash modelling, identifying HRLs, developing CMFs, traffic simulation, and estimating total benefits, that the better the understanding of the significant factors affecting crash occurrence, the greater the contribution can be in identifying the most appropriate safety treatments for HRLs.

7.2 Research Application

The CPMs used for this research were developed and validated using the datasets of observed crash history, traffic volume, and geometric attributes of the road network of Toowoomba City. The application of these models in safety investigations are applicable for regional cities with similar road characteristics. The models developed in the research can also be applied to regional cities with different crash frequency level and risk factors by recalibration of the models (Harwood et al. 2000; Cunto et al. 2014).

Three applications of the CPMs are described: predicting road crashes; identifying and ranking HRLs; and estimating the effect of single and combined CMFs. The cross-sectional method (regression approach) was used to estimate CMFs as functions for all treatments proposed at examined intersections, roundabouts, and roadway segments. It is worth mentioning that the cross-sectional method does not take into account the effects of factors that are not included in the analysis, i.e. external causal factors (Gross et al. 2010; Hauer 2013). However, this method was adopted in the analysis in preference to other methods (e.g., observational before-after studies) based on the availability of the data, as discussed earlier. The CMFs were estimated for various safety treatments in Toowoomba and the applicability of these treatments was discussed in detail in sections 4.9, 5.9, and 6.9. It should be noted that the CMFs in this research are only applicable to severe injury and fatal crashes. Thus, it is not appropriate to apply CMFs from this research to investigate the effect of a particular safety treatment on other crash types such as property damage.

The results concluded that the effect on road safety of treatments does not depend on the number of treatments that have been applied but rather depends on the quality and the suitability of these treatments relative to the treated site’s operating environment.

The research started by applying the most effective treatments gradually. It was observed that the greatest expected crash reduction was obtained after applying the first treatment. Most of the later treatments achieved only minor crash reduction. As a result, road authorities and practitioners would usually find that the most effective single treatment would be sufficient to achieve a meaningful crash reduction, although some secondary treatments may be cost effective to implement at the same time as the primary treatment is applied. For instance, reducing the entry lane width by 0.6 m is associated with a more significant crash reduction compared with increasing the exit lane width by 0.6 m on the same leg at a particular roundabout (see Table 5.25). Although the second treatment has not significantly affected safety to the first extent as the first one, it would be recommended to apply these treatments together to achieve cost-effectiveness (i.e., only by moving the median island from exit lane towards entry lane).

7.3 Future Research

While this research has achieved the proposed objectives, further research would be beneficial to extend its scope. The following areas are recommended for further research:

- Recalibrate the developed models using data from a number of regions (i.e., case studies) to verify the transferability of findings to other regions.
- Studying additional explanatory variables related to geometric features and traffic conditions should be included in the modelling process whenever possible. This would extend the scope for applying the findings from the current investigation. For example, the road intersections in the modelling process were analysed as a whole to investigate the effect of common risk factors (e.g., number of legs and type of traffic control). It would be useful to analyse intersections in different groups, such as three-legged intersections and four-legged intersections.
- Roadway segmentation is a primary step in the CPM calibration. Therefore, further research could investigate the effect of different segmentation methods on the performance of the developed CPMs at roadway segments, in terms of goodness-of-fit.
- It is important to estimate the safety effects (i.e., CMFs) based on various severity levels and crash types. From this it may be possible to identify the impact of various treatment types on crash type and severity.
- The VISSIM simulation package was employed to investigate the effect of suggested safety treatments on traffic conditions in terms of LOS, delay time, travel time, and average speed. Further research can be recommended to investigate the main limitations associated with VISSIM. Moreover, applying other simulation packages (e.g., CORSIM and HCS) and performance measures may be needed to confirm VISSIM results.

References

AASHTO 2010, *Highway Safety Manual*, 1st edn, American Association of State Highway and Transportation Officials, Washington, DC.

Abdel-Aty, M & Keller, J 2005, 'Exploring the overall and specific crash severity levels at signalized intersections', *Accident Analysis & Prevention*, vol. 37, no. 3, pp. 417-25.

Abdel-Aty, M & Haleem, K 2011, 'Analyzing angle crashes at unsignalized intersections using machine learning techniques', *Accident Analysis & Prevention*, vol. 43, no. 1, pp. 461-70.

Abdel-Aty, M, Lee, C, Park, J, Wang, J, Abuzwidah, M & Al-Arifi, S 2014, *Validation and Application of Highway Safety Manual (Part D) in Florida*, Final Report, University of Central Florida, Orlando, Florida.

Abdel-Aty, MA & Radwan, AE 2000, 'Modeling traffic accident occurrence and involvement', *Accident Analysis & Prevention*, vol. 32, no. 5, pp. 633-42.

Abdul Manan, MM, Jonsson, T & Várhelyi, A 2013, 'Development of a safety performance function for motorcycle accident fatalities on Malaysian primary roads', *Safety Science*, vol. 60, pp. 13-20.

ABS 2012, *History of Roads in Australia*, Australia Burua of Statistics (ABS), Canberra, Australia, viewed 20 August 2016, <http://www.abs.gov.au/ausstats/abs@.nsf/0/2e904c15091c39a5ca2569de0028b416?OpenDocument>.

ABS 2015, *Regional Population Growth, Australia*, Australia Burua of Statistics (ABS), Canberra, Australia, viewed 30 September 2016, <http://www.abs.gov.au/AUSSTATS/abs@.nsf/DetailsPage/3218.02014-15?OpenDocument>.

Ackaah, W & Salifu, M 2011, 'Crash prediction model for two-lane rural highways in the Ashanti region of Ghana', *IATSS Research*, vol. 35, no. 1, pp. 34-40.

Agresti, A 2002, *Categorical data analysis*, 2nd edn, Wiley series in probability and statistics, Gainesville, Florida.

Akaike, H 1974, 'A new look at the statistical model identification', *IEEE transactions on automatic control*, vol. 19, no. 6, pp. 716-23.

Anjana, S & Anjaneyulu, MVLR 2014, 'Development of safety performance measures for urban roundabouts in India', *Journal of Transportation Engineering*, vol. 141, no. 1, p. 8.

Arndt, O & Troutbeck, RJ 1998, 'Relationship between roundabout geometry and accident rates', *Transportation Research Board*, vol. 28, pp. 1-16.

Austroads 2005, 'Application of Design Principles and Guidelines', in *Road planning and design manual*, 1st edn, Department of Transport and Main Roads, QLD.

Austroads 2015, *Guide to road design part 4B: roundabouts*, third edn, Austroads, Sydney, NSW, Australia.

Bahar, G 2010, *Methodology for the Development and Inclusion of Crash Modification Factors in the First Edition of the Highway Safety Manual*, Transportation Research Circular E-C142, Transportation Research Board, Washington, DC.

Barrios, E, Ridgway, M & Choa, F 2001, 'The Best Simulation Tool For Bus Operations', in *Improving Transportation System Safety and Performance. 2001 ITE Spring Conference and Exhibit*, Monterey, California.

Basu, S & Saha, P 2017, 'Regression Models of Highway Traffic Crashes: A Review of Recent Research and Future Research Needs', *Procedia Engineering*, vol. 187, pp. 59-66.

Bauer, K & Harwood, D 1996, *Statistical Models of At-Grade Intersection Crashes*, US Department of Transportation and Federal Highway Administration publication No. FHWA-RD-96-125, Department of Transportation, U.S.

Bauer, K & Harwood, D 2013, 'Safety effects of horizontal curve and grade combinations on rural two-lane highways', *Transportation Research Record*, vol. 2398, pp. 37-49.

Bauer, K, Harwood, D, Hughes, W & Richard, K 2004, 'Safety effects of narrow lanes and shoulder-use lanes to increase capacity of urban freeways', *Transportation Research Record*, vol. 1897, pp. 71-80.

Bauer, KM & Harwood, DW 2000, *Statistical Models of At-Grade Intersection Accidents—Addendum*, FHWA-RD-99-094, Midwest Research Institute, Georgetown Pike.

Bissonette, JA & Cramer, PC 2008, *Evaluation of the use and effectiveness of wildlife crossings*, 615, Transportation Research Board, Washington DC.

BITRE 2009, *Road crash costs in Australia 2006*, Bureau of Infrastructure, Transport and Regional Economicsb(BITRE), Canberra, Australia.

BITRE 2015, *Yearbook 2015: Australian Infrastructure Statistical Report*, Bureau of Infrastructure, Transport and Regional Economics (BITRE) Canberra ACT. viewed 10 April 2016, <https://bitre.gov.au/publications/2015/files/BITRE_yearbook_2015_full_report.pdf>.

BITRE 2017, *Road trauma Australia 2016 statistical summary*, Bureau of Infrastructure, Transport and Regional Economics (BITRE), Canberra ACT. viewed 10 January 2018, <https://bitre.gov.au/publications/ongoing/files/Road_Trauma_Australia_2016_Web.pdf>.

Bonneson, JA & McCoy, PT 1993, 'Estimation of safety at two-way stop-controlled intersections on rural highways', *Transportation Research Record*, vol. 1401, pp. 83-9.

Bonneson, JA & Lord, D 2005, *Role and application of accident modification factors in the highway design process*, FHW A/TX-05/0-4703-2, Texas Transportation Institute, Texas.

Bonneson, JA & Pratt, MP 2009, *Roadway Safety Design Workbook*, FHWA/TX-09/0-4703-P2, Texas Transportation Institute, Texas.

Cafiso, S, D'Agostino, C & Persaud, B 2018, 'Investigating the influence of segmentation in estimating safety performance functions for roadway sections', *Journal of traffic and transportation engineering (English edition)*, pp. 1-8.

Cafiso, S, Di Graziano, A, Di Silvestro, G, La Cava, G & Persaud, B 2010, 'Development of comprehensive accident models for two-lane rural highways using exposure, geometry, consistency and context variables', *Accident Analysis & Prevention*, vol. 42, no. 4, pp. 1072-9.

Caliendo, C, Guida, M & Parisi, A 2007, 'A crash-prediction model for multilane roads', *Accident Analysis & Prevention*, vol. 39, no. 4, pp. 657-70.

Chen, H, Cao, L & Logan, DB 2012, 'Analysis of risk factors affecting the severity of intersection crashes by logistic regression', *Traffic injury prevention*, vol. 13, no. 3, pp. 300-7.

Cheng, W & Washington, SP 2005, 'Experimental evaluation of hotspot identification methods', *Accident Analysis & Prevention*, vol. 37, no. 5, pp. 870-81.

Chin, HC & Quddus, MA 2003, 'Applying the random effect negative binomial model to examine traffic accident occurrence at signalized intersections', *Accident Analysis & Prevention*, vol. 35, no. 2, pp. 253-9.

Choa, F, Milam, RT & Stanek, D 2004, 'Corsim, paramics, and vissim: What the manuals never told you', in *Ninth TRB Conference on the Application of Transportation Planning Methods*, Louisiana.

Choi, YH, Park, SH, Ko, H, Kim, KH & Yun, I 2018, 'Development of safety performance functions and crash modification factors for expressway ramps', *KSCE Journal of Civil Engineering*, vol. 22, no. 2, pp. 804-12.

Claros, B, Edara, P & Sun, C 2017, 'When driving on the left side is safe: safety of the diverging diamond interchange ramp terminals', *Accident Analysis & Prevention*, vol. 100, pp. 133-42.

Corben, BF & Wai, FC 1990, *Pro-active traffic engineering safety study, final report: Part 2-right-turn-against crashes at traffic signals*, Report No. 11, Monash University, Australia.

Couto, A & Ferreira, S 2011, 'A note on modeling road accident frequency: A flexible elasticity model', *Accident Analysis & Prevention*, vol. 43, no. 6, pp. 2104-11.

Cunto, F, Sobreira, L & Ferreira, S 2014, 'Assessing the transferability of the Highway Safety Manual predictive method for urban roads in Fortaleza City, Brazil', *Journal of Transportation Engineering*, vol. 141, no. 1, p. 8.

Da Costa, S, Qu, X & Parajuli, PM 2015, 'A crash severity-based black spot identification model', *Journal of Transportation Safety & Security*, vol. 7, no. 3, pp. 268-77.

Daniels, S, Brijs, T, Nuyts, E & Wets, G 2010, 'Explaining variation in safety performance of roundabouts', *Accident Analysis & Prevention*, vol. 42, no. 2, pp. 393-402.

Daniels, S, Brijs, T, Nuyts, E & Wets, G 2011, 'Extended prediction models for crashes at roundabouts', *Safety Science*, vol. 49, no. 2, pp. 198-207.

David, NA & Norman, J 1975, *Motor vehicle accidents in relation to geometric and traffic features of highway intersections*, FHWA-RD-76-129, Federal Highway Administration, Washington, DC.

De Brabander, B & Vereeck, L 2007, 'Safety effects of roundabouts in Flanders: signal type, speed limits and vulnerable road users', *Accident Analysis & Prevention*, vol. 39, no. 3, pp. 591-9.

DIRD 2016, *Road Safety*, Department of Infrastructure and Regional Development (DIRD), Australian Government. viewed 12 April 2016, <https://infrastructure.gov.au/roads/safety/index.aspx>.

Dissanayake, S & Roy, U 2014, 'Crash severity analysis of single vehicle run-off-road crashes', *Journal of Transportation Technologies*, vol. 4, pp. 1-10.

Dong, C, Shi, J, Huang, B, Chen, X & Ma, Z 2017, 'A study of factors affecting intersection crash frequencies using random-parameter multivariate zero-inflated models', *International journal of injury control and safety promotion*, vol. 24, no. 2, pp. 208-21.

El-Basyouny, K & Sayed, T 2010, 'Full Bayes Approach to Before-and-After Safety Evaluation with Matched Comparisons: Case Study of Stop-Sign In-Fill Program', *Transportation Research Record*, vol. 2148, pp. 1-8.

El-Basyouny, K & Sayed, T 2013, 'Safety performance functions using traffic conflicts', *Safety Science*, vol. 51, no. 1, pp. 160-4.

Elvik, R 2007, *State-of-the-art approaches to road accident black spot management and safety analysis of road networks*, Report 883, Institute of Transport Economics, Norway.

Elvik, R 2008a, 'Comparative analysis of techniques for identifying locations of hazardous roads', *Transportation Research Record*, vol. 2083, pp. 72-5.

Elvik, R 2008b, 'A survey of operational definitions of hazardous road locations in some European countries', *Accident Analysis & Prevention*, vol. 40, no. 6, pp. 1830-5.

Elvik, R 2009, 'Developing accident modification functions: exploratory study', *Transportation Research Record*, vol. 2103, pp. 18-24.

Elvik, R, Vaa, T, Hoye, A & Sorensen, M 2009, *The handbook of road safety measures*, 2nd edn, Emerald Group Publishing.

Elvik, R, Ulstein, H, Wifstad, K, Syrstad, RS, Seeberg, AR, Gulbrandsen, MU & Welde, M 2017, 'An Empirical Bayes before-after evaluation of road safety effects of a new motorway in Norway', *Accident Analysis & Prevention*, vol. 108, pp. 285-96.

Farag, SG & Hashim, IH 2017, 'Safety performance appraisal at roundabouts: Case study of Salalah City in Oman', *Journal of Transportation Safety & Security*, vol. 9, no. 1, pp. 67-82.

FDOT 2014, *Traffic Analysis Handbook*, Florida Department of Transportation (FDOT), Florida, viewed 4 September 2017, <http://www.fdot.gov/planning/systems/programs/sm/traffic/default.shtm>.

Freeman, MF & Tukey, JW 1950, 'Transformations related to the angular and the square root', *The Annals of Mathematical Statistics*, pp. 607-11.

Galgamuwa, U & Dissanayake, S 2018, 'Evaluation of the safety effectiveness of lane-departure countermeasures on two-lane undivided roadways using crash modification factors', *Journal of Transportation Safety & Security*, pp. 1-21.

Gan, A, Shen, J & Rodriguez, A 2005, *Update of Florida crash reduction factors and countermeasures to improve the development of district safety improvement projects*, Final Report, FDOT 99700-3596-119, Florida Department of Transportation, Tallahassee.

Garber, NJ & Ehrhart, AA 2000, *The effect of speed, flow, and geometric characteristics on crash rates for different types of Virginia highways*, VTRC 00-R15 Final Report, Virginia Transportation Research Council, Charlottesville, Virginia.

Gargoum, SA & El-Basyouny, K 2016, 'Exploring the association between speed and safety: A path analysis approach', *Accident Analysis & Prevention*, vol. 93, pp. 32-40.

Ghadi, M & Török, Á 2017, 'Comparison Different Black Spot Identification Methods', *Transportation Research Procedia*, vol. 27, pp. 1105-12.

Gitelman, V, Doveh, E & Bekhor, S 2017, 'The relationship between free-flow travel speeds, infrastructure characteristics and accidents, on single-carriageway roads', *Transportation Research Procedia*, vol. 25, pp. 2026-43.

Golob, TF, Recker, WW & Alvarez, VM 2004, 'Safety aspects of freeway weaving sections', *Transportation Research Part A: Policy and Practice*, vol. 38, no. 1, pp. 35-51.

Gomes, SV, Geedipally, SR & Lord, D 2012, 'Estimating the safety performance of urban intersections in Lisbon, Portugal', *Safety Science*, vol. 50, no. 9, pp. 1732-9.

Greibe, P 2003, 'Accident prediction models for urban roads', *Accident Analysis & Prevention*, vol. 35, no. 2, pp. 273-85.

Gross, F 2006, 'A Dissertation in Civil Engineering: Alternative Methods for Estimating Safety Effectiveness on Rural, Two-Lane Highways: Case–control and Cohort Methods', Ph. D. Dissertation thesis, The Pennsylvania State University.

Gross, F & Hamidi, A 2011, *Investigation of Existing and Alternative Methods for Combining Multiple CMFs*, T-06-013, Highway Safety Improvement Program Technical Support, FHWA, U.S.

Gross, F, Persaud, B & Lyon, C 2010, *A guide to developing quality crash modification factors*, FHWA-SA-10-032, U.S. Department of Transportation, Washington DC.

Hadi, MA, Aruldhas, J, Chow, L-F & Wattleworth, JA 1995, 'Estimating safety effects of cross-section design for various highway types using negative binomial regression', *Transportation Research Record*, vol. 1500, pp. 169-77.

Haleem, K, Abdel-Aty, M & Mackie, K 2010, 'Using a reliability process to reduce uncertainty in predicting crashes at unsignalized intersections', *Accident Analysis & Prevention*, vol. 42, no. 2, pp. 654-66.

Hamidi, A, Fontaine, MD & Demetsky, MJ 2010, *A Planning-Level Methodology for Identifying High-Crash Sections of Virginia's Primary System*, FHWA/VTRC 11-R4, Virginia Transportation Research Council, Charlottesville, VA.

Hanley, K, Gibby, A & Ferrara, T 2000, 'Analysis of accident-reduction factors on California state highways', *Transportation Research Record*, vol. 1717, pp. 37-45.

Hanna, JT, Flynn, TE & Tyler, WL 1976, 'Characteristics of intersection accidents in rural municipalities', *Transportation Research Record*, vol. 601, pp. 79-82.

Haque, MM, Chin, HC & Huang, H 2010, 'Applying Bayesian hierarchical models to examine motorcycle crashes at signalized intersections', *Accident Analysis & Prevention*, vol. 42, no. 1, pp. 203-12.

Harkey, DL, Council, F, Srinivasan, R, Lyon, C, Persaud, B, Eccles, K, Lefler, N, Gross, F, Baek, J & Hauer, E 2008, *Accident modification factors for traffic engineering and ITS* *improvements*, National Cooperative Highway Research Program (NCHRP) Report 617, Transportation Research Board, Washington, DC.

Harwood, D, Bauer, K, Potts, I, Torbic, D, Richard, K, Rabbani, E, Hauer, E, Elefteriadou, L & Griffith, M 2003, 'Safety effectiveness of intersection left-and right-turn lanes', *Transportation Research Record*, vol. 1840, no. 131-139.

Harwood, DW, Pietrucha, MT & Fitzpatrick, K 1995, *Median intersection design*, National Cooperative Highway Research Program, NCHRP Report 375, Transportation Research Board, Washington, DC.

Harwood, DW, Council, F, Hauer, E, Hughes, W & Vogt, A 2000, *Prediction of the expected safety performance of rural two-lane highways*, FHWA-RD-99-207, Midwest Research Institute, Virginia.

Hauer, E 1988, 'The safety of older persons at intersections', *Special Report 218: Transportation in an Aging Society: Improving Mobility and Safety for Older Persons*, vol. 2, pp. 194-252.

Hauer, E 1995, 'On exposure and accident rate', *Traffic engineering & control*, vol. 36, no. 3, pp. 134-8.

Hauer, E 1997, *Observational before/after studies in road safety: Estimating the effect of highway and traffic engineering measures on road safety*, Emerald Publishing Limited, London, United Kingdom.

Hauer, E 2013, *Even perfect regressions may not tell the effect of interventions*, In: Transportation Research Board 92nd Annual Meeting, Washington DC.

Hauer, E & Persaud, B 1987, 'How to estimate the safety of rail-highway grade crossings and the safety effects of warning devices', *Transportation Research Record*, vol. 1114, pp. 141-51.

Hauer, E & Bamfo, J 1997, 'Two tools for finding what function links the dependent variable to the explanatory variables', in *proceedings of the ICTCT 1997 Conference, Lund, Sweden*.

Hauer, E & Bonneson, J 2006, *An empirical examination of the relationship between speed and road accidents based on data by Elvik, Christensen and Amundsen*, NCHRP 17-25 Final Report, Transportation Research Board, Washington, DC.

Hauer, E, Council, F & Mohammedshah, Y 2004, 'Safety models for urban four-lane undivided road segments', *Transportation Research Record*, vol. 1897, pp. 96-105.

Hauer, E, Harwood, D, Council, F & Griffith, M 2002, 'Estimating safety by the empirical Bayes method: a tutorial', *Transportation Research Record*, vol. 1784, pp. 126-31.

Hauer, E, Kononov, J, Allery, B & Griffith, M 2002, 'Screening the road network for sites with promise', *Transportation Research Record*, vol. 1784, pp. 27-32.

Heng, W & Perugu, HC 2009, 'Oversaturation inherence and traffic diversion effect at urban intersections through simulation', *Journal of Transportation Systems Engineering and Information Technology*, vol. 9, no. 4, pp. 72-82.

Hu, M-C, Pavlicova, M & Nunes, EV 2011, 'Zero-inflated and hurdle models of count data with extra zeros: examples from an HIV-risk reduction intervention trial', *The American journal of drug and alcohol abuse*, vol. 37, no. 5, pp. 367-75.

IBM Corp 2015, *IBM SPSS Statistics for Windows*, Version 23, NY: IBM Corp, Armonk.

Isebrands, H & Hallmark, S 2012, 'Statistical analysis and development of crash prediction model for roundabouts on high-speed rural roadways', *Transportation Research Record*, vol. 2312, pp. 3-13.

Ismail, MA & Abdelmageed, S 2010, 'Cost of road traffic accidents in Egypt', *World Academy of Science, Engineering & Technology*, vol. 66, pp. 1322-8.

Jacobs, G & Aeron-Thomas, A 2000, *A review of global road accident fatalities*, Transport research laboratory, Crowthorne, England.

Johansson, R 2009, 'Vision Zero–Implementing a policy for traffic safety', *Safety Science*, vol. 47, no. 6, pp. 826-31.

Kamla, J, Parry, T & Dawson, A 2016, 'Roundabout accident prediction model: random-parameter negative binomial approach', *Transportation Research Record*, vol. 2585, pp. 11-9.

Khan, M, Abdel-Rahim, A & Williams, CJ 2015, 'Potential crash reduction benefits of shoulder rumble strips in two-lane rural highways', *Accident Analysis & Prevention*, vol. 75, pp. 35-42.

Kibar, FT, Celik, F & Aytac, BP 2018, 'Statistical analysis of truck accidents for divided multilane interurban roads in Turkey', *KSCE Journal of Civil Engineering*, vol. 22, no. 5, pp. 1927-36.

Kim, D-G, Lee, Y, Washington, S & Choi, K 2007, 'Modeling crash outcome probabilities at rural intersections: Application of hierarchical binomial logistic models', *Accident Analysis & Prevention*, vol. 39, no. 1, pp. 125-34.

Kim, S & Choi, J 2013, 'Safety analysis of roundabout designs based on geometric and speed characteristics', *KSCE Journal of Civil Engineering*, vol. 17, no. 6, pp. 1446-54.

Kim, SH, Chung, SB, Song, KH & Chon, KS 2005, 'Development of an Accident Prediction Model using GLIM (Generalized Log-linear Model) and EB method: A case of Seoul', *Journal of the Eastern Asia society for Transportation studies*, vol. 6, pp. 3669-82.

Kloeden, C, Woolley, J & McLean, J 2007, 'A follow-up evaluation of the 50km/h default urban speed limit in South Australia', in *Australasian Road Safety Research, Education and Policing Conference,* Melbourne, Australia.

Ko, M, Geedipally, S & Walden, T 2013, 'Effectiveness and site selection criteria for red light camera systems', *Transportation Research Record*, vol. 2327, pp. 53-60.

Kononov, J, Bailey, B & Allery, B 2008, 'Relationships between safety and both congestion and number of lanes on urban freeways', *Transportation Research Record*, vol. 2083, pp. 26-39.

Kumara, S & Chin, HC 2003, 'Modeling accident occurrence at signalized tee intersections with special emphasis on excess zeros', *Traffic injury prevention*, vol. 4, no. 1, pp. 53-7.

Kumara, S & Chin, H 2005, 'Application of Poisson underreporting model to examine crash frequencies at signalized three-legged intersections', *Transportation Research Record*, vol. 1908, pp. 46-50.

Kutner, MH, Nachtsheim, CJ, Neter, J & Li, W 2005, *Applied linear statistical models*, 5th edn, McGraw-Hill Irwin, New York.

Lan, B, Persaud, B, Lyon, C & Bhim, R 2009, 'Validation of a full Bayes methodology for observational before–after road safety studies and application to evaluation of rural signal conversions', *Accident Analysis & Prevention*, vol. 41, no. 3, pp. 574-80.

Lee, C, Abdel-Aty, M, Park, J & Wang, J-H 2015, 'Development of crash modification factors for changing lane width on roadway segments using generalized nonlinear models', *Accident Analysis & Prevention*, vol. 76, pp. 83-91.

Lee, J & Mannering, FL 1999, *Analysis of roadside accident frequency and severity and roadside safety management*, WA-RD 475.1, Washington State Department of Transportation, Olympia, Washington.

Lee, J & Mannering, F 2002, 'Impact of roadside features on the frequency and severity of run-off-roadway accidents: an empirical analysis', *Accident Analysis & Prevention*, vol. 34, no. 2, pp. 149-61.

Leng, J, Zhang, Y & Sun, M 2008, 'VISSIM-Based Simulation Approach to Evaluation of Design and Operational Performance of U-turn at Intersection in China', in *International Workshop on Modelling, Simulation and Optimization*, pp. 309-12.

Leong, H 1973, 'Relationship between accidents and traffic volumes at urban intersections', *Australian road research*, vol. 5, no. 3, pp. 72-90.

Li, X, Lord, D & Zhang, Y 2010, 'Development of accident modification factors for rural frontage road segments in Texas using generalized additive models', *Journal of Transportation Engineering*, vol. 137, no. 1, pp. 74-83.

Litchfield, F 2017, *The cost of road crashes in Australia 2016: An overview of safety strategies* Australian National University, Canberra, Australia.

Litman, T 2009, *Transportation cost and benefit analysis: Techniques, Estimates and Implications*, Victoria Transport Policy Institute, Victoria, Canada. viewed 2 June 2017, <http://www.vtpi.org/tca>.

Liu, P, Huang, J, Wang, W & Xu, C 2011, 'Effects of transverse rumble strips on safety of pedestrian crosswalks on rural roads in China', *Accident Analysis & Prevention*, vol. 43, no. 6, pp. 1947-54.

Lord, D & Bonneson, J 2006, 'Role and application of accident modification factors within highway design process', *Transportation Research Record*, vol. 1961, pp. 65-73.

Lord, D & Bonneson, J 2007, 'Development of accident modification factors for rural frontage road segments in Texas', *Transportation Research Record*, vol. 2023, pp. 20-7.

Lord, D & Mannering, F 2010, 'The statistical analysis of crash-frequency data: a review and assessment of methodological alternatives', *Transportation Research Part A: Policy and Practice*, vol. 44, no. 5, pp. 291-305.

Lord, D, Manar, A & Vizioli, A 2005, 'Modeling crash-flow-density and crash-flow-V/C ratio relationships for rural and urban freeway segments', *Accident Analysis & Prevention*, vol. 37, no. 1, pp. 185-99.

Lord, D, Washington, SP & Ivan, JN 2005, 'Poisson, Poisson-gamma and zero-inflated regression models of motor vehicle crashes: balancing statistical fit and theory', *Accident Analysis & Prevention*, vol. 37, no. 1, pp. 35-46.

Lord, D, Washington, S & Ivan, JN 2007, 'Further notes on the application of zero-inflated models in highway safety', *Accident Analysis & Prevention*, vol. 39, no. 1, pp. 53-7.

Lu, J 2013, 'Development of safety performance functions for SafetyAnalyst applications in Florida', Doctoral Dissertation thesis thesis, Florida international university, Miami, Florida.

Lyon, C, Haq, A, Persaud, B & Kodama, S 2005, 'Safety performance functions for signalized intersections in large urban areas: Development and application to evaluation of left-turn priority treatment', *Transportation Research Record*, vol. 1908, pp. 165-71.

Mahmud, S, Ferreira, L, Hoque, MS & Tavassoli, A 2016, 'Using micro-simulation as a surrogate safety evaluation approach: a synthesis for potential application in non-lane based heterogeneous traffic environments', in *Australasian Transport Research Forum (ATRF) 2016 Proceedings*, Victoria, Australia.

Maina, E 2009, 'Developing crash modification factors for operational parameters on urban freeway', New Jersey Institute of Technology, Newark, NJ.

Mandavilli, S, Rys, MJ & Russell, ER 2008, 'Environmental impact of modern roundabouts', *International Journal of Industrial Ergonomics*, vol. 38, no. 2, pp. 135-42.

Mandavilli, S, McCartt, AT & Retting, RA 2009, 'Crash patterns and potential engineering countermeasures at Maryland roundabouts', *Traffic injury prevention*, vol. 10, no. 1, pp. 44-50.

Mbatta, G 2011, 'Developing crash modification factors for urban highway with substandard wide curb lane', Doctoral Dissertation thesis, The Florida State University, Florida.

Mehta, G & Lou, Y 2013, 'Safety performance function calibration and development for the State of Alabama: Two-lane two-way rural roads and four-lane divided highways', in *92nd Transportation Research Board Annual Meeting*, Washington DC.

Meuleners, LB, Hendrie, D, Lee, AH & Legge, M 2008, 'Effectiveness of the black spot programs in Western Australia', *Accident Analysis & Prevention*, vol. 40, no. 3, pp. 1211-6.

Miaou, S-P 1994, 'The relationship between truck accidents and geometric design of road sections: Poisson versus negative binomial regressions', *Accident Analysis & Prevention*, vol. 26, no. 4, pp. 471-82.

Miaou, S-P 1996, *Measuring the goodness-of-fit of accident prediction models*, FHWA-RD-96-040, Federal Highway Administration, Washington, DC.

Miaou, S-P & Lum, H 1993, 'Modeling vehicle accidents and highway geometric design relationships', *Accident Analysis & Prevention*, vol. 25, no. 6, pp. 689-709.

Milton, J & Mannering, F 1998, 'The relationship among highway geometrics, traffic-related elements and motor-vehicle accident frequencies', *Transportation*, vol. 25, no. 4, pp. 395-413.

Miranda-Moreno, L, Fu, L, Saccomanno, F & Labbe, A 2005, 'Alternative risk models for ranking locations for safety improvement', *Transportation Research Record*, vol. 1908, pp. 1-8.

Moghaddam, AM, Tabibi, Z, Sadeghi, A, Ayati, E & Ravandi, AG 2017, 'Screening out accident-prone Iranian drivers: are their at-fault accidents related to driving behavior?', *Transportation research part F: traffic psychology and behaviour*, vol. 46, pp. 451-61.

Montella, A 2010, 'A comparative analysis of hotspot identification methods', *Accident Analysis & Prevention*, vol. 42, no. 2, pp. 571-81.

Montella, A 2011, 'Identifying crash contributory factors at urban roundabouts and using association rules to explore their relationships to different crash types', *Accident Analysis & Prevention*, vol. 43, no. 4, pp. 1451-63.

Montella, A 2018, 'Roundabouts', in *Safe Mobility: Challenges, Methodology and Solutions*, Emerald Publishing Limited, pp. 147-74.

Montella, A, Colantuoni, L & Lamberti, R 2008, 'Crash prediction models for rural motorways', *Transportation Research Record*, vol. 2083, pp. 180-9.

Montella, A, Turner, S, Chiaradonna, S & Aldridge, D 2012, 'Proposals for improvement of the Italian roundabout geometric design standard', *Procedia-Social and Behavioral Sciences*, vol. 53, pp. 189-202.

Mountain, L, Fawaz, B & Sineng, L 1992, 'Assessment of changes in accident frequencies on link segments. A comparison of four methods', *Traffic engineering and control*, vol. 33, no. 7-8, pp. 429-31.

Mussa, R & Chimba, D 2006, 'Analysis of crashes occurring on Florida six-lane roadways', *Advances in Transportation Studies*, no. 8, pp. 43-56.

Mustakim, F & Fujita, M 2011, 'Development of accident predictive model for rural roadway', *World Academy of Science, Engineering and Technology*, vol. 58, no. 10, pp. 126-31.

Nambuusi, B, Brijs, T & Hermans, E 2008, *A review of accident prediction models for road intersections*, R A-MOW-2008-004, Infrastructuur en ruimte, Diepenbeek, Belgium.

Navidi, WC 2008, *Statistics for engineers and scientists*, 2 edn, vol. 1, McGraw-Hill New York, New York.

NCHRP 2008, *Crash Reduction Factors for Traffic Engineering and ITS Improvements*, Final Report Project 17-25, National Cooperative Highway Research Program (NCHRP), Washington DC.

Noland, RB & Oh, L 2004, 'The effect of infrastructure and demographic change on traffic-related fatalities and crashes: a case study of Illinois county-level data', *Accident Analysis & Prevention*, vol. 36, no. 4, pp. 525-32.

O'Brien, A 1976, 'Some Safety Considerations for Rural Intersections', in *Australian Road Research Board Conference Proc*, Perth, Australia.

Ogle, JH, Alluri, P & Sarasua, WA 2011, 'Model Minimum Uniform Crash Criteria and Minimum Inventory Roadway Elements: Role of Segmentation in Safety Analysis', in *Transportation Research Board 90th Annual Meeting*, Washington, DC.

Oh, J, Washington, S & Choi, K 2004, 'Development of accident prediction models for rural highway intersections', *Transportation Research Record*, vol. 1897, pp. 18-27.

Park, J 2015, 'Exploration and development of crash modification factors and functions for single and multiple treatments', University of Central Florida, Orlando, Florida.

Park, J & Abdel-Aty, M 2015, 'Development of adjustment functions to assess combined safety effects of multiple treatments on rural two-lane roadways', *Accident Analysis & Prevention*, vol. 75, pp. 310-9.

Park, J & Abdel-Aty, M 2016, 'Evaluation of safety effectiveness of multiple cross sectional features on urban arterials', *Accident Analysis & Prevention*, vol. 92, pp. 245-55.

Park, J & Abdel-Aty, MA 2017, 'Alternative Approach for Combining Multiple Crash Modification Factors Using Adjustment Function and Analytic Hierarchy Process', *Transportation Research Record*, vol. 2636, pp. 15-22.

Park, J, Abdel-Aty, M & Lee, C 2014, 'Exploration and comparison of crash modification factors for multiple treatments on rural multilane roadways', *Accident Analysis & Prevention*, vol. 70, pp. 167-77.

Park, J, Abdel-Aty, M, Wang, J-H & Lee, C 2015, 'Assessment of safety effects for widening urban roadways in developing crash modification functions using nonlinearizing link functions', *Accident Analysis & Prevention*, vol. 79, pp. 80-7.

Park, J, Abdel-Aty, M, Lee, J & Lee, C 2015, 'Developing crash modification functions to assess safety effects of adding bike lanes for urban arterials with different roadway and socio-economic characteristics', *Accident Analysis & Prevention*, vol. 74, pp. 179-91.

Park, JS, OH, JT, OH, SJ & KIM, YJ 2016, 'Analysis of Contributory Factors in Causing Crashes at Rural Unsignalized intersections Based on Statistical Modeling', *Journal of Korean Society of Transportation*, vol. 34, no. 2, pp. 123-34.

Pecchini, D, Mauro, R & Giuliani, F 2014, 'Model of potential crash rates of rural roundabouts with geometrical features', *Journal of Transportation Engineering*, vol. 140, no. 11, pp. 1-13.

Pendleton, O 1991, *Application of New Crash Analysis Methodologies: Volume 1-General Methodology*, Texas Transportation Institute. The Texas A&M University System, College Station.

Pernia, JC, Lu, JJ, Weng, M, Xie, X & Yu, Z 2002, *Development of models to quantify the impacts of signalization on intersection crashes*, University of South Florida, Florida.

Persaud, B & Lyon, C 2007, 'Empirical Bayes before–after safety studies: lessons learned from two decades of experience and future directions', *Accident Analysis & Prevention*, vol. 39, no. 3, pp. 546-55.

Persaud, B, Hauer, E & Lovell, J 1984, 'The safety effect of conversion to all-way stop control in Philadelphia', *University of Toronto, Department of Civil Engineering Publication*, no. 84, p. 14.

Persaud, B, Lyon, C & Nguyen, T 1999, 'Empirical Bayes procedure for ranking sites for safety investigation by potential for safety improvement', *Transportation Research Record*, vol. 1665, pp. 7-12.

Persaud, B, Retting, R, Garder, P & Lord, D 2001, 'Safety effect of roundabout conversions in the united states: Empirical bayes observational before-after study', *Transportation Research Record*, vol. 1751, pp. 1-8.

Persaud, B, Lan, B, Lyon, C & Bhim, R 2010, 'Comparison of empirical Bayes and full Bayes approaches for before–after road safety evaluations', *Accident Analysis & Prevention*, vol. 42, no. 1, pp. 38-43.

Persaud, B, Hauer, E, Retting, R, Vallurupalli, R & Mucsi, K 1997, 'Crash reductions related to traffic signal removal in Philadelphia', *Accident Analysis & Prevention*, vol. 29, no. 6, pp. 803-10.

Persaud, B, Council, F, Lyon, C, Eccles, K & Griffith, M 2005, 'Multijurisdictional safety evaluation of red light cameras', *Transportation Research Record*, vol. 1922, pp. 29-37.

Persaud, BN, Retting, RA & Lyon, CA 2004, 'Crash reduction following installation of centerline rumble strips on rural two-lane roads', *Accident Analysis & Prevention*, vol. 36, no. 6, pp. 1073-9.

Pitale, JT, Shankwitz, C, Preston, H & Barry, M 2009, *Benefit: Cost Analysis of In-vehicle Technologies and Infrastructure Modifications as a Means to Prevent Crashes Along Curves and Shoulders*, MN/RC 2009-39, Minnesota Department of Transportation, Minneapolis.

Polders, E, Daniels, S, Casters, W & Brijs, T 2015, 'Identifying crash patterns on roundabouts', *Traffic injury prevention*, vol. 16, no. 2, pp. 202-7.

PTV 2016, *PTV Vissim 9.0 User Manual*, Planning Transport Verkehr (PTV), Karlsruhe, Germany.

Qin, X, Ivan, JN & Ravishanker, N 2004, 'Selecting exposure measures in crash rate prediction for two-lane highway segments', *Accident Analysis & Prevention*, vol. 36, no. 2, pp. 183-91.

Queensland Government 2016, *Crash data from Queensland roads*, viewed 1 January 2017, < https://data.qld.gov.au/dataset/crash-data-from-queensland-roads>.

Ratanavaraha, V & Suangka, S 2014, 'Impacts of accident severity factors and loss values of crashes on expressways in Thailand', *IATSS Research*, vol. 37, no. 2, pp. 130-6.

Retting, R 2006, 'Enhancing Intersection Safety Through Roundabouts: A Proposed ITE Informational Report', in *ITE 2006 Technical Conference and Exhibit Compendium of Technical Papers*, San Antonio, Texas.

Robinson, BW, Rodegerdts, L, Scarborough, W, Kittelson, W, Troutbeck, R, Brilon, W, Bondzio, L, Courage, K, Kyte, M & Mason, J 2000, *Roundabouts: An informational guide*, FHWA-RD-00-067, Federal Highway Administration, McLean, Virginia.

Rodegerdts, L, Bansen, J, Tiesler, C, Knudsen, J, Myers, E, Johnsonm, M, Moule, M, Persaud, B, Lyon, C, Hallmark, S, Isebrands, H, Guichet, B & O'Brien, A 2010, *Roundabouts: An informational guide*, NCHRP Report 672, Transportation Research Board, Washington DC.

Sacchi, E & Sayed, T 2015, 'Investigating the accuracy of Bayesian techniques for before–after safety studies: The case of a “no treatment” evaluation', *Accident Analysis & Prevention*, vol. 78, pp. 138-45.

Sacchi, E, Bassani, M & Persaud, B 2011, 'Comparison of safety performance models for urban roundabouts in Italy and other countries', *Transportation Research Record*, vol. 2265, pp. 253-9.

Sacchi, E, Sayed, T & El-Basyouny, K 2014, 'Collision modification functions: Incorporating changes over time', *Accident Analysis & Prevention*, vol. 70, pp. 46-54.

Saccomanno, FF, Park, PY-J & Fu, L 2007, 'Estimating countermeasure effects for reducing collisions at highway–railway grade crossings', *Accident Analysis & Prevention*, vol. 39, no. 2, pp. 406-16.

Saccomanno, FF, Grossi, R, Greco, D & Mehmood, A 2001, 'Identifying black spots along highway SS107 in Southern Italy using two models', *Journal of Transportation Engineering*, vol. 127, no. 6, pp. 515-22.

Sayed, T & Rodriguez, F 1999, 'Accident prediction models for urban unsignalized intersections in British Columbia', *Transportation Research Record*, vol. 1665, pp. 93-9.

Schwarz, G 1978, 'Estimating the dimension of a model', *The annals of statistics*, vol. 6, no. 2, pp. 461-4.

Šenk, P & Ambros, J 2011, 'Estimation of Accident Frequency at Newly-built Roundabouts in the Czech Republic', *Transactions on Transport Sciences*, vol. 4, no. 4, pp. 199-206.

Šenk, P, Ambros, J, Pokorný, P & Striegler, R 2012, 'Use of Accident Prediction Models in Identifying Hazardous Road Locations', *Transactions on Transport Sciences*, vol. 5, no. 4, pp. 223-32.

Shadpour, E 2012, *Safety Effects of Roundabouts*, Laurier Centre for Economic Research & Policy Analysis, Wilfrid Laurier University.

Shahdah, U, Saccomanno, F & Persaud, B 2014, 'Integrated traffic conflict model for estimating crash modification factors', *Accident Analysis & Prevention*, vol. 71, pp. 228-35.

Shankar, V, Milton, J & Mannering, F 1997, 'Modeling accident frequencies as zero-altered probability processes: an empirical inquiry', *Accident Analysis & Prevention*, vol. 29, no. 6, pp. 829-37.

Shen, J & Gan, A 2003, 'Development of crash reduction factors: methods, problems, and research needs', *Transportation Research Record*, vol. 1840, pp. 50-6.

Shen, Q 2007, 'Development of safety performance functions for empirical Bayes estimation of crash reduction factors', Florida International University, Miami, Florida.

Sisiopiku, VP & Oh, H-U 2001, 'Evaluation of roundabout performance using SIDRA', *Journal of Transportation Engineering*, vol. 127, no. 2, pp. 143-50.

Sjölinder, K & Ek, H 2001, *Black Spot Manual*, General Directorate of Highways, SweRoad, Ankara.

Srinivasan, R & Carter, DL 2011, *Development of safety performance functions for North Carolina*, North Carolina Department of Transportation, Research and Analysis Group Chapel Hill, NC.

Strathman, JG, Dueker, KJ, Zhang, J & Williams, T 2001, *Analysis of design attributes and crashes on the Oregon highway system*, Oregon Department of Transportation, Portland.

Sun, X, Das, S, Fruge, N, Bertinot, RL & Magri, D 2013, 'Four-lane to five-lane urban roadway conversions for safety', *Journal of Transportation Safety & Security*, vol. 5, no. 2, pp. 106-17.

Tay, R & Rifaat, SM 2007, 'Factors contributing to the severity of intersection crashes', *Journal of Advanced Transportation*, vol. 41, no. 3, pp. 245-65.

Tegge, RA, Jo, J-H & Ouyang, Y 2010, *Development and application of safety performance functions for Illinois*, FHWA-ICT-10-066, Illinois Center for Transportation, Illinois.

Thomas, GB & Smith, DJ 2001, *Effectiveness of roadway safety improvements, Final report*, Center for Transportation Research and Education, Iowa State University.

Tian, Z, Urbanik, T, Engelbrecht, R & Balke, K 2002, 'Variations in capacity and delay estimates from microscopic traffic simulation models', *Transportation Research Record*, vol. 1802, pp. 23-31.

Transportation Research Board 2010, *Highway Capacity Manual*, vol. 3, National Research Council, Washington, DC.

Trueblood, M & Dale, J 2003, 'Simulating roundabouts with Vissim', in *Proceedings of the 2nd Urban Street Symposium*, Anaheim, California.

Turner, B 2011, *Estimating the Safety Benefits when Using Multiple Road Engineering Treatments*, Road Safety Risk Reporter, 11, Australian Road Research Board (ARRB), Austroads.

Turner, S, Roozenburg, AP & Smith, A 2009, *Roundabout crash prediction models*, NZ Transport Agency research report 386, Beca Infrastructure, New Zealand.

Turner, S, Singh, R & Nates, G 2012, *The next generation of rural road crash prediction models: final report*, NZ Transport Agency research report 509, New Zealand.

Usman, T, Fu, L & Miranda-Moreno, LF 2010, 'Quantifying safety benefit of winter road maintenance: Accident frequency modeling', *Accident Analysis & Prevention*, vol. 42, no. 6, pp. 1878-87.

Vayalamkuzhi, P & Amirthalingam, V 2016, 'Influence of geometric design characteristics on safety under heterogeneous traffic flow', *Journal of traffic and transportation engineering (English edition)*, vol. 3, no. 6, pp. 559-70.

Vogt, A 1999, *Crash models for rural intersections: Four-lane by two-lane stop-controlled and two-lane by two-lane signalized*, FHWA-RD.99-I28, Federal Highway Administration, McLean, Virginia.

Wang, C, Quddus, M & Ison, S 2013, 'A spatio-temporal analysis of the impact of congestion on traffic safety on major roads in the UK', *Transportmetrica A: Transport Science*, vol. 9, no. 2, pp. 124-48.

Wang, J-H & Abdel-Aty, MA 2014, 'Comparison of safety evaluation approaches for intersection signalization in Florida', in *Transportation Research Board 93rd Annual Meeting*, Washington, DC.

Wang, J-H, Abdel-Aty, M & Wang, L 2017, 'Examination of the reliability of the crash modification factors using empirical Bayes method with resampling technique', *Accident Analysis & Prevention*, vol. 104, pp. 96-105.

Wang, L, Abdel-Aty, M, Wang, X & Yu, R 2018, 'Analysis and comparison of safety models using average daily, average hourly, and microscopic traffic', *Accident Analysis & Prevention*, vol. 111, pp. 271-9.

Wang, Y & Nihan, NL 2001, 'Quantitative analysis on angle-accident risk at signalized intersections', in *World Transport Research, Selected Proceedings of the 9th World Conference on Transport Research*, Seoul.

Washington, SP, Karlaftis, MG & Mannering, F 2010, *Statistical and econometric methods for transportation data analysis*, 2nd edn, Chapman and Hall/CRC, Boca Raton, FL.

Washington, SP, Persaud, BN, Lyon, C & Oh, J 2005, *Validation of accident models for intersections*, FHWA-RD-03-037, Department of Transportation, Washington DC.

WHO 2013, *Global Status Report on Road Safety 2013*, World Health Organization (WHO), viewed 15 April 2016, <http://www.who.int/violence_injury_prevention/road_safety_status/2013/en/10 >.

WHO 2015, *Global Status Report on Road Safety 2015*, World Health Organization (WHO), viewed 5 July 2016, <http://www.who.int/violence_injury_prevention/road_safety_status/2015/en/>.

Wong, S, Sze, N-N & Li, Y-C 2007, 'Contributory factors to traffic crashes at signalized intersections in Hong Kong', *Accident Analysis & Prevention*, vol. 39, no. 6, pp. 1107-13.

Wood, JS, Donnell, ET & Porter, RJ 2015, 'Comparison of safety effect estimates obtained from empirical Bayes before–after study, propensity scores-potential outcomes framework, and regression model with cross-sectional data', *Accident Analysis & Prevention*, vol. 75, pp. 144-54.

Wu, L & Lord, D 2016, 'Investigating the Influence of Dependence between Variables on Crash Modification Factors Developed Using Regression Models', in *Transportation Research Board 95th Annual Meeting, TRB*, Washington, DC.

Xiang, Y, Li, Z, Wang, W, Chen, J, Wang, H & Li, Y 2016, 'Evaluating the Operational Features of an Unconventional Dual-Bay U-Turn Design for Intersections', *PLoS One*, vol. 11, no. 7, p. 18.

Yan, X, Radwan, E & Abdel-Aty, M 2005, 'Characteristics of rear-end accidents at signalized intersections using multiple logistic regression model', *Accident Analysis & Prevention*, vol. 37, no. 6, pp. 983-95.

Yanmaz-Tuzel, O & Ozbay, K 2010, 'A comparative Full Bayesian before-and-after analysis and application to urban road safety countermeasures in New Jersey', *Accident Analysis & Prevention*, vol. 42, no. 6, pp. 2099-107.

Young, J & Park, PY 2013, 'Benefits of small municipalities using jurisdiction-specific safety performance functions rather than the Highway Safety Manual's calibrated or uncalibrated safety performance functions', *Canadian Journal of Civil Engineering*, vol. 40, no. 6, pp. 517-27.

Zegeer, CV & Deacon, JA 1987, 'Effect of lane width, shoulder width, and shoulder type on highway safety', *State of the art report*, vol. 6, pp. 1-21.

Zou, Y, Lord, D, Zhang, Y & Peng, Y 2013, 'Comparison of sichel and negative binomial models in estimating empirical bayes estimates', *Transportation Research Record*, vol. 2392, pp. 11-21.

Appendix A: Selected Sites

Selected Road Intersections

Table A. 1 Details of selected intersections

Intersection ID	Road Location		Road Name		Traffic control type
	Latitude	Longitude	Major	Minor	
I_NE1	-27.552	151.955	Ruthven St	Bridge St	Operating traffic lights
I_NE2	-27.565	151.953	Ruthven St	Herries St	Operating traffic lights
I_NE3	-27.569	151.952	James St	Ruthven St	Operating traffic lights
I_NE4	-27.569	151.954	James St	Neil St	Operating traffic lights
I_NE5	-27.569	151.956	James St	Hume St	Operating traffic lights
I_NE6	-27.570	151.961	James St	Geddes St	Stop sign
I_NE7	-27.570	151.964	James St	Kitchener St	Operating traffic lights
I_NE8	-27.571	151.970	James St	Mackenzie St	Operating traffic lights
I_NE9	-27.571	151.972	James St	Curzon St	Stop sign
I_NE10	-27.565	151.955	Herries St	Neil St	Operating traffic lights
I_NE11	-27.562	151.958	Hume St	Margaret St	Operating traffic lights
I_NE12	-27.552	151.959	Bridge St	Hume St	Operating traffic lights
I_NE13	-27.552	151.957	Bridge St	Raff St	Stop sign
I_NE14	-27.568	151.975	Cohoe St	Herries St	Stop sign
I_NE15	-27.553	151.965	Bridge St	Lindsay St	No traffic control
I_NE16	-27.543	151.961	North St	Hume St	Give way sign
I_NE17	-27.529	151.958	Ruthven St.	Griffiths St	Operating traffic lights
I_NE18	-27.563	151.966	Margaret St	Mary St	Operating traffic lights
I_NE19	-27.546	151.956	Ruthven St.	Jellicoe St	Operating traffic lights
I_NE20	-27.561	151.956	Margaret St	Neil St	Operating traffic lights
I_NE21	-27.560	151.958	Hume St	Chalk Dr	Operating traffic lights
I_NE22	-27.556	151.959	Hume St	Campbell St	Operating traffic lights
I_NE23	-27.556	151.954	Ruthven St	Campbell St	Operating traffic lights
I_NE24	-27.542	151.954	Ruthven St	North St	Operating traffic lights
I_NE25	-27.554	151.968	Bridge St	Mary St	Operating traffic lights
I_NE26	-27.566	151.962	Kitchener St	Herries St	Operating traffic lights
I_NE27	-27.547	151.960	Hume St	Jellicoe St	Operating traffic lights
I_NE28	-27.572	151.974	Cohoe St	James St	Give way sign
I_NW1	-27.561	151.928	Tor St	Hursley Rd	Operating traffic lights
I_NW2	-27.563	151.931	Anzac Ave	Vacy St & Lendrum St	Give way sign
I_NW3	-27.550	151.930	Tor St	Victory St	Stop sign
I_NW4	-27.556	151.940	West St	Taylor St	Operating traffic lights
I_NW5	-27.550	151.945	West St	Bridge St	Operating traffic lights

Table A. 1 Details of selected intersections (continue)

Intersection ID	Site Location		Road Name		Traffic control type
	Latitude	Longitude	Major	Minor	
I_NW6	-27.560	151.943	West St	Margaret St	Operating traffic lights
I_NW7	-27.565	151.942	West St	Vacy St	Give way sign
I_NW8	-27.554	151.929	Tor St	Taylor St	Operating traffic lights
I_NW9	-27.547	151.930	Bridge St	Tor St	Operating traffic lights
I_NW10	-27.545	151.927	Bridge St	Tara St	Give way sign
I_NW11	-27.549	151.935	Bridge St	Holberton St	Operating traffic lights
I_NW12	-27.551	151.950	Bridge St	Mort St	Operating traffic lights
I_NW13	-27.544	151.923	Bridge St	Richmond Dr	Operating traffic lights
I_NW14	-27.550	151.903	Taylor St	Boundary St	Operating traffic lights
I_NW15	-27.557	151.901	Boundary St	Hursley Rd	Operating traffic lights
I_NW16	-27.564	151.947	Clifford St	Herries St	Operating traffic lights
I_NW17	-27.564	151.948	Herries St	Prescott St	Give way sign
I_NW18	-27.560	151.948	Clifford St	Margaret St	Operating traffic lights
I_NW19	-27.560	151.949	Margaret St	Mylne St	Operating traffic lights
I_NW20	-27.557	151.949	Taylor St	Mort St	Stop sign
I_NW21	-27.562	151.933	Anzac Ave	Herries St	Give way sign
I_NW22	-27.538	151.925	North St	Richmond Dr	Operating traffic lights
I_NW23	-27.553	151.924	Taylor St	McGregor St	Give way sign
I_NW24	-27.552	151.917	Taylor St	Greenwattle St	Operating traffic lights
I_NW25	-27.542	151.919	Bridge St	Greenwattle St	Stop sign
I_NW26	-27.549	151.938	Bridge St	Gordon Ave	Stop sign
I_NW27	-27.538	151.912	Bridge St	McDougall St	Operating traffic lights
I_NW28	-27.551	151.909	Taylor St	McDougall St	Operating traffic lights
I_NW29	-27.553	151.921	Taylor St	Wyalla St	Give way sign
I_NW30	-27.563	151.943	West St	Herries St	Operating traffic lights
I_NW31	-27.536	151.905	Bridge St	Boundary St	Operating traffic lights
I_NW32	-27.554	151.934	Taylor St	Holberton St	Operating traffic lights
I_NW33	-27.541	151.944	North St	Mort St	Operating traffic lights
I_NW34	-27.545	151.925	Bridge St	McGregor St	Operating traffic lights
I_SW1	-27.599	151.936	West St	Spring St	Operating traffic lights
I_SW2	-27.592	151.938	West St	Stenner St	Operating traffic lights
I_SW3	-27.590	151.938	West St	Charnley St	Give way sign
I_SW4	-27.583	151.939	West St	Alderley St	Operating traffic lights
I_SW5	-27.576	151.941	West St	South St	Operating traffic lights
I_SW6	-27.572	151.941	West St	Stephen St	Operating traffic lights
I_SW7	-27.569	151.942	West St	O'Quinn St	Give way sign
I_SW8	-27.567	151.942	James St	West St	Operating traffic lights
I_SW9	-27.579	151.940	West St	Derwak St	Give way sign
I_SW10	-27.574	151.924	Anzac Ave	South St	Operating traffic lights
I_SW11	-27.566	151.930	James St	Anzac Ave	Operating traffic lights

Table A. 1 Details of selected intersections (continue)

Intersection ID	Site Location		Road Name		Traffic control type
	Latitude	Longitude	Major	Minor	
I_SW12	-27.560	151.921	Hursley Rd	Japonica St	No traffic control
I_SW13	-27.568	151.947	James St	Pechey St	Operating traffic lights
I_SW14	-27.568	151.949	James St	Fitzgibbon St	Stop sign
I_SW15	-27.590	151.916	Anzac Ave	Ball St	Stop sign
I_SW16	-27.586	151.940	Cortess St	Hoey St	Stop sign
I_SW17	-27.591	151.927	Luck St	Wuth St	Give way sign
I_SW18	-27.592	151.935	Stenner St	Platz St	No traffic control
I_SW19	-27.580	151.920	Anzac Ave	Alderley St	Operating traffic lights
I_SW20	-27.570	151.927	Anzac Ave	Stephen St	Operating traffic lights
I_SW21	-27.576	151.939	Drayton Rd	South St	Operating traffic lights
I_SW22	-27.575	151.932	South St	Burton St	Give way sign
I_SW23	-27.565	151.923	Glenvale Rd	Hampton St	Stop sign
I_SE1	-27.576	151.974	Perth St	Cohoe St	Give way sign
I_SE2	-27.573	151.958	Perth St	Phillip St	Give way sign
I_SE3	-27.586	151.959	Alderley St	Hogan St	No traffic control
I_SE4	-27.612	151.949	Nelson St	Hume St	Stop sign
I_SE5	-27.593	151.961	Ramsay St	Ruth St	No traffic control
I_SE6	-27.579	151.964	Ramsay St	Cranley St	No traffic control
I_SE7	-27.577	151.967	Long St	View St	No traffic control
I_SE8	-27.582	151.978	South St	High St	Stop sign
I_SE9	-27.573	151.951	Ruthven St	Perth St	Operating traffic lights
I_SE10	-27.575	151.951	Ruthven St	Long St	Operating traffic lights
I_SE11	-27.577	151.951	Ruthven St	Healy St	Give way sign
I_SE12	-27.578	151.950	Ruthven St	South St	Give way sign & stop sign
I_SE13	-27.585	151.949	Ruthven St	Alderley St	Operating traffic lights
I_SE14	-27.593	151.948	Ruthven St	Stenner St	Operating traffic lights
I_SE15	-27.601	151.947	Ruthven St	Spring St	Operating traffic lights
I_SE16	-27.612	151.945	Ruthven St	Nelson St	Operating traffic lights
I_SE17	-27.590	151.948	Ruthven St	Donahue St	Stop sign
I_SE18	-27.588	151.949	Ruthven St	Carey St	Give way sign
I_SE19	-27.583	151.950	Ruthven St	Pierce St	Give way sign
I_SE20	-27.587	151.953	Hume St	Crotty St	Give way sign
I_SE21	-27.580	151.962	South St	Ramsay St	Operating traffic lights

Selected Roundabouts

Table A. 2 Details of selected roundabouts

Roundabout ID	Site Location		Road Name	
	latitude	Longitude	Major Road	Minor Road
R_NE1	-27.563	151.907	Curzon St	Herries Rd
R_NE2	-27.564	151.915	Herries St	Mary St.
R_NE3	-27.573	151.914	Jellicoe St	Stuart St.
R_NE4	-27.581	151.924	Bridge St	Mackenzie St.
R_NE5	-27.594	151.929	Bridge St	Curzon St.
R_NE6	-27.583	151.933	Mackenzie St	Herries Rd
R_NE7	-27.590	151.928	James St	Burke St
R_NE8	-27.605	151.977	Margaret St	Kitchener St
R_NE9	-27.604	151.968	Margaret St	Lindsay St
R_NW1	-27.602	151.960	Hursley Rd	Markelee St
R_NW2	-27.601	151.951	Hursley Rd	Greenwattle St
R_NW3	-27.596	151.970	North St	Tor St
R_NW4	-27.595	151.961	North St	Holberton St
R_NW5	-27.594	151.952	Hursley Rd	Corfield Dr.
R_NW6	-27.592	151.970	Carrington Rd	Toowoomba-Cecil-Plains Rd & Troys Rd
R_NW7	-27.591	151.983	Anzac Ave.	Hursley Rd& holberton St
R_NW8	-27.588	151.971	West St	Russell St & Anzac Ave
R_SW1	-27.587	151.962	Glenvale Rd	Boundary St
R_SW2	-27.585	151.954	Glenvale Rd	McDougall St
R_SW3	-27.563	151.907	Greenwattle St	Glenvale Rd
R_SW4	-27.564	151.915	Greenwattle St	South St
R_SW5	-27.573	151.914	Alderley St	Spencer St
R_SW6	-27.581	151.924	Wuth St	Gorman St
R_SW7	-27.594	151.929	Drayton Rd	Alderley St
R_SW8	-27.583	151.933	Stenner St	Luck St & Drayton Rd
R_SE1	-27.590	151.928	Spring St	Rowbotham St
R_SE2	-27.605	151.977	Spring St	Mackenzie St
R_SE3	-27.604	151.968	Ramsay St	Spring St
R_SE4	-27.602	151.960	Hume St	Spring St
R_SE5	-27.601	151.951	Mackenzie St	Stenner St
R_SE6	-27.596	151.970	Ramsay St	Stenner St
R_SE7	-27.595	151.961	Hume St	Stenner St
R_SE8	-27.594	151.952	Mackenzie St	Ballin Dr.& Waterbird Dr.
R_SE9	-27.592	151.970	Alderley St	Rowbotham St
R_SE10	-27.591	151.983	Mackenzie St	Alderley St
R_SE11	-27.588	151.971	Ramsay St	Alderley St
R_SE12	-27.587	151.962	Hume St	Alderley St

Table A. 2 Details of selected roundabouts (continue)

Roundabout ID	Site Location		Road Name	
	latitude	Longitude	Major Road	Minor Road
R_SE13	-27.581	151.971	Mackenzie St	South St
R_SE14	-27.580	151.959	South St	Geddes St
R_SE15	-27.579	151.955	Hume St	South St
R_SE16	-27.578	151.979	Tourist Rd	long St & High St
R_SE17	-27.577	151.969	Mackenzie St	Long St
R_SE18	-27.576	151.964	Ramsay St	Long St
R_SE19	-27.576	151.960	Long St	Geddes St
R_SE20	-27.575	151.955	Hume St	Long St
R_SE21	-27.575	151.969	Mackenzie St	Perth St
R_SE22	-27.574	151.965	Ramsay St	Perth St
R_SE23	-27.574	151.960	Perth St	Geddes St
R_SE24	-27.573	151.956	Hume St	Perth St

Selected Roadway Segments:

Table A. 3 Details of selected road segments

Segment ID	From Coordinates		To Coordinates		Road Name	Segment Range		Length (m)
	Latitude	Longitude	Latitude	Longitude		From	To	
S_SW1	-27.602	151.936	-27.599	151.936	West St	Heather St	Spring St	359
S_SW2	-27.599	151.936	-27.592	151.938	West St	Spring St	Stenner St	820
S_SW3	-27.590	151.938	-27.584	151.939	West St	Charnley St	Alderley St	705
S_SW4	-27.584	151.939	-27.577	151.940	West St	Alderley St	Peak St	736
S_SW5	-27.572	151.941	-27.569	151.942	West St	Stephen St	O'Quinn St	360
S_SW6	-27.599	151.936	-27.600	151.946	Spring St	West St	Ruthven St	995
S_SW7	-27.602	151.936	-27.596	151.930	Wuth St	West St	Platz St	840
S_SW8	-27.574	151.924	-27.570	151.926	Anzac Ave	South St	Stephen St	463
S_SW9	-27.570	151.926	-27.567	151.929	Anzac Ave	Stephen St	O'Quinn St	370
S_SW10	-27.592	151.938	-27.593	151.944	Stenner St	West St	Lemway Ave	620
S_SW11	-27.584	151.939	-27.585	151.948	Alderley St	West St	Ruthven St	880
S_SW12	-27.583	151.939	-27.529	151.934	Alderley St	West St	Drayton Rd	544
S_SW13	-27.583	151.933	-27.582	151.928	Alderley St	Drayton Rd	Chilla St	500
S_SW14	-27.583	151.933	-27.587	151.932	Drayton Rd	Alderley St	Eiser St	420
S_SW15	-27.576	151.939	-27.574	151.925	South St	Drayton Rd	Condammine St	1,400
S_SW16	-27.592	151.938	-27.590	151.928	Stenner St	West St	Drayton Rd	900
S_SW17	-27.565	151.923	-27.564	151.915	Glenvale Rd	Hampton St	Greenwattle St	750
S_SW18	-27.577	151.951	-27.577	151.947	Healy St	Ruthven St	Water St	340
S_SW19	-27.590	151.916	-27.593	151.914	Anzac Ave	Ball St	Parker St	400

Table A. 3 Details of selected road segments (continue)

Segment ID	From Coordinates		To Coordinates		Road Name	Segment Range		Length (m)
	Latitude	Longitude	Latitude	Longitude		From	To	
S_SW20	-27.593	151.944	-27.593	151.947	Stenner St	Lemway Ave	Ruthven St	370
S_SW21	-27.603	151.936	-27.610	151.935	West St	Nelson St	Heather St	778
S_NW1	-27.567	151.942	-27.567	151.939	James St	Ruthven St	Helen St	220
S_NW2	-27.567	151.939	-27.567	151.935	James St	Helen St	Mirle St	322
S_NW3	-27.558	151.943	-27.560	151.937	Anzac Ave	West St	Hill St	700
S_NW4	-27.554	151.944	-27.551	151.945	West St	Campbell St	Bridge St	350
S_NW5	-27.550	151.945	-27.549	151.938	Bridge St	West St	Gordon Ave	560
S_NW6	-27.563	151.943	-27.560	151.943	West St	Herries St	Margaret St	390
S_NW7	-27.549	151.938	-27.549	151.935	Bridge St	Gordon Ave	Holberton St	300
S_NW8	-27.549	151.935	-27.547	151.930	Bridge St	Holberton St	Tor St	420
S_NW9	-27.544	151.923	-27.542	151.919	Bridge St	Richmond Dr	Greenwattle St	460
S_NW10	-27.544	151.910	-27.539	151.911	McDougall St	Carroll St	Bridge St	580
S_NW11	-27.550	151.902	-27.547	151.895	Carrington Rd	Boundary St	Rielly St	780
S_NW12	-27.547	151.930	-27.551	151.930	Tor St	Bridge St	Pottinger St	440
S_NW13	-27.544	151.910	-27.543	151.906	Carroll St	McDougall St	Industrial Ave	410
S_NW14	-27.556	151.929	-27.554	151.929	Tor St	Ascot St	Taylor St	280
S_NW15	-27.561	151.928	-27.560	151.916	Hursley Rd	Tor St	Greenwattle St	1,180
S_NW16	-27.560	151.916	-27.558	151.908	Hursley Rd	Greenwattle St	McDougall St	469
S_NW17	-27.564	151.915	-27.560	151.916	Greenwattle St	Glenvale Rd	Hursley Rd	520
S_NW18	-27.536	151.906	-27.538	151.911	Bridge St	Boundary St	McDougall St	630
S_NW19	-27.545	151.925	-27.553	151.924	McGregor St	Bridge St	Taylor St	920
S_NW20	-27.553	151.924	-27.552	151.918	Taylor St	McGregor St	Greenwattle St	620
S_NW21	-27.603	151.936	-27.610	151.935	James St	Mirle St	Anzac Ave	250

Table A. 3 Details of selected road segments (continue)

Segment ID	From Coordinates		To Coordinates		Road Name	Segment Range		Length (m)
	Latitude	Longitude	Latitude	Longitude		From	To	
S_NW22	-27.561	151.928	-27.564	151.927	Tor St	Hursley Rd	Gatfield St	290
S_NW23	-27.536	151.933	-27.539	151.932	Tor St	Welcombe Ave	North St	374
S_SE1	-27.593	151.948	-27.594	151.952	Stenner St	Ruthven St	Hume St	360
S_SE2	-27.594	151.952	-27.595	151.961	Stenner St	Hume St	Ramsay St	850
S_SE3	-27.585	151.954	-27.579	151.955	Hume St	Alderley St	South St	710
S_SE4	-27.601	151.951	-27.602	151.954	Spring St	Hume St	Ramsay St	350
S_SE5	-27.587	151.962	-27.586	151.958	Alderley St	Ramsay St	Geddes St	420
S_SE6	-27.601	151.946	-27.606	151.946	Ruthven St	Spring St	Nelson St	540
S_SE7	-27.587	151.962	-27.593	151.961	Ramsay St	Alderley St	Stenner St	700
S_SE8	-27.583	151.950	-27.578	151.950	Ruthven St	Alderley St	South St	520
S_SE9	-27.577	151.951	-27.575	151.951	Ruthven St	South St	Long St	270
S_SE10	-27.573	151.956	-27.569	151.956	Hume St	Perth St	James St	440
S_SE11	-27.573	151.956	-27.574	151.960	Perth St	Hume St	Geddes St	430
S_SE12	-27.580	151.963	-27.581	151.971	South St	Ramsay St	Mackenzie St	750
S_SE13	-27.580	151.963	-27.576	151.964	Ramsay St	South St	Long St	420
S_SE14	-27.602	151.960	-27.604	151.968	Spring St	Ramsay St	Mackenzie St	760
S_SE15	-27.586	151.958	-27.580	151.959	Geddes St	Alderley St	South St	700
S_SE16	-27.574	151.964	-27.571	151.965	Ramsay St	Perth St	Kitchener St	260
S_SE17	-27.595	151.961	-27.596	151.970	Stenner St	Ramsay St	Mackenzie St	870
S_SE18	-27.587	151.962	-27.580	151.963	Ramsay St	Alderley St	South St	710
S_SE19	-27.575	151.955	-27.573	151.956	Hume St	Long St	Perth St	230
S_SE20	-27.575	151.955	-27.579	151.955	Hume St	Long St	South St	430
S_NE1	-27.569	151.957	-27.570	151.961	James St	Hume St	Geddes St	420

Table A.3 Details of selected road segments (continue)

Segment ID	From Coordinates		To Coordinates		Road Name	Segment Range		Length (m)
	Latitude	Longitude	Latitude	Longitude		From	To	
S_NE2	-27.569	151.957	-27.569	151.954	James St	Hume St	Neil St	210
S_NE3	-27.569	151.957	-27.567	151.957	Hume St	James St	Gore St	300
S_NE4	-27.569	151.952	-27.568	151.949	James St	Ruthven St	Fitzgibbon St	839
S_NE5	-27.554	151.968	-27.552	151.959	Bridge St	Mary St	Hume St	860
S_NE6	-27.568	151.947	-27.568	151.943	James St	Pechey St	West St	480
S_NE7	-27.564	151.947	-27.563	151.943	Herries St	Clifford St	West St	380
S_NE8	-27.560	151.947	-27.560	151.944	Margaret St	Clifford St	West St	410
S_NE9	-27.562	151.964	-27.562	151.959	Margaret St	Lindsay St	Kitchener St	430
S_NE10	-27.562	151.959	-27.566	151.961	Kitchener St	Margaret St	Herries St	530
S_NE11	-27.566	151.973	-27.568	151.973	Curzon St	Margaret St	Herries St	200
S_NE12	-27.536	151.955	-27.540	151.954	Ruthven St	Jones St	Mole St	360
S_NE13	-27.552	151.959	-27.556	151.959	Hume St	Bridge St	Campbell St	430
S_NE14	-27.533	151.956	-27.536	151.955	Ruthven St	Kate St	Mole St	340
S_NE15	-27.565	151.957	-27.562	151.958	Hume St	Aubigny St	Margaret St	310
S_NE16	-27.532	151.956	-27.530	151.957	Ruthven St	Mabel St	Gregory St	523
S_NE17	-27.554	151.972	-27.554	151.968	Bridge St	Mackenzie St	Mary St	470
S_NE18	-27.556	151.959	-27.556	151.954	Campbell St	Hume St	Ruthven St	420
S_NE19	-27.550	151.955	-27.546	151.956	Ruthven St	Delacy St	Jellicoe St	410
S_NE20	-27.543	151.961	-27.542	151.954	North St	Hume St	Ruthven St	670

Appendix B: Modelling Outputs

Statistical Modelling Results for Intersections:

Model I

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-9.251	2.1548	-12.795	-5.707	18.432	1	.000
Lg_i	.622	.1568	.364	.880	15.750	1	.000
LT_1	.056	.1420	-.177	.290	.158	1	.091
RT_1	-.034	.0779	-.162	.094	.195	1	.005
Q_{major}	.283	.1938	-.036	.602	2.130	1	.144
Q_{minor}	.281	.1697	.002	.560	2.737	1	.098
SL_2	.316	.1035	.146	.486	9.310	1	.000
MI_2	-.329	.1366	-.554	-.104	5.797	1	.016
V_i	.038	.0226	.000	.075	2.743	1	.000
Overdispersion parameter (Scale)	.210 [a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, Lg_i, LT_1, RT_1, Q_{major}, Q_{minor}, SL_2, MI_2, V_i.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	81.126	96	.845
Scaled Deviance	97.993	96	
Pearson Chi-Square	79.470	96	.825
Scaled Pearson Chi-Square	96.000	96	
Log Likelihood [b,c]	-117.083		
Adjusted Log Likelihood [d]	-141.425		
Akaike's Information Criterion (AIC)	254.166		
Finite Sample Corrected AIC (AICC)	256.482		
Bayesian Information Criterion (BIC)	280.801		
Consistent AIC (CAIC)	290.801		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, Lg_i, LT_1, RT_1, Q_{major}, Q_{minor}, SL_2, MI_2, V_i.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model II

Parameter Estimates

			90% Wald Confidence Interval		Hypothesis Test		
Parameter	*β*	*Std. Error*	*Lower*	*Upper*	*Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-1.536	.4273	-2.238	-.833	12.911	1	.000
LE_{i1}	.448	.1216	.248	.648	13.598	1	.000
LE_{i2}	.166	.1043	-.006	.337	2.530	1	.112
LT_1	.298	.2028	-.035	.632	2.165	1	.141
SL_1	-.068	.1819	-.368	.231	.141	1	.707
MI_1	-.560	.2290	-.937	-.184	5.987	1	.014
Overdispersion parameter (Scale)	.102[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{i1}, LE_{i2}, LT_1, SL_1, MI_1.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	df	Value/df
Deviance	103.509	100	1.035
Scaled Deviance	109.808	100	
Pearson Chi-Square	94.263	100	.943
Scaled Pearson Chi-Square	100.000	100	
Log Likelihood[b,c]	-137.555		
Adjusted Log Likelihood[d]	-145.926		
Akaike's Information Criterion (AIC)	287.110		
Finite Sample Corrected AIC (AICC)	287.958		
Bayesian Information Criterion (BIC)	303.090		
Consistent AIC (CAIC)	309.090		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{i1}, LE_{i2}, LT_1, SL_1, MI_1.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model III

Parameter Estimates

			90% Wald Confidence Interval		Hypothesis Test		
Parameter	β	Std. Error	Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-4.094	1.6476	-6.804	-1.384	6.175	1	.013
LN_{I2}	.116	.0527	.029	.203	4.853	1	.028
LE_{i1}	.146	.0731	.026	.266	3.983	1	.006
LT_2	-.075	.1480	-.318	.169	.256	1	.000
RT_2	-.067	.0934	-.221	.086	.516	1	.473
Q_{minor}	.430	.1894	.119	.742	5.157	1	.023
SL_2	.247	.1158	.057	.437	4.554	1	.000
MI_1	-.154	.1399	-.384	.076	1.218	1	.270
Overdispersion parameter (Scale)	.330[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{I2}, LE_{i1}, LT_2, RT_2, Q_{minor}, SL_2, MI_1.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	91.564	99	.925
Scaled Deviance	113.222	99	
Pearson Chi-Square	80.063	99	.809
Scaled Pearson Chi-Square	99.000	99	
Log Likelihood[b,c]	-141.377		
Adjusted Log Likelihood[d]	-174.817		
Akaike's Information Criterion (AIC)	294.754		
Finite Sample Corrected AIC (AICC)	295.896		
Bayesian Information Criterion (BIC)	313.398		
Consistent AIC (CAIC)	320.398		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{l2}, LE_{i1}, LT_2, RT_2, Q_{minor}, SL_2, MI_1.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model IV

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-1.300	.4770	-2.084	-.515	7.423	1	.006
LN_{i1}	.398	.1387	.170	.626	8.220	1	.000
TC	-.136	.2504	-.548	.276	.294	1	.588
LT_1	.472	.2190	.112	.832	4.648	1	.031
RT_2	.231	.1503	-.016	.478	2.360	1	.124
SL_2	.021	.2106	-.367	.326	.010	1	.000
MI_1	-.597	.2409	-.993	-.201	6.137	1	.013
MI_2	.392	.2714	-.054	.838	2.085	1	.149
Overdispersion parameter (Scale)	.271[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{i1}, *TC*, LT_1, RT_2, SL_2, MI_1, MI_2.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	92.836	98	.947
Scaled Deviance	114.685	98	
Pearson Chi-Square	79.329	98	.809
Scaled Pearson Chi-Square	98.000	98	
Log Likelihood[b,c]	-139.710		
Adjusted Log Likelihood[d]	-172.591		
Akaike's Information Criterion (AIC)	295.419		
Finite Sample Corrected AIC (AICC)	296.904		
Bayesian Information Criterion (BIC)	316.727		
Consistent AIC (CAIC)	324.727		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{i1}, *TC*, LT_1, RT_2, SL_2, MI_1, MI_2.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Statistical Modelling Results for Roundabouts:

Model I

Parameter Estimates

			90% Wald Confidence Interval		Hypothesis Test		
Parameter	*β*	*Std. Error*	*Lower*	*Upper*	*Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-15.930	3.4560	-21.615	-10.246	21.247	1	.000
LE_{r1}	.338	.0559	-.565	1.242	.379	1	.008
EX_1	-.068	.0054	-.521	.385	.061	1	.000
Q_{major}	.241	.0909	-.402	.884	.381	1	.117
Q_{minor}	1.121	.0566	.534	1.707	9.880	1	.000
WW	.305	.1431	.070	.541	4.547	1	.033
CD	-.005	.0206	-.039	.029	.055	1	.001
V_r	.038	.0410	-.030	.105	.849	1	.057
Overdispersion parameter (Scale)	.208[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{r1}, EX_1, Q_{major}, Q_{minor}, *WW*, *CD*, V_r.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	37.557	41	.916
Scaled Deviance	43.663	41	
Pearson Chi-Square	35.266	41	.860
Scaled Pearson Chi-Square	41.000	41	
Log Likelihood[b,c]	-70.133		
Adjusted Log Likelihood[d]	-81.536		
Akaike's Information Criterion (AIC)	156.265		
Finite Sample Corrected AIC (AICC)	159.865		
Bayesian Information Criterion (BIC)	171.400		
Consistent AIC (CAIC)	179.400		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{r1}, EX_1, Q_{major}, Q_{minor}, *WW*, *CD*, V_r.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model II

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-15.471	3.8411	-21.789	-9.153	16.223	1	.000
Lg_r	.467	.0502	-.364	1.298	.855	1	.021
Rn_2	.035	.0089	.020	.050	15.512	1	.000
Q_{major}	1.163	.1965	.675	1.650	15.377	1	.000
F	-.052	.2721	-.500	.396	.036	1	.103
WL	-.010	.0698	-.124	.105	.019	1	.006
CD	.012	.0246	-.029	.052	.224	1	.037
V_r	.023	.0399	-.043	.088	.323	1	.138
Overdispersion parameter (Scale)	.110[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, Lg_r, Rn_2, Q_{major}, *F*, *WL*, *CD*, V_r.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	40.348	41	.984
Scaled Deviance	44.495	41	
Pearson Chi-Square	37.179	41	.907
Scaled Pearson Chi-Square	41.000	41	
Log Likelihood[b,c]	-68.756		
Adjusted Log Likelihood[d]	-75.823		
Akaike's Information Criterion (AIC)	153.512		
Finite Sample Corrected AIC (AICC)	157.112		
Bayesian Information Criterion (BIC)	168.647		
Consistent AIC (CAIC)	176.647		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, Lg_r, Rn_2, Q_{major}, *F*, *WL*, *CD*, V_r.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model III

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-10.618	2.8619	-15.326	-5.911	13.765	1	.000
LN_{r2}	.022	.0282	-.443	.486	.006	1	.233
En_2	.367	.0296	-.121	.855	1.532	1	.004
Rx_2	-.024	.0149	-.048	.001	2.561	1	.000
Q_{major}	.403	.0823	-.226	1.032	1.110	1	.063
Q_{minor}	.915	.0544	.332	1.498	6.670	1	.000
CD	-.020	.0155	-.046	.005	1.702	1	.000
Overdispersion parameter (Scale)	.200[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{r2}, En_2, Rx_2, Q_{major}, Q_{minor}, *CD.*

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	35.937	42	.856
Scaled Deviance	41.257	42	
Pearson Chi-Square	36.584	42	.871
Scaled Pearson Chi-Square	42.000	42	
Log Likelihood[b,c]	-69.113		
Adjusted Log Likelihood[d]	-79.345		
Akaike's Information Criterion (AIC)	152.227		
Finite Sample Corrected AIC (AICC)	154.959		
Bayesian Information Criterion (BIC)	165.470		
Consistent AIC (CAIC)	172.470		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{r2}, En_2, Rx_2, Q_{major}, Q_{major}, *CD.*

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model IV

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-10.616	3.5633	-16.477	-4.755	8.876	1	.003
LN_{r1}	.564	.0421	-.113	1.240	1.877	1	.000
Ex_2	-.005	.0652	-.441	.431	.000	1	.108
Rn_1	.032	.0103	.015	.049	9.650	1	.000
Rx_1	-.020	.0109	-.038	-.003	3.529	1	.000
Q_{major}	.954	.1422	.392	1.517	7.778	1	.000
CW	.063	.1971	-.261	.387	.103	1	.208
Overdispersion parameter (Scale)	.220[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{r1}, Ex_2, Rn_1, Rx_1, Q_{major}, *CW.*

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	48.262	41	1.177
Scaled Deviance	44.852	41	
Pearson Chi-Square	44.118	41	1.076
Scaled Pearson Chi-Square	41.000	41	
Log Likelihood[b,c]	-69.187		
Adjusted Log Likelihood[d]	-64.297		
Akaike's Information Criterion (AIC)	154.373		
Finite Sample Corrected AIC (AICC)	157.973		
Bayesian Information Criterion (BIC)	169.508		
Consistent AIC (CAIC)	177.508		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LN_{r1}, Ex_2, Rn_1, Rx_1, Q_{major}, *CW.*

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model V

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-12.606	2.8285	-17.259	-7.954	19.864	1	.000
LE_{r2}	.079	.4712	-.696	.854	.028	1	.267
En_1	.307	.1059	.133	.481	8.417	1	.000
Q_{major}	.438	.0344	-.129	1.004	1.613	1	.004
Q_{minor}	.923	.0327	.384	1.461	7.942	1	.000
Overdispersion parameter (Scale)	.203[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{r2}, En_1, Q_{major}, Q_{minor}.

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	46.719	43	1.086
Scaled Deviance	43.211	43	
Pearson Chi-Square	46.490	43	1.081
Scaled Pearson Chi-Square	43.000	43	
Log Likelihood[b,c]	-67.984		
Adjusted Log Likelihood[d]	-62.880		
Akaike's Information Criterion (AIC)	147.967		
Finite Sample Corrected AIC (AICC)	149.967		
Bayesian Information Criterion (BIC)	159.318		
Consistent AIC (CAIC)	165.318		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept)*, LE_{r2}, En_1, Q_{major}, Q_{minor}.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Statistical Modelling Results for Roadway Segments:

Model I

Parameter Estimates

Parameter	β	Std. Error	90% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-6.380	2.2581	-10.094	-2.666	7.982	1	.001
SL	.340	.1921	.024	.656	3.130	1	.008
Q	.535	.1684	.258	.812	10.087	1	.000
MI	-.390	.2161	-.745	-.034	3.256	1	.001
G	.025	.0394	-.039	.090	.415	1	.320
Overdispersion parameter (Scale)	.550[a]						

Dependent Variable: $N_{pre,i}$
Model: *(Intercept), SL, Q, MI, G*
a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	df	Value/df
Deviance	84.060	79	1.064
Scaled Deviance	94.961	79	
Pearson Chi-Square	69.931	79	.885
Scaled Pearson Chi-Square	79.000	79	
Log Likelihood[b,c]	-136.970		
Adjusted Log Likelihood[d]	-154.734		
Akaike's Information Criterion (AIC)	283.941		
Finite Sample Corrected AIC (AICC)	284.710		
Bayesian Information Criterion (BIC)	296.095		
Consistent AIC (CAIC)	301.095		

Dependent Variable: Yi
Model: (Intercept), SL, Q, X4, X7
a. Information criteria are in smaller-is-better form.
b. The full log likelihood function is displayed and used in computing information criteria.
c. The log likelihood is based on a scale parameter fixed at 1.
d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model II

Parameter Estimates

Parameter	β	*Std. Error*	90% Wald Confidence Interval *Lower*	*Upper*	Hypothesis Test *Wald Chi-Square*	*df*	*Sig.*
(Intercept)	-8.284	2.4075	-12.244	-4.324	11.841	1	.000
SL	.282	.2008	-.048	.612	1.970	1	.060
Q	.878	.2353	.491	1.266	13.935	1	.000
NL	-.541	.2191	-.901	-.181	6.095	1	.000
EL	-.130	.1564	-.387	.127	.691	1	.106
CL	-.088	.3540	-.670	.494	.062	1	.204
Overdispersion parameter (Scale)	.490[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, NL, EL, CL.*

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	82.160	77	1.067
Scaled Deviance	92.610	77	
Pearson Chi-Square	68.311	77	.887
Scaled Pearson Chi-Square	77.000	77	
Log Likelihood[b,c]	-135.907		
Adjusted Log Likelihood[d]	-153.194		
Akaike's Information Criterion (AIC)	285.814		
Finite Sample Corrected AIC (AICC)	287.288		
Bayesian Information Criterion (BIC)	302.830		
Consistent AIC (CAIC)	309.830		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, NL, EL, CL.*

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model III

Parameter Estimates

Parameter	β	Std. Error	90% Wald Confidence Interval: Lower	90% Wald Confidence Interval: Upper	Hypothesis Test: Wald Chi-Square	df	Sig.
(Intercept)	-6.943	2.5674	-11.166	-2.720	7.312	1	.000
SL	.401	.1974	.076	.725	4.120	1	.002
Q	.367	.2057	.029	.705	3.183	1	.001
LW	-.135	.1096	-.315	.045	1.519	1	.000
SW	-.065	.0482	-.145	.014	1.825	1	.177
V_s	.040	.0310	-.011	.091	1.668	1	.197
Overdispersion parameter (Scale)	.610[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, LW, SW,* V_s

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	df	Value/df
Deviance	80.307	78	1.030
Scaled Deviance	93.489	78	
Pearson Chi-Square	67.002	78	.859
Scaled Pearson Chi-Square	78.000	78	
Log Likelihood[b,c]	-136.260		
Adjusted Log Likelihood[d]	-158.626		
Akaike's Information Criterion (AIC)	284.519		
Finite Sample Corrected AIC (AICC)	285.610		
Bayesian Information Criterion (BIC)	299.104		
Consistent AIC (CAIC)	305.104		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, LW, SW,* V_s.

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Model IV

Parameter Estimates

			90% Wald Confidence Interval		Hypothesis Test		
Parameter	β	Std. Error	Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-6.719	2.3401	-10.568	-2.870	8.244	1	.004
SL	.391	.2075	.049	.732	3.543	1	.000
Q	.536	.1731	.252	.821	9.595	1	.000
SW	-.062	.0455	-.137	.013	1.845	1	.174
AP	.038	.0437	-.034	.110	.760	1	.382
Overdispersion parameter (Scale)	.520[a]						

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, SW, AP.*

a. Computed based on the Pearson chi-square.

Goodness of Fit[a]

	Value	*df*	*Value/df*
Deviance	84.926	78	1.089
Scaled Deviance	93.972	78	
Pearson Chi-Square	70.491	78	.904
Scaled Pearson Chi-Square	78.000	78	
Log Likelihood[b,c]	-137.319		
Adjusted Log Likelihood[d]	-151.945		
Akaike's Information Criterion (AIC)	286.638		
Finite Sample Corrected AIC (AICC)	287.728		
Bayesian Information Criterion (BIC)	301.222		
Consistent AIC (CAIC)	307.222		

Dependent Variable: $N_{pre,i}$

Model: *(Intercept), SL, Q, SW, AP*

a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

c. The log likelihood is based on a scale parameter fixed at 1.

d. The adjusted log likelihood is based on an estimated scale parameter and is used in the model fitting omnibus test.

Appendix C: Simulation Outputs

Traffic Simulation Results for Intersection

Table C. 1 Average delay and LOS for intersection I_NW9

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	16.43	B	16.02	B
91	14.72	B	14.64	B
92	16.03	B	15.97	B
93	15.67	B	15.22	B
94	16.08	B	15.78	B
95	16.18	B	16.07	B
96	13.69	B	13.39	B
97	15.61	B	15.62	B
98	16.50	B	16.55	B
99	17.82	B	17.53	B
Average [b]	15.87	B	15.68	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 2 Average delay and LOS for intersection I_NE5

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	15.10	B	14.89	B
91	15.29	B	14.22	B
92	12.63	B	12.56	B
93	13.18	B	12.66	B
94	12.94	B	13.68	B
95	13.39	B	14.11	B
96	12.55	B	12.60	B
97	12.99	B	12.80	B
98	13.20	B	12.90	B
99	13.84	B	13.06	B
Average [b]	13.51	B	13.35	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 3 Average delay and LOS for intersection I_SE12

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	9.61	A	20.80	C
91	8.06	A	14.68	B
92	11.79	B	17.87	B
93	4.26	A	8.69	A
94	7.24	A	9.33	A
95	14.52	B	18.62	B
96	8.87	A	15.30	B
97	8.69	A	12.36	B
98	6.47	A	8.60	A
99	6.02	A	17.20	B
Average [b]	8.55	A	14.34	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 4 Average delay and LOS for intersection I_NW15

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	20.45	C	18.56	B
91	19.50	B	16.19	B
92	18.51	B	11.95	B
93	15.32	B	12.69	B
94	16.07	B	13.23	B
95	18.65	B	15.84	B
96	15.57	B	13.83	B
97	16.48	B	13.55	B
98	16.85	B	14.44	B
99	17.50	B	15.03	B
Average [b]	17.49	B	14.53	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 5 Average delay and LOS for intersection I_NE6

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	2.94	A	3.77	A
91	3.23	A	3.07	A
92	3.04	A	2.97	A
93	3.43	A	5.36	A
94	3.09	A	2.13	A
95	1.88	A	2.99	A
96	6.04	A	2.98	A
97	2.65	A	3.06	A
98	2.98	A	3.24	A
99	3.13	A	3.34	A
Average [b]	3.24	A	3.29	A

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 6 Average delay and LOS for intersection I_NW6

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	21.57	C	20.32	C
91	18.49	B	17.51	B
92	21.76	C	19.86	B
93	20.33	C	18.64	B
94	20.94	C	19.35	B
95	18.81	B	17.92	B
96	22.84	C	22.81	C
97	18.04	B	17.77	B
98	24.61	C	23.96	C
99	24.49	C	25.01	C
Average [b]	21.19	C	20.31	C

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 7 Average delay and LOS for intersection I_NE4

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	15.03	B	14.52	B
91	20.73	C	18.33	B
92	15.23	B	14.09	B
93	14.97	B	13.81	B
94	17.46	B	17.32	B
95	17.63	B	15.90	B
96	15.62	B	15.06	B
97	16.73	B	15.00	B
98	16.40	B	15.38	B
99	18.03	B	17.58	B
Average [b]	16.78	B	15.70	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 8 Average delay and LOS for intersection I_SW19

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	20.19	C	13.58	B
91	21.96	C	14.02	B
92	24.49	C	12.61	B
93	21.40	C	12.39	B
94	19.06	B	13.37	B
95	21.27	C	13.44	B
96	20.33	C	12.20	B
97	21.20	C	12.04	B
98	21.83	C	12.61	B
99	20.13	C	12.31	B
Average [b]	21.19	C	12.86	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 9 Average delay and LOS for intersection I_NW5

Seed Value	**Before treatments**		**After treatments**	
	Delay	**LOS [a]**	**Delay**	**LOS [a]**
90	16.67	B	16.74	B
91	18.15	B	17.75	B
92	16.78	B	17.76	B
93	19.30	B	19.94	B
94	16.93	B	16.83	B
95	18.12	B	17.68	B
96	18.43	B	18.85	B
97	17.66	B	17.21	B
98	19.60	B	20.11	C
99	17.54	B	17.93	B
Average [b]	17.92	B	18.08	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Table C. 10 Average delay and LOS for intersection I_NE28

Seed Value	**Before treatments**		**After treatments**	
	Delay	**LOS [a]**	**Delay**	**LOS [a]**
90	15.04	B	13.77	B
91	8.00	A	12.16	B
92	10.99	B	11.90	B
93	11.76	B	11.92	B
94	13.53	B	12.31	B
95	10.30	B	12.17	B
96	9.38	A	10.33	B
97	6.91	A	11.46	B
98	11.38	B	10.66	B
99	11.71	B	13.38	B
Average [b]	10.90	B	12.01	B

[a] LOS: Level of Service at intersections

[b] This result was based on 10-simulation runs with random seed values

Traffic Simulation Results for Roundabouts

Table C. 11 Average delay and LOS for roundabout R_NW7

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	11.61	B	8.18	A
91	20.64	C	17.23	C
92	19.85	C	10.45	B
93	12.92	B	9.86	A
94	11.81	B	8.29	A
95	18.28	C	8.02	A
96	17.93	C	9.84	A
97	13.58	B	21.34	C
98	7.61	A	4.79	A
99	17.31	C	13.21	B
Average [b]	15.15	C	11.12	B

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 12 Average delay and LOS for roundabout R_SE11

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	22.19	C	18.08	C
91	10.00	B	8.70	A
92	21.38	C	9.26	A
93	10.78	B	8.83	A
94	15.71	C	12.69	B
95	25.62	D	11.63	B
96	17.33	C	10.34	B
97	10.84	B	8.91	A
98	24.11	C	9.65	A
99	16.48	C	10.65	B
Average [b]	17.44	C	10.87	B

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 13 Average delay and LOS for roundabout R_SW3

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	15.15	C	13.71	B
91	13.12	B	8.30	A
92	12.56	B	5.96	A
93	13.79	B	8.24	A
94	11.42	B	9.85	A
95	15.05	C	10.09	B
96	24.60	C	11.85	B
97	17.22	C	11.87	B
98	14.51	B	11.88	B
99	25.02	D	11.06	B
Average [b]	16.24	C	10.28	B

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 14 Average delay and LOS for roundabout R_SW2

Seed Value	Before treatments		After treatments	
	Delay	**LOS [a]**	**Delay**	**LOS [a]**
90	8.02	A	5.35	A
91	5.58	A	6.03	A
92	6.07	A	7.50	A
93	11.18	B	14.86	B
94	4.93	A	5.66	A
95	6.46	A	4.13	A
96	4.09	A	4.49	A
97	5.92	A	5.04	A
98	6.39	A	5.65	A
99	5.98	A	9.30	A
Average [b]	6.46	A	6.80	A

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 15 Average delay and LOS for roundabout R_NE1.

Seed Value	Before treatments		After treatments	
	Delay	**LOS [a]**	**Delay**	**LOS [a]**
90	5.12	A	6.88	A
91	5.44	A	8.29	A
92	10.55	B	9.03	A
93	9.40	A	8.25	A
94	7.44	A	7.04	A
95	5.21	A	6.45	A
96	5.12	A	8.13	A
97	5.92	A	5.51	A
98	8.06	A	8.06	A
99	6.95	A	9.22	A
Average [b]	6.92	A	7.68	A

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 16 Average delay and LOS for roundabout R_NE4.

Seed Value	Before treatments		After treatments	
	Delay	**LOS [a]**	**Delay**	**LOS [a]**
90	14.20	B	6.82	A
91	11.10	B	6.24	A
92	7.19	A	4.23	A
93	16.18	C	21.55	C
94	11.57	B	10.95	B
95	6.97	A	4.23	A
96	7.19	A	4.58	A
97	13.13	B	6.58	A
98	13.73	B	7.57	A
99	9.58	A	5.65	A
Average [b]	11.08	B	7.84	A

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 17 Average delay and LOS for roundabout R_NE7

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	5.51	A	5.53	A
91	12.60	B	8.38	A
92	5.97	A	5.61	A
93	5.90	A	9.25	A
94	5.60	A	4.71	A
95	10.53	B	6.59	A
96	7.51	A	5.72	A
97	10.96	B	7.58	A
98	5.46	A	5.96	A
99	7.04	A	5.64	A
Average [b]	7.71	A	6.50	A

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 18 Average delay and LOS for roundabout R_SE2.

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	5.94	A	10.46	B
91	5.01	A	5.24	A
92	6.67	A	7.73	A
93	5.14	A	5.46	A
94	4.68	A	6.20	A
95	5.07	A	9.13	A
96	4.91	A	5.14	A
97	4.78	A	5.07	A
98	2.73	A	2.27	A
99	13.58	B	16.15	C
Average [b]	5.85	A	7.28	A

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 19 Average delay and LOS for roundabout R_SE6

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	23.31	C	21.74	C
91	27.52	D	8.54	A
92	14.76	B	15.74	C
93	7.54	A	6.65	A
94	15.56	C	12.80	B
95	16.60	C	10.93	B
96	12.36	B	8.76	A
97	16.68	C	10.74	B
98	10.90	B	11.20	B
99	16.92	C	22.63	C
Average [b]	16.21	C	12.97	B

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Table C. 20 Average delay and LOS for roundabout R_SE13

Seed Value	Before treatments		After treatments	
	Delay	LOS [a]	Delay	LOS [a]
90	12.61	B	11.11	B
91	12.15	B	13.30	B
92	14.68	B	14.10	B
93	10.78	B	9.87	A
94	11.71	B	8.17	B
95	13.87	B	12.42	B
96	13.59	B	6.62	A
97	11.36	B	9.76	A
98	8.51	A	9.96	A
99	14.33	B	16.25	C
Average [b]	12.36	B	11.15	B

[a] LOS: Level of Service at roundabouts

[b] This result was based on 10-simulation runs with random seed values

Traffic Simulation Results for Roadway Segments

Table C. 21 Sample of VISSIM results for travel time at road segment S_NE8

Time Interval	Travel time Measurement	Vehicle (All)	Travel Time (sec/veh)	Distance Travelled (m)
0-600	1: Margaret_EB	30	19.77	312
0-600	2: Margaret_WB	23	19.09	312
600-1200	1: Margaret_EB	28	19.09	312
600-1200	2: Margaret_WB	32	18.84	312
1200-1800	1: Margaret_EB	17	18.75	312
1200-1800	2: Margaret_WB	30	19.09	312
1800-2400	1: Margaret_EB	25	18.97	312
1800-2400	2: Margaret_WB	25	19.51	312
2400-3000	1: Margaret_EB	35	19.40	312
2400-3000	2: Margaret_WB	23	20.04	312
3000-3600	1: Margaret_EB	21	20.13	312
3000-3600	2: Margaret_WB	32	19.22	312
Total	1: Margaret_EB	156	116.10	3744
Total	2: Margaret_WB	165	115.80	3432
Average	1: Margaret_EB	26	19.35	312
Average	2: Margaret_WB	28	19.30	312
Standard deviation	1: Margaret_EB	6	0.52	-
Standard deviation	2: Margaret_WB	4	0.42	-
Minimum	1: Margaret_EB	17	18.75	312
Minimum	2: Margaret_WB	23	18.84	312
Maximum	1: Margaret_EB	35	20.13	312
Maximum	2: Margaret_WB	32	20.04	312

Note: this simulation run was carried out using seed value equal to 90

Table C. 22 Average travel time for road segment S_NE8

Seed Value	Travel time Measurement		Ave. Travel Time
	Margaret-EB	Margaret-WB	
90	19.35	19.30	19.33
91	19.18	19.67	19.43
92	19.68	19.08	19.38
93	19.47	19.37	19.42
94	19.65	19.40	19.52
95	20.58	19.42	20.00
96	19.74	19.66	19.70
97	19.75	19.12	19.43
98	18.95	19.10	19.02
99	19.94	19.69	19.81
Grant average [a]			**19.50**

[a] This result was based on 10-simulation runs with random seed values

Table C. 23 Average travel time for road segment S_NE8 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Margaret-EB	Margaret-WB	
90	23.19	22.77	22.98
91	22.90	23.07	22.98
92	23.20	22.72	22.96
93	22.96	22.70	22.83
94	23.48	22.87	23.17
95	23.69	23.12	23.41
96	23.03	23.45	23.24
97	23.01	22.88	22.94
98	22.53	22.70	22.61
99	23.17	22.96	23.06
Grant average [a]			**23.02**

[a] This result was based on 10-simulation runs with random seed values

Table C. 24 Average travel time for road segment S_NW1

Seed Value	Travel time Measurement		Ave. Travel Time
	James-EB	James-WB	
90	13.88	13.94	13.91
91	13.93	13.94	13.93
92	13.86	13.87	13.86
93	13.86	13.91	13.89
94	13.86	13.84	13.85
95	13.88	13.93	13.90
96	13.91	13.88	13.89
97	13.88	13.91	13.89
98	13.89	13.85	13.87
99	13.86	13.93	13.90
Grant average [a]			**13.89**

[a] This result was based on 10-simulation runs with random seed values

Table C. 25 Average travel time for road segment S_NW1 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	James-EB	James-WB	
90	16.52	16.60	16.56
91	16.59	16.59	16.59
92	16.49	16.47	16.48
93	16.51	16.53	16.52
94	16.50	16.45	16.47
95	16.52	16.56	16.54
96	16.56	16.51	16.54
97	16.53	16.54	16.53
98	16.54	16.47	16.50
99	16.50	16.57	16.54
Grant average [a]			**16.53**

[a] This result was based on 10-simulation runs with random seed values

Table C. 26 Average travel time for road segment S_NW21

Seed Value	Travel time Measurement		Ave. Travel Time
	James-EB	James-WB	
90	12.47	12.69	12.58
91	12.51	2.69	7.60
92	12.46	12.64	12.55
93	12.46	12.69	12.57
94	12.45	12.65	12.55
95	12.49	12.62	12.55
96	12.50	12.72	12.61
97	12.46	12.72	12.59
98	12.49	12.65	12.57
99	12.46	12.71	12.59
Grant average [a]			**12.08**

[a] This result was based on 10-simulation runs with random seed values

Table C. 27 Average travel time for road segment S_NW21 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	James-EB	James-WB	
90	14.84	15.10	14.97
91	14.88	15.11	14.99
92	14.83	15.11	14.97
93	14.81	15.13	14.97
94	14.80	15.06	14.93
95	14.83	15.03	14.93
96	14.85	15.07	14.96
97	14.80	15.08	14.94
98	14.86	15.09	14.98
99	14.81	15.09	14.95
Grant average [a]			**14.96**

[a] This result was based on 10-simulation runs with random seed values

Table C. 28 Average travel time for road segment S_NW22

Seed Value	Travel time Measurement		Ave. Travel Time
	Tor-NB	Tor-SB	
90	16.48	16.02	16.25
91	16.49	15.99	16.24
92	16.38	15.96	16.17
93	16.56	15.99	16.27
94	16.54	15.93	16.23
95	16.45	16.02	16.23
96	16.52	15.94	16.23
97	16.53	15.97	16.25
98	16.49	15.94	16.22
99	16.53	16.00	16.27
Grant average [a]			**16.24**

[a] This result was based on 10-simulation runs with random seed values

Table C. 29 Average travel time for road segment S_NW22 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Tor-NB	Tor-SB	
90	19.47	19.11	19.29
91	19.42	19.08	19.25
92	19.37	19.04	19.21
93	19.42	19.05	19.24
94	19.35	18.96	19.16
95	19.41	19.11	19.26
96	19.49	18.99	19.24
97	19.46	19.14	19.30
98	19.42	18.98	19.20
99	19.44	19.09	19.26
Grant average [a]			**19.24**

[a] This result was based on 10-simulation runs with random seed values

Table C. 30 Average travel time for road segment S_SE5

Seed Value	Travel time Measurement		Ave. Travel Time
	Alderley-EB	Alderley-WB	
90	24.89	25.17	25.03
91	24.91	25.13	25.02
92	24.68	25.05	24.86
93	24.76	25.11	24.94
94	24.89	25.11	25.00
95	25.12	25.72	25.42
96	24.91	25.15	25.03
97	24.71	25.24	24.98
98	24.67	25.10	24.88
99	24.78	25.20	24.99
Grant average [a]			**25.01**

[a] This result was based on 10-simulation runs with random seed values

Table C. 31 Average travel time for road segment S_SE5 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Alderley-EB	Alderley-WB	
90	28.92	29.20	29.06
91	28.94	29.04	28.99
92	28.75	28.95	28.85
93	28.77	29.01	28.89
94	29.00	29.10	29.05
95	29.33	29.70	29.52
96	28.94	29.32	29.13
97	28.66	29.28	28.97
98	28.71	28.98	28.85
99	28.74	29.08	28.91
Grant average [a]			**29.02**

[a] This result was based on 10-simulation runs with random seed values

Table C. 32 Average travel time for road segment S_SE9

Seed Value	Travel time Measurement		Ave. Travel Time
	Ruthven-NB	Ruthven-SB	
90	12.90	12.89	12.89
91	13.04	12.76	12.90
92	13.03	12.71	12.87
93	12.86	12.14	12.50
94	12.92	12.83	12.88
95	13.00	12.86	12.93
96	12.93	12.79	12.86
97	13.08	12.90	12.99
98	12.99	12.87	12.93
99	13.00	12.10	12.55
Grant average [a]			**12.83**

[a] This result was based on 10-simulation runs with random seed values

Table C. 33 Average travel time for road segment S_SE9 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Ruthven-NB	Ruthven-SB	
90	15.11	15.13	15.12
91	15.13	14.98	15.06
92	15.15	14.93	15.04
93	15.09	14.98	15.03
94	15.17	14.94	15.05
95	15.13	15.10	15.12
96	15.18	15.02	15.10
97	15.16	15.07	15.11
98	15.17	15.05	15.11
99	15.13	15.04	15.09
Grant average [a]			**15.08**

[a] This result was based on 10-simulation runs with random seed values

Table C. 34 Average travel time for road segment S_SW4

Seed Value	Travel time Measurement		Ave. Travel Time
	West-NB	West-SB	
90	42.05	41.61	41.83
91	41.72	41.33	41.52
92	42.19	41.43	41.81
93	41.49	41.18	41.34
94	41.71	41.43	41.57
95	41.37	41.47	41.42
96	41.26	41.17	41.22
97	42.60	41.27	41.93
98	41.44	41.70	41.57
99	41.53	41.81	41.67
Grant average [a]			**41.59**

[a] This result was based on 10-simulation runs with random seed values

Table C. 35 Average travel time for road segment S_SW4 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	West-NB	West-SB	
90	49.01	49.71	49.36
91	48.69	49.16	48.93
92	48.77	49.78	49.28
93	48.53	49.06	48.79
94	48.67	49.22	48.94
95	48.96	48.77	48.87
96	48.51	49.05	48.78
97	48.70	50.34	49.52
98	49.02	48.80	48.91
99	49.23	49.01	49.12
Grant average [a]			**49.05**

[a] This result was based on 10-simulation runs with random seed values

Table C. 36 Average travel time for road segment S_SW8

Seed Value	Travel time Measurement		Ave. Travel Time
	Anzac-NB	Anzac-SB	
90	22.56	22.68	22.62
91	22.64	22.56	22.60
92	22.50	22.50	22.50
93	22.49	22.62	22.55
94	22.53	22.49	22.51
95	22.56	22.66	22.61
96	22.61	22.57	22.59
97	22.56	22.62	22.59
98	22.55	22.54	22.55
99	22.55	22.65	22.60
Grant average [a]			**22.57**

[a] This result was based on 10-simulation runs with random seed values

Table C. 37 Average travel time for road segment S_SW8 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Anzac-NB	Anzac-SB	
90	27.05	26.82	26.94
91	26.87	26.96	26.91
92	26.80	26.79	26.79
93	26.92	26.75	26.83
94	26.75	26.81	26.78
95	26.99	26.85	26.92
96	26.86	26.91	26.88
97	26.96	26.83	26.89
98	26.84	26.84	26.84
99	26.97	26.82	26.90
Grant average [a]			**26.87**

[a] This result was based on 10-simulation runs with random seed values

Table C. 38 Average travel time for road segment S_SW16

Seed Value	Travel time Measurement		Ave. Travel Time
	Stenner-EB	Stenner-WB	
90	53.10	53.91	53.50
91	53.06	53.58	53.32
92	53.11	53.74	53.42
93	52.74	53.38	53.06
94	53.28	53.53	53.40
95	52.96	53.98	53.47
96	53.22	53.57	53.39
97	52.61	53.69	53.15
98	53.07	53.48	53.28
99	52.78	53.61	53.19
Grant average [a]			**53.32**

[a] This result was based on 10-simulation runs with random seed values.

Table C. 39 Average travel time for road segment S_SW16 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Stenner-EB	Stenner-WB	
90	63.32	64.28	63.80
91	63.24	63.68	63.46
92	63.51	63.59	63.55
93	62.83	63.49	63.16
94	63.49	63.56	63.53
95	63.12	64.18	63.65
96	63.40	63.68	63.54
97	62.65	63.89	63.27
98	63.09	63.33	63.21
99	62.85	63.78	63.31
Grant average [a]			**63.45**

[a] This result was based on 10-simulation runs with random seed values

Table C. 40 Average travel time for road segment S_SW19

Seed Value	Travel time Measurement		Ave. Travel Time
	Anzac-NB	**Anzac-SB**	
90	22.58	21.83	22.20
91	22.96	21.67	22.32
92	22.86	21.63	22.25
93	22.73	21.67	22.20
94	22.74	21.62	22.18
95	23.17	21.81	22.49
96	22.64	21.70	22.17
97	22.61	21.72	22.17
98	22.84	21.66	22.25
99	22.96	21.73	22.35
Grant average [a]			**22.26**

[a] This result was based on 10-simulation runs with random seed values

Table C. 41 Average travel time for road segment S_SW19 (After)

Seed Value	Travel time Measurement		Ave. Travel Time
	Anzac-NB	**Anzac-SB**	
90	25.77	25.82	25.79
91	25.84	25.65	25.75
92	25.76	25.58	25.67
93	25.67	25.71	25.69
94	25.82	25.53	25.67
95	25.86	25.76	25.81
96	25.86	25.64	25.75
97	25.74	25.72	25.73
98	25.81	25.62	25.72
99	25.81	25.76	25.78
Grant average [a]			**25.74**

[a] This result was based on 10-simulation runs with random seed values

Appendix D: Economic Analysis

Benefit Analysis for Intersection Treatments

Table D. 1 Benefit analysis at intersections by treatment type

Intersection ID	Suggested Treatments	CMFs	Ave. Crashes/year		Crashes cost/ year (2006)		Cost saved /year (2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
I_NW9	$V_{60\text{-}50}$	0.68	5.50	3.74	1,753,319	1,192,257	736,162	5,970,935
	$V_{60\text{-}50}$+ AM_{minors}	0.59	5.50	3.26	1,753,319	1,038,842	937,457	7,603,612
	$V_{60\text{-}50}$+ AM_{minors}+ $A1LT_{1minor}$	0.58	5.50	3.20	1,753,319	1,019,847	962,379	**7,805,753**
I_NE5	AM_{majors}	0.58	4.00	2.32	1,275,141	739,582	702,700	5,699,529
	AM_{majors}+ $V_{60\text{-}50}$	0.50	4.00	1.98	1,275,141	632,258	843,519	6,841,696
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}	0.43	4.00	1.71	1,275,141	545,123	957,847	7,769,001
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ RTL_{minors}	0.41	4.00	1.63	1,275,141	519,620	991,309	8,040,407
	AM_{majors}+$V_{60\text{-}50}$+ AM_{minors}+ RTL_{minors} +$A1LT_{1minor}$	0.39	4.00	1.58	1,275,141	502,618	1,013,617	**8,221,344**
I_SE12	AM_{majors}	0.58	2.90	1.68	924,477	536,197	509,458	4,132,158
	AM_{majors}+$V_{60\text{-}50}$	0.50	2.90	1.44	924,477	458,387	611,551	4,960,230
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}	0.43	2.90	1.24	924,477	395,214	694,439	5,632,525
	AM_{majors}+$V_{60\text{-}50}$+AM_{minors}+ Signal	0.40	2.90	1.16	924,477	370,561	726,786	**5,894,885**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Discount rate (r) used equal 4%
Note: The calculations were performed without rounding

Table D. 1 Benefit analysis at intersections by treatment type (continue)

Intersection ID	Suggested Treatments	CMFs	Ave. Crashes/ year		Crashes cost/ year		Cost saved /year(2017)	PV [b] ($AUD)
			Before	After	Before	After		
I_NW15	AM_{majors}	0.58	3.30	1.91	1,051,992	610,155	579,728	4,702,111
	AM_{majors}+V_{60-50}	0.50	3.30	1.64	1,051,992	521,612	695,903	5,644,399
	AM_{majors}+V_{60-50}+AM_{minors}	0.43	3.30	1.41	1,051,992	449,726	790,224	6,409,425
	AM_{majors}+V_{60-50}+AM_{minors}+ $A1LT_{minors}$	0.41	3.30	1.37	1,051,992	435,700	808,628	6,558,699
	AM_{majors}+V_{60-50}+AM_{minors}+ $A1LT_{minors}$+$A1SL_{majors}$	0.40	3.30	1.32	1,051,992	421,673	827,032	**6,707,972**
I_NE6	AM_{majors}	0.58	2.50	1.45	796,963	462,239	439,188	3,562,205
	AM_{majors}+V_{60-50}	0.50	2.50	1.24	796,963	395,161	527,199	4,276,060
	AM_{majors}+V_{60-50}+AM_{minors}	0.38	2.50	0.94	796,963	300,190	651,810	**5,286,765**
I_NW6	AM_{majors}	0.58	2.60	1.51	828,842	480,728	456,755	3,704,694
	AM_{majors}+V_{60-50}	0.50	2.60	1.29	828,842	410,967	548,287	4,447,103
	AM_{majors}+V_{60-50}+ AM_{minors}	0.43	2.60	1.11	828,842	354,330	622,601	5,049,850
	AM_{majors}+V_{60-50}+ AM_{minors}+ RTL_{minors}	0.41	2.60	1.06	828,842	337,753	644,351	5,226,264
	AM_{majors}+V_{60-50}+ AM_{minors}+ RTL_{minors}+$A1LT_{minors}$	0.39	2.60	1.02	828,842	326,702	658,851	**5,343,874**

[a] using the average inflation rate 2.5% between 2006-2017

[b] Discount rate (r) used equal 4%

Note: The calculations were performed without rounding

Table D. 1 Benefit analysis at intersections by treatment type (continue)

Intersection ID	Suggested Treatments	CMFs	Ave. Crashes/ year		Crashes cost/ year		Cost saved /year(2017)	PV [b] ($AUD)
			Before	After	Before	After		
I_NE4	AM_{majors}	0.58	2.90	1.68	924,477	536,197	509,458	4,132,158
	AM_{majors}+ RTL_{majors}	0.5	2.90	1.44	924,477	458,387	611,551	4,960,230
	AM_{majors}+ RTL_{majors}+ V_{60-50}	0.41	2.90	1.20	924,477	382,888	710,613	5,763,705
	AM_{majors}+ RTL_{majors}+ V_{60-50}+AM_{minors}	0.37	2.90	1.07	924,477	339,745	767,219	6,222,834
	AM_{majors}+ RTL_{majors}+ V_{60-50}+AM_{minors}+ RTL_{minors}	0.35	2.90	1.02	924,477	325,108	786,425	6,378,610
	AM_{majors}+ RTL_{majors}+ V_{60-50}+AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$	0.34	2.90	0.99	924,477	315,093	799,566	6,485,193
	AM_{majors}+ RTL_{majors}+ V_{60-50}+AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$+ $A1RT_{majors}$	0.34	2.90	0.99	924,477	315,093	799,566	**6,485,193**
I_SW19	V_{60-50}	0.68	2.50	1.70	796,963	541,935	334,619	2,714,061
	V_{60-50}+ AM_{minors}	0.59	2.50	1.48	796,963	472,201	426,117	3,456,187
	V_{60-50}+ AM_{minors}+ RTL_{minors}	0.56	2.50	1.39	796,963	444,307	462,716	3,753,038
	V_{60-50}+ AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$	0.54	2.50	1.35	796,963	430,360	481,015	3,901,463
	V_{60-50}+ AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$+ $A1RT_{minors}$	0.53	2.50	1.31	796,963	418,406	496,700	4,028,685
	V_{60-50}+ AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$+ $A1RT_{minors}$+ $A1RT_{majors}$	0.52	2.50	1.30	796,963	413,093	503,672	4,085,228
	V_{60-50}+ AM_{minors}+ RTL_{minors}+ $A1LT_{minors}$+ $A1RT_{minors}$+ $A1RT_{majors}$ + $A1SL_{1major}$	0.51	2.50	1.29	796,963	409,772	508,029	**4,120,567**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Discount rate (r) used equal 4%
Note: The calculations were performed without rounding

Table D. 1 Benefit analysis at intersections by treatment type (continue)

Intersection ID	Suggested Treatments	CMFs	Ave. Crashes/ year		Crashes cost/ year		Cost saved /year(2017)	PV [b] ($AUD)
			Before	After	Before	After		
I_NW5	$V_{60\text{-}50}$	0.68	2.6	1.8	828,842	563,612	348,004	2,822,624
	$V_{60\text{-}50}$+ RTL_{1minor}	0.68	2.6	1.8	828,842	565,685	345,285	2,800,572
	$V_{60\text{-}50}$+ RTL_{1minor}+ $A1LT_{1minor}$	0.67	2.6	1.7	828,842	553,252	361,598	2,932,883
	$V_{60\text{-}50}$+ RTL_{1minor}+ $A1LT_{1minor}$+ $A1SL_{1major}$	0.66	2.6	1.7	828,842	547,726	368,848	**2,991,687**
I_NE28	$V_{60\text{-}50}$	0.68	1.9	1.3	605,692	411,871	254,311	2,062,687
	$V_{60\text{-}50}$+Signal	0.66	1.9	1.3	605,692	397,738	272,854	**2,213,091**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Discount rate (r) used equal 4%
Note: The calculations were performed without rounding

Benefit Analysis for Roundabout Treatments

Table D. 2 Benefit analysis at roundabouts by treatment type

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017)[a]	PV [b] ($AUD)
			Before	After	Before	After		
R_NW7	0.6_REn_{minors}	0.53	8.50	4.51	2,709,675	1,436,128	557,002	4,517,781
	0.6_REn_{minors} + 0.6_REn_{majors}	0.46	8.50	3.91	2,709,675	1,246,451	639,959	5,190,642
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major}	0.40	8.50	3.37	2,709,675	1,072,923	715,854	5,806,214
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50}	0.36	8.50	3.07	2,709,675	979,596	756,671	6,137,281
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW	0.34	8.50	2.87	2,709,675	916,134	784,427	6,362,406
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW + 0.6_IEx_{majors}	0.33	8.50	2.80	2,709,675	891,311	795,284	6,450,466
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW + 0.6_IEx_{majors} + A_Fixed	0.32	8.50	2.75	2,709,675	877,392	801,372	6,499,841
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr	0.32	8.50	2.72	2,709,675	867,104	805,871	6,536,337
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr + 1.2_ICi	0.32	8.50	2.70	2,709,675	862,286	807,978	6,553,427
	0.6_REn_{minors} + 0.6_REn_{majors} + 10_REnR_{major} + R_V_{60-50} + 0.6_RW + 0.6_IEx_{majors} + A_Fixed + 0.6_RCr + 1.2_ICi + 0.6_IEx_{minors}	0.32	8.50	2.70	2,709,675	859,986	808,984	6,561,588

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_NW7	0.6_REn$_{minors}$ + 0.6_REn$_{majors}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_RW + 0.6_IEx$_{majors}$ + A_Fixed + 0.6_RCr + 1.2_ICi + 0.6_IEx$_{minors}$	0.32	8.50	2.70	2,709,675	859,986	808,984	**6,561,588**
R_SE11	10_REnR$_{minors}$	0.50	2.50	1.25	796,963	398,482	174,281	1,413,574
	10_REnR$_{minors}$ + 10_REnR$_{majors}$	0.39	2.50	0.96	796,963	306,831	214,365	1,738,696
	10_REnR$_{minors}$ + 10_REnR$_{majors}$ + R_V$_{60-50}$	0.35	2.50	0.87	796,963	275,949	227,872	1,848,247
	10_REnR$_{minors}$ + 10_REnR$_{majors}$ + R_V$_{60-50}$ + 0.6_RW	0.32	2.50	0.80	796,963	255,513	236,810	1,920,740
	10_REnR$_{minors}$ + 10_REnR$_{majors}$ + R_V$_{60-50}$ + 0.6_RW + 0.6_RCr	0.32	2.50	0.79	796,963	251,582	238,529	1,934,686
	10_REnR$_{minors}$ + 10_REnR$_{majors}$ + R_V$_{60-50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi	0.31	2.50	0.78	796,963	249,796	239,310	**1,941,021**
R_SW3	0.6_REn$_{minors}$	0.53	4.50	2.39	1,434,534	760,303	294,883	2,391,767
	0.6_REn$_{minors}$ + 1.2_RW	0.46	4.50	2.08	1,434,534	662,695	337,573	2,738,021
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$	0.39	4.50	1.75	1,434,534	557,874	383,418	3,109,863
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$	0.34	4.50	1.52	1,434,534	485,729	414,971	3,365,790
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$	0.31	4.50	1.37	1,434,534	437,857	435,909	3,535,612

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_SW3	$0.6\_REn_{minors} + 1.2\_RW + 0.6\_REn_{majors} + 10\_REnR_{minor} + 10\_REnR_{major} + R\_V_{60\text{-}50}$	0.29	4.50	1.29	1,434,534	410,488	447,879	3,632,700
	$0.6\_REn_{minors} + 1.2\_RW + 0.6\_REn_{majors} + 10\_REnR_{minor} + 10\_REnR_{major} + R\_V_{60\text{-}50} + 0.6\_IEx_{majors}$	0.28	4.50	1.26	1,434,534	401,457	451,829	3,664,737
	$0.6\_REn_{minors} + 1.2\_RW + 0.6\_REn_{majors} + 10\_REnR_{minor} + 10\_REnR_{major} + R\_V_{60\text{-}50} + 0.6\_IEx_{majors} + 1.2\_RCr$	0.27	4.50	1.24	1,434,534	394,348	454,938	3,689,955
	$0.6\_REn_{minors} + 1.2\_RW + 0.6\_REn_{majors} + 10\_REnR_{minor} + 10\_REnR_{major} + R\_V_{60\text{-}50} + 0.6\_IEx_{majors} + 1.2\_RCr + 2.4\_ICi$	0.27	4.5	1.22	1,434,534	389,718	456,963	3,706,380
	$0.6\_REn_{minors} + 1.2\_RW + 0.6\_REn_{majors} + 10\_REnR_{minor} + 10\_REnR_{major} + R\_V_{60\text{-}50} + 0.6\_IEx_{majors} + 1.2\_RCr + 2.4\_ICi + 0.6\_IEx_{minors}$	0.27	4.5	1.22	1,434,534	388,858	457,339	**3,709,430**
R_SW2	1.2_REn_{majors}	0.48	2.00	0.96	637,571	306,034	145,002	1,176,093
	$1.2\_REn_{majors} + 10\_REnR_{minors}$	0.36	2.00	0.72	637,571	230,057	178,231	1,445,615
	$1.2\_REn_{majors} + 10\_REnR_{minors} + 1.2\_RW$	0.30	2.00	0.61	637,571	193,821	194,079	1,574,156
	$1.2\_REn_{majors} + 10\_REnR_{minors} + 1.2\_RW + R\_V_{60\text{-}50}$	0.28	2.00	0.56	637,571	177,053	201,413	1,633,639
	$1.2\_REn_{majors} + 10\_REnR_{minors} + 1.2\_RW + R\_V_{60\text{-}50} + 1.2\_IEx_{majors}$	0.26	2.00	0.52	637,571	166,993	205,813	1,669,329

[a] using the average inflation rate 2.5% between 2006-2017

[b] Present value based on the discount rate (r) equal 4%

Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_SW2	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$ + 1.2_IEx$_{majors}$ + 1.2_RCr	0.26	2.00	0.52	637,571	163,039	207,542	1,683,354
	1.2_REn$_{majors}$ + 10_REnR$_{minors}$ + 1.2_RW + R_V$_{60-50}$ + 1.2_IEx$_{majors}$ + 1.2_RCr + 2.4_ICi	0.25	2.00	0.50	637,571	160,320	208,732	**1,693,000**
R_NE1	1.2_RW	0.69	2.00	1.38	637,571	439,924	86,443	701,133
	1.2_RW + 10_REnR$_{minor}$	0.59	2.00	1.18	637,571	377,495	113,747	922,592
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$	0.52	2.00	1.03	637,571	328,506	135,173	1,096,377
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$	0.47	2.00	0.95	637,571	301,803	146,852	1,191,104
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$	0.44	2.00	0.89	637,571	283,644	154,794	1,255,517
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr	0.44	2.00	0.87	637,571	277,431	157,511	1,277,558
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi	0.43	2.00	0.86	637,571	273,401	159,274	1,291,856
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi + A_Fixed	0.42	2.00	0.85	637,571	269,657	160,912	1,305,137
	1.2_RW + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_REn$_{major}$ + 1.2_RCr + 2.4_ICi + A_Fixed + 0.6_IEx$_{major}$	0.42	2.00	0.84	637,571	266,861	162,135	**1,315,057**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_NE4	10_REnR_{minors}	0.50	1.50	0.75	478,178	239,089	104,568	848,144
	10_REnR_{minors} + 20_REnR_{major}	0.39	1.50	0.58	478,178	184,099	128,619	1,043,217
	10_REnR_{minors} + 20_REnR_{major} + 10_IExR_{minor}	0.34	1.50	0.52	478,178	164,643	137,129	1,112,235
	10_REnR_{minors} + 20_REnR_{major} +10_IExR_{minor}+ $R\_V_{60\text{-}50}$	0.31	1.50	0.47	478,178	150,323	143,391	1,163,032
	10_REnR_{minors} + 20_REnR_{major} +10_IExR_{minor}+$R\_V_{60\text{-}50}$ + 0.6_RW	0.29	1.50	0.44	478,178	140,586	147,650	1,197,574
	10_REnR_{minors} + 20_REnR_{major} + 10_IExR_{minor}+$R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_RCr	0.29	1.50	0.44	478,178	138,681	148,483	1,204,331
	10_REnR_{minors} + 20_REnR_{major} + 10_IExR_{minor}+$R\_V_{60\text{-}50}$ + 0.6_RW + 0.6_RCr + 1.2_ICi	0.29	1.50	0.43	478,178	137,808	148,865	**1,207,429**
R_NE7	10_IExR_{minors}	0.62	1.00	0.62	318,785	197,647	52,981	429,726
	10_IExR_{minors} + 10_IExR_{majors}	0.53	1.00	0.53	318,785	167,415	66,203	536,969
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor}	0.45	1.00	0.45	318,785	142,893	76,929	623,961
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor}	0.40	1.00	0.40	318,785	127,085	83,842	680,038
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor} + $R\_V_{60\text{-}50}$	0.37	1.00	0.37	318,785	118,258	87,703	711,350
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor} + $R\_V_{60\text{-}50}$ + 0.6_RW	0.35	1.00	0.35	318,785	112,166	90,368	732,963

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_NE7	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor} + R_V_{60-50} + 0.6_RW + 0.6_RCr	0.35	1.00	0.35	318,785	110,961	90,894	737,235
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor} + R_V_{60-50} + 0.6_RW + 0.6_RCr + 1.2_ICi	0.35	1.00	0.35	318,785	110,403	91,139	739,217
	10_IExR_{minors} + 10_IExR_{majors} + 10_REnR_{minor} + 0.6_REn_{minor} + R_V_{60-50} + 0.6_RW + 0.6_RCr + 1.2_ICi + 0.6_IEx_{minor}	0.35	1.00	0.35	318,785	110,138	91,255	**740,156**
R_SE2	20_IExR_{major}	0.67	1.00	0.67	318,785	213,586	46,010	373,183
	20_IExR_{major} + 1.2_RW	0.57	1.00	0.57	318,785	182,412	59,645	483,772
	20_IExR_{major} + 1.2_RW + R_V_{60-50}	0.52	1.00	0.52	318,785	164,817	67,340	546,186
	20_IExR_{major} + 1.2_RW + R_V_{60-50} + 1.2_RCr	0.50	1.00	0.50	318,785	159,984	69,454	563,332
	20_IExR_{major} + 1.2_RW + R_V_{60-50} + 1.2_RCr + 2.4_ICi	0.49	1.00	0.49	318,785	156,903	70,801	**574,263**
R_SE6	0.6_REn_{minors}	0.53	2.0	1.06	637,571	337,912	131,059	1,063,007
	0.6_REn_{minors} + 1.2_RW	0.46	2.0	0.95	637,571	294,531	150,033	1,216,898
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors}	0.39	2.0	0.78	637,571	247,944	170,408	1,382,161
	0.6_REn_{minors} + 1.2_RW + 0.6_REn_{majors} + 10_REnR_{minor}	0.34	2.0	0.68	637,571	215,880	184,432	1,495,907

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_SE6	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$	0.31	2.0	0.61	637,571	194,603	193,737	1,571,383
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$	0.29	2.0	0.57	637,571	182,439	199,057	1,614,533
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$	0.28	2.0	0.56	637,571	178,425	200,813	1,628,772
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr	0.27	2.0	0.55	637,571	175,266	202,195	1,639,980
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr + 2.4_Ici	0.27	2.0	0.54	637,571	173,208	203,095	1,647,280
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr + 2.4_Ici + A_Fixed	0.27	2.0	0.54	637,571	171,297	203,930	1,654,058
	0.6_REn$_{minors}$ + 1.2_RW + 0.6_REn$_{majors}$ + 10_REnR$_{minor}$ + 10_REnR$_{major}$ + R_V$_{60-50}$ + 0.6_IEx$_{majors}$ + 1.2_RCr + 2.4_Ici + A_Fixed + 0.6_IEx$_{minors}$	0.27	2.0	0.54	637,571	170,941	204,086	**1,655,322**
R_SE13	0.6_REn$_{minors}$	0.53	2.00	1.06	637,571	337,912	131,059	1,063,007
	0.6_REn$_{minors}$ + 1.8_RW	0.42	2.00	0.85	637,571	270,277	160,640	1,302,938
	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$	0.36	2.00	0.72	637,571	228,491	178,916	1,451,169
	0.6_REn$_{minors}$ + 1.8_RW + 0.6_REn$_{majors}$ + R_V$_{60-50}$	0.33	2.00	0.66	637,571	209,252	187,330	1,519,417

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 2 Benefit analysis at roundabouts by treatment type (continue)

Roundabout ID	Suggested Treatments	CMFs	Ave. Crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017) [a]	PV [b] ($AUD)
			Before	After	Before	After		
R_SE13	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major}	0.31	2.00	0.61	637,571	195,400	193,389	1,568,556
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr	0.30	2.00	0.59	637,571	188,412	196,445	1,593,346
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr + 0.6_IEx_{majors}	0.29	2.00	0.58	637,571	183,958	198,393	1,609,144
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr + 0.6_IEx_{majors} + 3.6_ICi	0.28	2.00	0.57	637,571	180,445	199,930	1,621,607
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr + 0.6_IEx_{majors} + 3.6_ICi + A_Fixed	0.28	2.00	0.56	637,571	178,152	200,932	1,629,741
	0.6_REn_{minors} + 1.8_RW + 0.6_REn_{majors} + $R\_V_{60\text{-}50}$ + 10_IExR_{major} + 1.8_RCr + 0.6_IEx_{majors} + 3.6_ICi + A_Fixed + 0.6_IEx_{minors}	0.28	2.00	0.56	637,571	177,725	201,119	**1,631,255**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Benefit Analysis for Roadway Segment Treatments

Table D. 3 Benefit analysis at road segments by treatment type

Segment ID	Suggested Treatments	CMFs	Ave. crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017)[a]	PV [b] ($AUD)
			Before	After	Before	After		
R_NW7	R_V_{60-50}	0.67	5.00	3.35	1,593,927	1,067,931	230051	1,865,917
	R_V_{60-50} + AMI	0.57	5.00	2.84	1,593,927	905,616	301041	2,441,713
	R_V_{60-50} + AMI + 0.6_ILW_4	0.49	5.00	2.46	1,593,927	783,702	354362	2,874,191
	R_V_{60-50} + AMI + 0.6_ILW_4 + AEL	0.47	5.00	2.38	1,593,927	759,898	364773	**2,958,634**
S_NE8	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V_{60-50} + AEL	0.67	3.00	2.00	956,356	636,136	140,052	1,135,948
	R_V_{60-50} + AEL + 1.0_ISW	0.64	3.00	1.93	956,356	616,617	148,589	**1,205,190**
S_NW21	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V_{60-50} + 0.5_ILW_4	0.60	3.00	1.80	956,356	574,292	167,100	**1,355,334**
S_SW4	R_V_{60-50}	0.67	4.50	3.02	1,434,534	961,138	207,046	1,679,325
	R_V_{60-50} + 0.4_ILW_2	0.66	4.50	2.96	1,434,534	942,608	215,150	**1,745,057**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 3 Benefit analysis at road segments by treatment type (continue)

Segment ID	Suggested Treatments	CMFs	Ave. crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017)[a]	PV [b] ($AUD)
			Before	After	Before	After		
S_SW16	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V_{60-50} + AEL	0.67	3.00	2.00	956,356	636,136	140,052	1,135,948
	R_V_{60-50} + AEL + 1.0_ISW	0.64	3.00	1.93	956,356	616,617	148,589	**1,205,190**
S_SE9	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V_{60-50} + AMI	0.57	3.00	1.70	956,356	543,370	180,625	1,465,028
	R_V_{60-50} + AMI + 1.5_ISW	0.54	3.00	1.63	956,356	519,858	190,908	1,548,434
	R_V_{60-50} + AMI + 1.5_ISW + AEL	0.52	3.00	1.57	956,356	501,859	198,780	**1,612,282**
S_SE5	R_V_{60-50}	0.67	2.50	1.68	796,963	533,965	115,025	932,959
	R_V60-50 + AEL	0.67	2.50	1.66	796,963	530,113	116,710	946,623
	R_V60-50 + AEL + 1.0_ISW	0.64	2.50	1.61	796,963	513,847	123,824	**1,004,325**
S_SW8	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V60-50 + AMI	0.57	3.00	1.70	956,356	543,370	180,625	**1,465,028**

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding

Table D. 3 Benefit analysis at road segments by treatment type (continue)

Segment ID	Suggested Treatments	CMFs	Ave. crashes/ 3year		Crashes cost/ 3year		Cost saved /year(2017)[a]	PV [b] ($AUD)
			Before	After	Before	After		
S_SW19	1_$L_{dire.}$	0.34	3.00	1.02	956,356	325,161	276,061	2,239,101
	1_$L_{dire.}$ + R_V60-50	0.31	3.00	0.92	956,356	293,601	289,864	2,351,056
	1_$L_{dire.}$ + R_V60-50 + AMI	0.25	3.00	0.75	956,356	239,051	313,722	2,544,568
	1_$L_{dire.}$ + R_V60-50 + AMI + 0.5_ISW	0.25	3.00	0.74	956,356	235,406	315,316	2,557,498
S_SE9	R_V_{60-50}	0.67	3.00	2.01	956,356	640,759	138,030	1,119,550
	R_V60-50 + 0.6_ILW_4	0.60	3.00	1.79	956,356	569,988	168,983	1,370,601
	R_V60-50 + 0.6_ILW_4 + 1.5_ISW	0.57	3.00	1.72	956,356	547,308	178,902	1,451,056

[a] using the average inflation rate 2.5% between 2006-2017
[b] Present value based on the discount rate (r) equal 4%
Note: The calculations were performed without rounding