Crash prediction models for signalised intersections: signal phasing and geometry

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An important note for the reader

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Abbreviations and acronyms

AADT	annual average daily traffic
ARRB	Australian Road Reserve Board
BIC	Bayesian Information Criterion
IDM	Intersection Diagnostic Monitor
MV	motor vehicle
RENB	Random Effect Negative Binominal
SCATS®	Sydney Coordinated Adaptive Traffic System
SPF	safety performance function
TRL	Transport Research Laboratory
Crash codes	
C and D	loss-of-control crash
F	rear-end crash
HA crash	right-angle crash
LB crash	right-turn-against crash

Contents

Executive summary

The majority of the major intersections in urban areas have signalised control. In most cities the majority of crash black-spots occur at major intersections. While crash reduction studies often focus on the major signalised intersections, there is little information that links the phasing configuration, degree of saturation of each movement and overall cycle time to crashes.

This research attempted to quantify the effect of signal phasing on various crash types and various travel modes at traffic signals, taking into account speed limits, intersection geometry and land-use environment. The research objectives were to develop:

- crash prediction models for traffic signals in New Zealand
- a safety toolkit that could be used by transport engineers to predict the expected number of crashes at new and upgraded traffic signal sites.

Literature review

As there is only limited research in New Zealand on the safety implications of treatments at signalised intersections, a literature review was carried out on international research. The literature review considered key geometric, traffic and operational features of traffic signals, along with their effect on specific crash types such as red-light running, rear-end crashes, pedestrian–vehicle and bicycle–vehicle crashes, and right-turn-against crashes.

Overall, there appeared to be a paucity of data on the impacts of geometric changes at signalised intersections. The research available on the safety implications of phasing sequences was even more limited, although the research did show that increasing the number of signal phases and having more complicated phasing tended to increase crash rates. While the studies used a variety of methods, such as crash models, crash modification factors and analysis of historical crash trends, a holistic assessment of the various factors affecting safety at signalised intersections was missing.

In the various studies considered, the key factors that were shown to improve safety were extra all-red and yellow time, full right-turn phase protection, exclusive turning lanes, and gap termination of phases (for red-light-running crashes).

Site selection and data collection

A total of 238 low- and high-speed signalised intersections from Auckland, Wellington, Hamilton, Christchurch, Dunedin and Melbourne were selected for this study. These included both three-arm and four-arm intersections. Data collection on a wide range of physical and operational characteristics of signalised intersections was collected for these sites. This included intersection layout and geometry, signal phasing and coordination, road user counts (motor vehicles, pedestrians and cyclists), signal displays and crashes, among others. Automated methods that allowed analysis of the large amount of SCATS® data were also developed to determine signal operation parameters, including type of phasing, degree of saturation, frequency of pedestrian phase activation, signal cycle times, and green, yellow and all-red phase times. Data was collected prior to the changes to the New Zealand give-way rules that were implemented on 25 March 2012.

Data analysis

In addition to the above, the degree of saturation and pedestrian usage at the intersection was also estimated. Degree of saturation for the selected approaches was calculated using adjusted SCATS® traffic volumes and SCATS® signal-timing information, in conjunction with number of lanes and an assumed lane capacity. Pedestrian usage at the selected intersections was estimated through categorisation into five 'bins' (namely low, medium-low, medium, medium-high and high), using data available from SCATS® regarding the occurrence of pedestrian phases.

Crash prediction models

Crash prediction models were developed for the main crash types involving motor vehicles and pedestrians. For motor vehicles, these included rear-end, right-turn-against, right-angle and loss-of-control crashes. For pedestrians, these were right-angle and right-turning crashes involving a motor vehicle colliding with a pedestrian.

The table below shows the impacts of various intersections' physical and operational parameters on the key motor vehicle crash types. Cells shaded red indicate an increase in crashes due to the parameter, while those shaded green indicate a reduction in crashes.

Table 1 Effect of intersection parameters on motor vehicle crashes (see table 13.1 in this report for a more detailed table with explanatory notes for all models developed in this study)

Parameter	Right-angle	Right-turn-against	Loss-of-control	Rear-end crashes		
				Small intersections	Medium intersections	Large intersections
Higher traffic volumes	⇧	⇧	⇧	⇧	⇧	⇧
Higher degree of saturation		⇧	⇧			
Larger intersections	⇧					
More approach lanes			⇧		⇧	⇧
More through lanes		⇧				
Longer cycle time	⇩	⇩	⇩			
Longer all-red time	⇩					
Longer lost time (all-red + inter-green)				⇩	⇧	⇩
Full right-turn protection		⇩				
Split phasing	⇩		⇧	⇧	⇧	⇩
Mast arm	⇩					
Coordinated signals	⇧					
Additional advanced detector loops	⇧					
Shared turn lanes (eg right-turn/through and left-turn/through)	⇧					
Shared right-turn/through turn lane		⇩				
Raised median/central island	⇩	⇧				
Length of right-turn bay/lane		⇩		⇩		⇩
Free left turn for motor vehicles			⇧	⇧	⇧	⇧
Exit merge			⇧			
Cycle facilities		⇧		⇩	⇩	⇧
Upstream bus bay within 100m			⇧	⇧	⇩	
Upstream parking			⇩			
High speed limit (>=80kph)			⇧		⇧	

Legend

⇧	Increase in crashes due to the parameter
⇩	Decrease in crashes due to the parameter
	No identified change in crash rate due to the parameter

A number of intersection parameters, such as all-red time, shared turns and signal coordination, were observed to affect a specific crash type. However, the model results also highlighted the safety benefits obtained from longer cycle times and longer right-turning bays across multiple crash types. On the other hand, free left turns for motor vehicles, more approach lanes and near-saturated or over-saturated intersections were found to increase the risk of having a crash.

Phasing sequences also figured prominently in the models. Presence of full-right-turn protection reduced right-turn-against crashes. Split phasing sequences led to a reduction in right-angle crashes and rear-end crashes at larger intersections (those with three or more approach lanes, and an intersection depth of 40m or greater), but an increase in loss-of-control crashes, other crashes and rear-end crashes at small (one or two approach lanes, and an intersection depth of less than 25m) and medium intersections (all those not covered in the previous categories). The sites with advanced detectors had high numbers of crashes, a counterintuitive result that should be treated with caution. Additional analysis in the form of before-and-after studies is required to assess the safety offered by these loops.

In addition to the models shown in the table above, a combined Auckland-and-Melbourne model was developed for right-angle crashes, while peak-period models were built for right-angle, right-turn-against and rear-end crashes. Coordinated signals showed mixed trends in Auckland and Melbourne (fewer right-angle crashes) as compared with all cities together, where they were associated with more right-angle crashes. This could have been an outcome of drivers in larger cities being used to driving along coordinated corridors.

The presence of shared turns (ie both shared left-turn/through or right-turn/through lanes) had mixed effects, with an increase in right-angle crashes for all cities taken together and in peak periods, but a reduction at the Auckland and Melbourne sites.

Pedestrian–motor vehicle crashes

The models confirmed the ‘safety in numbers’ effect, whereby crashes increase at a decreasing rate as pedestrian numbers increase.

The model results also suggested that the increase in right-angle pedestrian–vehicle crashes because of longer cycle times was greater than the corresponding decrease in crashes involving a right-turning vehicle colliding with crossing pedestrians. This was in contrast to previous Transport Research Laboratory (TRL) research, which indicated that longer cycle times were safer for pedestrians. Longer lost times (in the form of either yellow or all-red times) negatively affected both crash types. In addition, full signal protection for right-turning vehicles reduced right-turning pedestrian–vehicle crashes, while a split phasing sequence lowered right-angles.

Right-angle crashes were negatively affected by the presence of shared turns and cycle facilities, but positively affected by the presence of a raised median or central island. Coordinated signals had a higher number of right-turning crash rates.

Peak-period crashes

The results for peak periods indicated that longer cycle and all-red times, and the presence of split phasing and mast arms, had a significant effect on improving the safety of right-angle turns. The presence of a raised median or central island was observed to increase right-angle and right-turn-against crashes in the peaks, as opposed to the whole day where the presence of these features improved the incidence of this crash type.

Rear-end crashes were particularly prevalent during AM and PM peaks. Peak-period traffic volume was a key factor for this crash type at large intersections. The presence of cycle facilities and free left turns for motor vehicles at large intersections reduced the incidence of rear-end crashes during peaks, as opposed to the increase that was observed for the whole day at these intersections. The model for medium-sized intersections showed similar trends for both the peak and all-day periods.

While the models showed that longer right-turn bays or lanes resulted in a reduction in right-turn-against crashes during the whole day, the effect was the opposite in the peaks. This could possibly be an outcome of the fact that sites with higher right-turning traffic volumes are often provided with longer right-turn bays.

Future research

The research team identified the need for more comprehensive and better-quality data collection for cyclists. Well-fitting models for cyclists could not be developed as part of this study, due to the lack of adequate data.

Data from 102 signalised intersections in Adelaide is already available as part of research conducted for Austroads. Collection of signal-phasing data for these sites would enable a more comprehensive database to be built (especially for cycle–vehicle crashes), which can be drawn upon for future studies.

It is also suggested that careful before-and-after monitoring of the effects of certain intersection improvements should be undertaken. The models have highlighted a number of intersection parameters that have mixed effects on safety, and further research into these aspects is required.

Abstract

In most cities and towns, the majority of crash black-spots occur at major intersections. Given this, crash reduction studies often focus on the major signalised intersections. However, there is limited information that links the phasing configuration, degree of saturation and overall cycle time to crashes. While a number of analysis tools are available for assessing the efficiency of intersections, there are very few tools that can assist engineers in assessing the safety effects of intersection upgrades and new intersections. Data from 238 signalised intersection sites in Auckland, Wellington, Christchurch, Hamilton, Dunedin and Melbourne were used to develop crash prediction models for key crash-causing movements at traffic signals. Separate models were built for peak periods and for motor vehicles and pedestrians. The key crash types that were analysed were right-angle, right-turning, lost-control and rear-end type crashes.

1 Introduction

1.1 Background

The majority of the bigger intersections in our urban areas have signalised controls. In most cities the majority of crash black-spots occur at major intersections. While crash reduction studies often focus on the major signalised controlled intersections, there is little information that links the phasing configuration of signals, degree of saturation of each movement, and overall cycle time to crashes. Most changes to the signal phasing, other than right-turning phases, occur for efficiency reasons. Safety improvements tend to focus on other factors such as conspicuousness of the signal displays, the amount of inter-green time, and the skid resistance of the pavement.

While there are a multitude of tools available for assessing the efficiency of intersections, there are very few tools that can help engineers assess the safety effects of intersection upgrades and new intersections – for example, we could not find any tools that allowed the prediction of crashes based on signal phasing. Hence traffic signal engineers have to assess the safety consequences of their decisions based on their general engineering skills and experience, and site-specific crash data and limited research.

1.2 Purpose of the research

This research attempted to quantify the effect that signal phasing has on various crash types and various travel modes at traffic signals, taking into account the speed limits (and where available, operating speeds), the intersection geometry, and the land-use environment (be it industrial, commercial – eg shopping, or residential) or a combination of these influences. Factors such as horizontal and vertical approach alignment were also factored into the evaluation, along with the duration and configuration of the lost time between signal phasing. This type of study would enable the trigger points at which traffic delays and signal cycle length started to create safety problems to be determined.

The research looked at crashes involving pedestrians, who are over-represented in the crash statistics. Recent research has established that there is a 'safety in numbers' effect at locations with high volumes of pedestrians and cyclists. However, pedestrian and cycle safety is compromised at high traffic-volume sites (links or intersections) where pedestrian and cycle volumes are low, as commonly occurs in the suburbs. As traffic volumes increase, safety is further compromised as drivers are busy focusing on other drivers and may miss seeing the 'smaller' pedestrian or cyclist.

1.3 Research objectives

The purpose of this research was to quantify the safety impact (in terms of crashes) of various traffic signal-phasing configurations and levels of intersection congestion (measured by degree of saturation) at low- and high-speed traffic signals in New Zealand and Australia.

The research objectives were to develop:

- crash prediction models for traffic signals in New Zealand
- a safety toolkit for traffic signals, which could be used by transport engineers to predict the expected number of crashes at new and upgraded sites.

This latter issue needs to be addressed, as road safety engineers have anecdotal experience that signal phasing and traffic congestion (and the resulting driver frustration) have an effect on road safety, but the effect has not been adequately quantified. While there is some information from before-and-after studies on the effects of various intersection features, much of this research has been undertaken in other countries, such as the UK and US, where signalised intersections have a distinctly different layout (eg signal displays are located quite differently to the way they are in New Zealand), and in most studies, the interaction between the various layout features and signal phasing has not been examined by using a multivariate crash models framework.

The outcome of this research will be a tool that can assist traffic signal engineers in assessing the safety consequences of their decisions. Although this tool will undoubtedly need to be refined as more research is undertaken, and there will still be a role for subjective opinion, it will still be a useful decision-making tool for engineers.

1.4 Structure of this report

This report contains the following sections:

- **Section 2** presents the results of an international literature review
- **Section 3** details the sample selection process and lists the final selected intersections
- **Section 4** describes the various parameters for which data collection was undertaken
- **Section 5** presents the distribution of parameters for which data was collected
- **Section 6** presents outcomes of the analysis of SCATS® signal operation data
- **Section 7** presents an analysis of crash data at the selected intersections
- **Section 8** analyses the impact of the key physical and operation characteristics of signals on safety
- **Section 9** introduces the crash modelling methodology and lists the various models developed
- **Section 10** presents crash prediction models developed for motor vehicle crashes
- **Section 11** presents crash prediction models developed for crashes involving pedestrians
- **Section 12** presents the way the prediction models can be applied
- **Section 13** summarises the key results from this study.

2 Literature review

2.1 Introduction

As there is only limited research in New Zealand on the safety implications of treatments at signalised intersections, a literature review was carried out on research conducted across the industrialised world. The UK, US and Singapore, in particular, are the source of a great many studies on the performance of signals. From this body of research, studies were selected to look at crash prediction models, the effectiveness of various signalised intersection countermeasures, and the variables that impact crash rates at these intersections.

2.2 Background

Intersections make up only a small portion of a country's total road distance, but they are the location of a disproportionate number of crashes. In order to combat this and improve signalised intersections overall, a wide variety of countermeasures have been employed both in New Zealand and abroad. Some countermeasures improve capacity; some improve traffic operations at the intersection or along a corridor; while still others improve the safety of certain user groups (eg pedestrians, right-turning traffic, potential red-light runners, etc).

However, there has been little research looking at the overall impacts of each of these countermeasures on the safety of all users. Countermeasures that improve the service or safety for one user group may have negative impacts on the service and safety of other user groups. This literature review sought to summarise the existing research into treatments for vehicles, cyclists and pedestrians, and to identify the datasets that those safety studies used and that should be collected in Australia and New Zealand to carry out similar research here.

2.3 Relevance to New Zealand

Much of the literature reviewed originated overseas, where driving habits and design standards may differ from those employed in New Zealand. In particular, the following key differences should be kept in mind while relating overseas research in a New Zealand setting:

- right-side versus left-side driving (in Canadian and US research)
- the use of overhead-mounted signal heads versus side/pole-mounted signal heads
- left-turn (right-turn in the US) treatments – turn-on-red (in the US), slip lanes, leading pedestrian interval
- the traffic control basis – SCATS®, SCOOT, pre-timed.

2.4 Major crash types occurring at signalised intersections

Some of the earliest crash prediction research was carried out by Transport Research Laboratory (TRL) in the UK. Hall (1986) analysed four years of crash data (1979–1982) at 177 four-leg urban intersections on 30m/hr roads throughout the UK. The report separated intersections into eight groups, based on the presence (or lack thereof) of Urban Traffic Control, pedestrian stages and right-turn stages (or more or less than two stages).

Hall used a generalised regression model and assumed a Poisson distribution, resulting in models of the form:

$$A = k \times QT^{\alpha} \times (c + PT^{\beta}) \quad \text{(Equation 2.1)}$$

Where:

A = crash (crash) frequency (annual)

QT = total vehicular flow

Α = coefficient of vehicular flow

c = constant (usually close to or equal to 0, but can be increased to account for zero pedestrians)

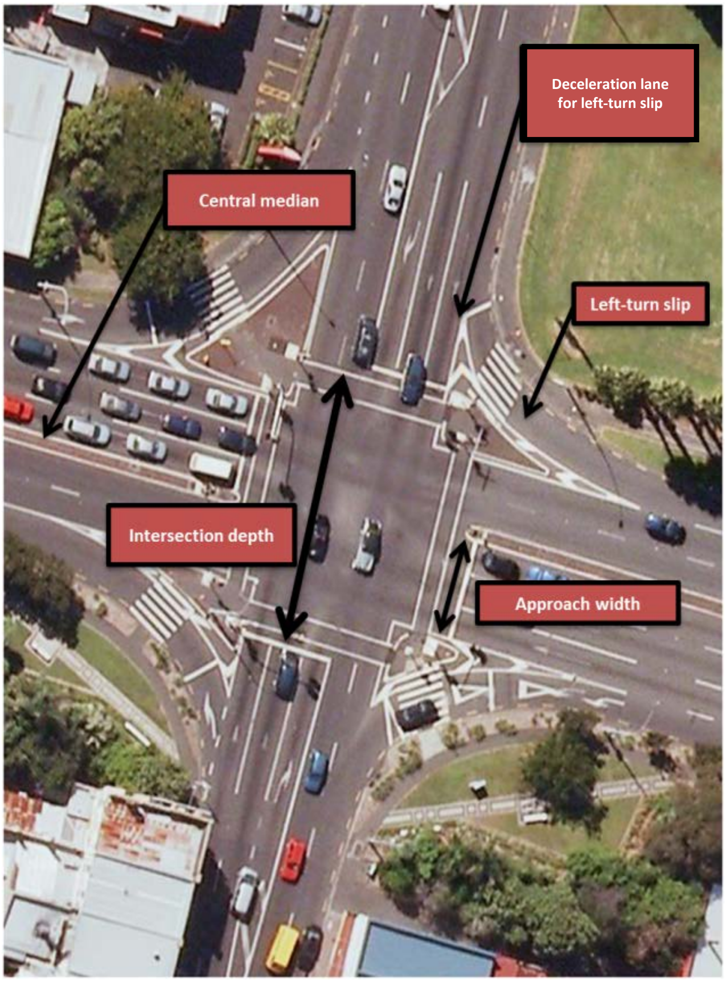
PT = total pedestrian flow

Β = coefficient of pedestrian flow.

Hall derived significant crash prediction models for total crashes, vehicle-only crashes, pedestrian–vehicle crashes, and 11 specific types of crashes. The best-fitting models (and the simplest) were functions of all 12 vehicular flows into the intersection (3 movements on each leg) and the total vehicular and pedestrian flows.

Hall further tested out geometric variables at the intersections and found significant models correlating crashes with approach width, number of approach lanes, approach horizontal curvature, sight distance and gradient on the approach, horizontal displacement across the intersection (eg approaches that were not exactly opposite one another), the angle of intersecting roadways, yellow box ‘no stopping’ markings, the position of the secondary signal, and the presence of a pedestrian refuge island. Operational variables that had a significant correlation with crashes included the sequencing of the right turn (leading vs lagging), the number of stages, the length of the cycle time, the degree of saturation, the inter-green time, and the presence of a pedestrian stage.

Figure 2.1 Various intersection geometric elements



Then, in the US, Poch and Mannering (1996) carried out similar research on 63 intersections in Bellevue, Washington, where intersection improvements had been carried out between 1987 and 1993. Not all of these intersections were signalised. Poch and Mannering used a negative binomial model to correlate crashes with intersection variables. Significant variables at the signalised intersections included the number of phases (eg whether left turns – or right turns in New Zealand – were given their own phase), protection of left turns (right turns in New Zealand), restricted sight distance, approach gradient, horizontal curvature, and the approach speed limit. Interestingly, Poch and Mannering also found an increase in the crash rate when the approach had two or more lanes and a shared left-through lane because:

> *(1) Left-turning vehicles that must stop and wait for a gap to complete the manoeuvre cause a high potential for rear-end crashes as through vehicles approach in the same lane at prevailing speed; and (2) stopped left-turning vehicles that face stopped left-turning vehicles in the opposing approach must overcome the sight restriction to the opposing through vehicles to successfully complete the manoeuvre* (Poch and Mannering 1996).

This arrangement (or rather, the combined right-turn/through lane) was used in a number of locations in New Zealand prior to the 2012 give-way priority law change, as a means to make yielding left-turning vehicles more visible to right-turning traffic.

Kumara and Chin (2005) evaluated signalised intersections in Singapore, which like New Zealand, has left-side driving. Singapore is notable in that its road network and driver behaviour is not too dissimilar to New Zealand and it has been the site of a lot of signalised intersection research, presumably due to relatively easily accessible crash data. Kumara and Chin's research used a modified Poisson underreporting model on a sample size of 104 three-legged intersections with nine years of crash data to identify crash causal factors and take into account the traditional underreporting of crashes to the police. This report took Kumara's previous work (Kumara et al 2003), which had used a random-effect negative binomial model, and emphasised correction of the underreporting of crashes. Variables considered in both studies are listed in table 2.1; those variables without an estimated crash relationship were not considered to be statistically significant. While similar variables are difficult to compare across models, due to the unknown structure of each model, the table is useful to highlight the relative impact of each variable: those with negative coefficients correlate with decreased crash rates, while positive coefficients correlate with increased crash rates and large coefficients have correspondingly larger impacts.

Table 2.1 Crash relationship variables in Singapore (Chin and Quddus 2003, Kumara et al 2003 and 2005)

Variable	Estimated coefficient RENB[a] (Chin and Quddus 2003)	Estimated coefficient RENB (Kumara 2003)	Estimated coefficient underreporting (Kumara 2005)
Total approach volume (AADT[b])	0.0071	0.0001	0.6310
Total left-turn volume (AADT)		0.0001	0.1843
Right-turn volume (AADT)	0.0101		
Short sight distance (<100m)	0.0006	0.4377	0.1303
Long sight distance (>300m)			0.1974
Length of left-turn slip road (m)			
Number of approach lanes			
Median width (m)	0.1947		
Gradient greater than +5%		-0.3140	-0.4642
Tight horizontal curve of radius (<100m)		0.3175	
Right turn channelisation			-0.4983
Provision of left-turn slip road	0.3052	0.2799	0.1837
Acceleration lane for left-turn slip road	-0.2783		-0.3695
Number of signal phases per cycle	0.1108	0.3600	
Unprotected filtered right-turn phase		0.6473	0.4985
Provision of adaptive signal control	-0.0522		
Surveillance camera	0.2438		-0.1897
Median railings			-0.1466
Provision of bus stops	0.0556		
Provision of bus bays	-0.0492		
Obtuse approach angle		-0.3052	

a) random effect negative binomial

b) annual average daily traffic

Kumara and Chin specifically highlighted that unprotected left-turn slip roads, the number of signal phases per cycle, the use of permissive right-turning phases, and restricted sight distances of less than 100m (or greater than 300m) are variables that increase crash rates, while right-turn channelisation, left-turning acceleration lanes, obvious camera surveillance, anti-pedestrian median railing, obtuse intersection angles, and approach gradients greater than 5% reduce crash rates. The report expressed some surprise at the reduction in crashes from uphill approaches, noting that ‘an uphill grade into an intersection may lead to reduced vehicle speeds, while obtuse angles require reduced turning speeds in order to navigate right turns’.

Mitra et al (2002) also looked into crashes at signalised intersections in Singapore, specifically at side-impact and rear-end crashes, which account for 84% of all crashes at four-legged intersections in Singapore. Their research then estimated crash prediction through zero-inflated probability models to account for data from intersections during intervals where there were no recorded crashes. They found that closely adjacent intersections and bus bays decreased the rate of side-impact crashes, whereas greater sight distance, the presence of pedestrian refuge islands, and higher approach speeds increased the rate. Rear-end crashes appeared to decrease with adaptive signal control and increase with camera surveillance (directly the opposite of Kumara and Chin’s findings with CCTV cameras, which concluded that drivers may behave more cautiously when they are aware they are under surveillance). Crashes of all kinds increased with the presence of uncontrolled channelised left turns, wider medians, higher approach volumes, and an increase in the number of signal phases.

Chin and Quddus (2003) used a random-effect negative binomial model to simulate the relationship between crash occurrence and geometric, traffic, and operations characteristics of Singapore intersections. The significant variables are also displayed above in table 2.1. It is interesting to note that the presence of bus stops leads to an increase in crashes while the presence of bus bays leads to a decrease. The latest design guidelines for bus stops are trending away from the construction of bus bays (except in bus lanes), due to operational concerns of bus drivers.

Ogden et al (1994) analysed 76 sites in Victoria, Australia, to determine the characteristics that were to be found at sites with higher-than-expected crash rates. Considering the traffic flow at the 76 sites, the expected crash rate was (1.079 + 0.052 × the flow rate), which is a linear relationship that is not constrained by having to go through the origin (zero flow should equal zero crashes). Based on this expected crash rate, which was based entirely on traffic volume, high-crash-rate sites (ie 1 crash annually above the expected rate) and low-crash-rate sites (ie 1 crash annually below the expected rate) were separated and analysed. Significant results from this analysis included the following:

- There was a clear tendency for sites with exclusive right-turn lanes to have lower crash rates than sites with shared right-turn/through lanes.
- The presence of a median greater than 0.9m in width led to lower crash rates, and wider median widths were safer still.
- There was no discernable impact from the presence of clearways on crash rates.
- The presence of gantry-mounted signal displays (discussed later) led to higher crash rates. This could be explained by VicRoads’ programme of installing gantries at high crash-rate locations.
- Interestingly enough, sites with high crash rates tended to have protected-only control for right turns and low crash-rate sites tended to have filtered-only control for right turns.

Ogden et al’s study lacked some of the statistical rigour found in other research, but it did cover some unusual intersection features. However, its conclusions should be considered in light of this lack of rigour, as each site characteristic was not evaluated alone (discounting other characteristics) or against a control group.

Past research in New Zealand into crash prediction models, summarised in Roozenburg and Turner (2004), has concentrated on using intersection and turning volumes as the basis for the models. These models have been incorporated into the economic evaluation processes of the NZTA *Economic evaluation manual.*

Total crash rates were correlated to the total intersection volume, while specific crash types were modelled against the movements in conflict for each type, using five years of crash data from across New Zealand.

At signalised crossroads, Roozenburg and Turner (2004) found that all crash types per vehicle decreased with increasing conflicting flows save rear-end crashes, which increased with increased traffic volumes through an intersection. Data on three-leg intersections showed similar trends for rear-end, loss-of-control, and the catchall 'others' crashes, but there were conflicting conclusions for right-turn-against and crossing crashes, which the report's authors felt required further research.

These models were further refined with the addition of the following non-volume variables to help quantify right-turn-phasing impacts:

- number of opposing through lanes
- right-turn bay offset
- intersection depth
- right-turn signal phasing (eg filtered turns or protected turns)
- visibility to opposing traffic.

However, only the number of opposing through lanes was deemed to improve the above models. A somewhat limited dataset may have limited some of the variables' influences.

2.4.1 Red-light running and rear-end crashes

Red-light running has been an area of concern recently, and there is a large body of research into the causes of red-light running as well as the impacts (both positive and negative) of the chief countermeasure – red-light cameras. As with other areas, most of the research has been carried out in the US and Europe, with a few studies in Australia as well.

Aeron-Thomas and Hess (2005) summarised research into the impacts of cameras across studies carried out in the US, Singapore, Australia, the UK and Norway. They compiled data from a wide variety of databases, including Australian Road Research Board (ARRB) and the Australian Transport Safety Bureau, seeking out appropriate studies that met their criteria in both before-and-after and randomised controlled trials. Ultimately, 10 studies were selected: 6 American studies, 3 Australian studies, and 1 Singaporean study. From these 10 studies, they concluded that while red-light cameras were proven effective in reducing total casualty crashes, there was uncertain evidence as to their impacts on crash rates for side-impact or rear-end crashes. Their report had a very rigorous procedure to evaluate the studies considered, attempting to account for regression to the mean and spill-over; hence the small number of studies and difficulty in drawing a conclusion about specific crash types. Regression to the mean occurs when the sites that are studied have had their cameras installed due to abnormally high crash rates that would drop for reasons other than having the camera there, thus overstating their impact. Spill-over occurs when intersections in close vicinity to camera-enforced intersections are used as control intersections, when in fact driver behaviour at these intersections is still impacted by the surrounding cameras (although the impacts of spill-over are still argued by some researchers to be minimal at best – see the literature review in Kloeden et al 2009).

Retting et al (2007) conducted a study of the impacts of red-light cameras *and* longer yellow periods in Philadelphia, and this study was not reviewed in the earlier summaries. This study looked at two sites in Philadelphia, Pennsylvania, compared against four control sites located far apart in New Jersey, discounting the effects of spill-over. However, Retting et al did not discuss any means to eliminate regression to the mean, although an allusion to the work by Aeron-Thomas and Hess was made. Retting et al found a 36% decrease in red-light running (but no discussion of the change in crash rates) with the implementation of an additional second of yellow time, and a further 96% reduction in red-light running after the installation of red-light cameras, compared with the control intersections.

A notable and more regionalised study was carried out by Kloeden et al (2009) covering 39 sites in Adelaide, Australia. Kloeden et al did a follow-up report on an initial set of red-light cameras installed in Adelaide in 1988 at 15 sites, and a further 24 sites installed in 2001. The report's authors did not take any statistical steps to eliminate regression to the mean, but discounted its effects due to what they perceived as not-abnormally high pre-installation crash rates. The original set of cameras installed in 1988 showed no statistically significant change in overall crash numbers or for crash types. However, casualty crash rates decreased by 21% for all crashes and 49% for side-impact crashes. The newer set of cameras was only observed for one year post installation and no significant impact on crash rates of any kind was discerned.

Further Australian research into red-light running, specifically by heavy vehicles, was carried out by Archer and Young (2009). This research looked at five signal treatments proposed for a signalised 3-leg 'seagull' intersection in suburban Melbourne, Australia – one direction on the primary roadway is uncontrolled and right-turns-in and -out are channelised through the median. The five treatments at the signal were:

- 1-second increase in the yellow time
- an extension of the green time upon detection of a heavy vehicle within the dilemma zone
- an extension of the all-red time upon detection of any vehicle 80m upstream from the intersection travelling at 75km/hr or more
- as for the third treatment, but for any vehicle 35m upstream from the intersection travelling at 45km/hr or faster
- a combination of extensions to green time and all-red time.

The five treatments were simulated in a VISSIM microsimulation model, so the concept has not been field-tested as yet. The simulation showed that the extension of yellow time reduced red-light running by the greatest amount, although this solution does make the signal less efficient and has the potential to encourage drivers to encroach further into the inter-green period after they adapt to a longer yellow time. The report's authors recommended the last treatment instead, and a field trial of the treatment will be undertaken by VicRoads in the near future.

2.4.2 Cyclist–motor vehicle crashes

Roozenberg and Turner (2004) developed two models for cyclist–motor vehicle conflicts: one for cyclists travelling parallel to the flow of traffic and crashing with stationary or parallel moving vehicles, and one for cyclists crashing with turning vehicles. Both models had a pronounced 'safety in numbers' effect in that as the number of cyclists increased, the crash rate per cyclist decreased.

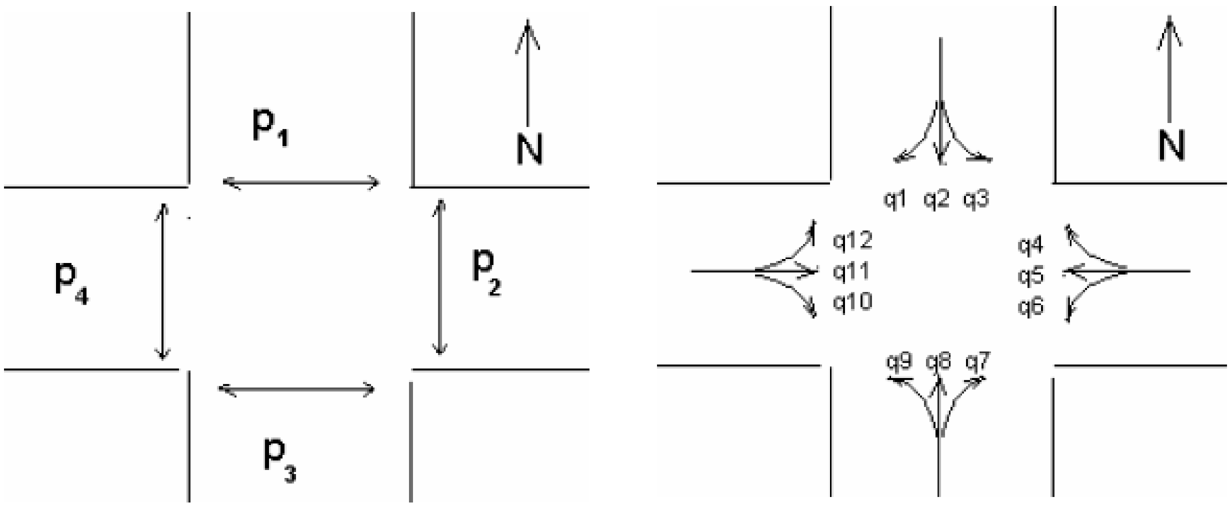
Turner et al's following research (2009) added a variable for cycle lanes and found that their presence increased the crash rate for parallel crashes. This crash model was developed over only 21 intersections, and further research is currently being carried out in this area across both Australia and New Zealand.

2.4.3 Pedestrian–motor vehicle crashes

As with the cyclist models, Roozenburg and Turner (2004) developed models for pedestrians crossing perpendicular to the through flow of traffic (which represent 50% of pedestrian–vehicle crashes at signals) and pedestrians conflicting with right-turning traffic (which represent a further 36%). The crash rate per pedestrian decreased with a higher number of pedestrians and conflicting vehicles, but not to the same degree as with cyclists.

Further research by Turner et al (2006) refined these crash prediction models to be calibrated using specific vehicular and pedestrian flows at intersections. Based on the following flows, models were constructed for pedestrian–vehicle crash rates with through-travelling cars and with right-turning cars.

Figure 2.2 Pedestrian and traffic movements



The total number of crashes involving vehicles travelling straight through an intersection and colliding with a pedestrian who was crossing at a right angle was found to be:

$$A_{UPXT1} = 7.28 \times 10^{-6} \left[(Q_{1,3}^{0.634} (p_1 + p_3)^{0.396} + (Q_{2,4}^{0.634} (p_2 + p_4)^{0.396} \right] \quad \text{(Equation 2.2)}$$

Where $Q_{1,3}$ is the total two-way daily vehicular flow for the north–south direction and $Q_{2,4}$ is the total two-way daily vehicular flow for the east–west direction. The second model for right-turning vehicles conflicting with pedestrians crossing parallel to the road was found to be:

$$A_{UPXT3} = 5.43 \times 10^{-5} \left[(q_1^{0.644}\, p_4^{0.513}) + (q_4^{0.644}\, p_1^{0.513}) + (q_7^{0.644}\, p_2^{0.513}) + (q_{10}^{0.644}\, p_3^{0.513}) \right] \quad \text{(Equation 2.3)}$$

These models came from crash data produced by the Ministry of Transport's Crash Analysis System.

2.4.4 Right-turn-against crashes

Right turns (left turns in the US) are probably one of the most studied conflicts at signalised intersections, with a wide variety of countermeasures to improve safety resulting from these many studies. An early report by Bui et al (1991) looked at 217 approaches at 129 intersections through Victoria, Australia where some change had occurred to the right-turn control – either a change from no control to fully protected control, from no control to protected/filtered control, or protected/filtered control to fully protected control. Of the 217 selected approaches, 135 had a fully protected control installed. The before-and-after study found no statistically significant changes to the crash rate with the installation of partially protected/filtered control. Installation of fully protected control led to a reduction of all crashes by 45%, a reduction of right-turning/through crashes by 82%, a reduction of cross-traffic crashes by 48%, and a reduction of pedestrian–vehicle crashes by 35%, although some of this improvement could be attributed to ancillary roadworks carried out during the installation of the signal control. At the same time, there was a rise of 72% in rear-end and left-rear crashes, although the report does not go so far as to attribute this increase completely to the full control of the right turns. Conversion of partially protected/filtered turns to fully protected turns led to a decrease in the total crash rate of 65%, a 93% reduction in right-turn/through crashes, and a 51% reduction of cross-traffic crashes.

More recently, Kloeden et al (2007) analysed right-turn crashes in South Australia. Specifically, an in-depth study of crashes in the Adelaide area was included, looking at only a small sample size from the entire right-turn crash dataset. They concluded that filtered right turns were responsible for most of the crashes investigated, stating that:

> *... over 90% of these signalised intersections had red and green arrows to control right turns but, as far as could be determined, almost all of the collisions occurred when the arrows were no longer illuminated and through traffic still had a green signal. At least one driver stated that she became confused when the red right turn arrow was turned off but the green signal for through traffic remained on. She assumed that it meant that it was safe to turn, only to be confronted with oncoming traffic that still had a green signal. This effect may be a factor in why right-turn crash rates at partially controlled intersections appear to be little different from uncontrolled intersections.*

In the US, Davis and Aul (2007) determined crash modification factors associated with different left-turn phasing schemes, specifically at intersections where the approach speed limit exceeded 40m/hr (64km/hr) in Minneapolis, Minnesota. Crash modification factors merely estimate the expected reduction (or increase) in crashes after a particular countermeasure is implemented. The summary of the crash modification factors are detailed in table 2.2.

Table 2.2 Summary of crash modification factors by left-turn (NZ right) treatment (Davis and Aul 2007)

Countermeasure	Total crashes	Rear-end crashes	Side-impact crashes	Total left-turn crashes	Major approach left-turn crashes	Minor approach left-turn crashes
New signal with protected major approach	No effect detected	Increase	Decrease	No effect detected	No effect detected	No effect detected
New signal with filtered/protected major approach	No effect detected	No effect detected	No effect detected	Increase	Increase	No effect detected
Minor approach change from filtered to filtered/protected	No effect detected	No effect detected	No effect detected	Inconclusive	Inconclusive	Inconclusive
Minor approach change from filtered to fully protected	No effect detected	No effect detected	No effect detected	Inconclusive	Inconclusive	Inconclusive
Minor approach change from filtered/protected to fully protected	No effect detected	No effect detected	No effect detected	Inconclusive	Inconclusive	Inconclusive
Major approach change from filtered/protected to fully protected	No effect detected	No effect detected	No effect detected	Decrease	Decrease	Decrease
Major approach change from protected to filtered/protected	No effect detected	No effect detected	Decrease	Inconclusive	Inconclusive	Inconclusive

Qi et al (2009) evaluated the safety impacts of the three different types of left-turn signal phasing also under the US driving regime, focusing on 76 intersections in Austin and Houston, Texas. They used

Poisson and negative binomial regression models to correlate left-turn treatments with crash rates. The results of the models are shown in table 2.3.

The report used a reference treatment type of filtered/protected treatment, and the model showed that a change to fully protected treatment reduced the crash rate. Further, the reference treatment of both left turns lagging (that is, phased at the end of the through movements) had a higher crash rate than either both turns leading (before the through movements) or a lead-lag phasing (one for each left turn). And as discussed previously, an increased number of approach lanes leads to a higher crash rate.

Table 2.3 Left-turn (NZ right) crash relationship variables for left-turn treatments (Qi et al 2009)

Variable	Estimated coefficient (negative binomial)
Protected-only	-0.6969
Filtered/protected (reference)	0
Lead-lag phasing	-1.099
Lead-lead phasing	-1.1559
Lag-lag phasing (reference)	0
Number of lanes	0.1263

Additionally, Qi et al (2009) looked at consistency in left-turn treatments; that is, how consistently left-turn signals are used along a corridor. The research showed that an increasing number of different signal-phasing changes along a corridor leads to a higher crash risk, with a major increase above about one-third of the intersections changing their signal-phasing plans along the corridor.

2.5 Impacts of intersection improvements

2.5.1 Geometric changes

There appeared to be a paucity of data on the impacts of geometric changes at signalised intersections, except the addition of lanes for right- and left-turning traffic, and a general trend towards additional lanes (and width) leading to increased crash rates. The impact of short additional through lanes, for example, which is a commonplace feature at New Zealand intersections, does not appear to have been evaluated for safety impacts.

2.5.2 Phasing/operational changes

As discussed earlier, increased numbers of signal phases and more complicated phasing tends to increase crash rates, according to overseas research. Very little research exists on the safety implications of particular phasing schemes, such as split phasing (where opposing directions of flow are each given their own full phases instead of running concurrently) or the reintroduction of turning phases after they have already been called once in a cycle.

Abdel-Aty and Wang (2006) looked into the impact of coordinated signals on crash rates, using negative binomial regression. The study looked at 476 intersections along 41 coordinated corridors in Miami, Florida. The study found that larger numbers of approach lanes, short distances between signalised intersections along the corridor, high speed limits (not just at the intersection but along the entire corridor), and complicated signal plans with a high number of phases led to increased crash rates. Three-legged intersections, exclusive right-turn (left-turn in New Zealand) lanes, and protected left-turn (right-turn in New Zealand) phasing led to lowered crash rates.

Tindale and Hsu (2005) looked at coordinated one-way corridors in Tampa-St Petersburg, Florida. This arrangement is also commonplace in the CBDs of major New Zealand cities, from Auckland to Christchurch to Dunedin, and is anecdotally an arrangement that leads to greater-than-expected red-light encroachment. The couplet of one-way streets that Tindale and Hsu looked at had a high 25% distribution of red-light-running crashes that occurred within the local district. There was no discussion on effective countermeasures for this occurrence, although red-light running has been covered already.

Lyon et al (2005) developed safety performance functions (SPFs) based on the crash database from the city of Toronto, Ontario, primarily to test out left-turn (right turn in New Zealand) countermeasures, and secondarily to look at the impact of the number of approach lanes, left- and right-turn lanes, and pedestrian activity. Ultimately, though, the SPFs were used to evaluate two types of left-turn priority phasing – flashing advanced green and a left-turn green arrow – across 35 intersections with three years of crash data. The two left-turn treatments resulted in a 17% reduction in left-turn collisions and a 19% reduction in left-turn side-impact collisions. That latter figure was an average of the two treatments – flashing advanced green resulted in a 12% reduction, while left-turn green arrows resulted in a 25% reduction. No other crash types were evaluated with these SPFs for the report.

2.5.3 Pedestrian and cyclist accommodation

Mak et al (2006) looked at a new countermeasure that was being trialled in Australia to provide additional notification to turning drivers of conflicting crossing pedestrians at signalised intersections. This countermeasure is a yellow flashing turn arrow that accompanies the flashing red man on the corresponding pedestrian phase. Mak et al undertook a before-and-after crash analysis of 36 sites in New South Wales, Australia, where the countermeasure had been installed, using 1.5 to 3 years of crash data before and after. The analysis showed a decrease of 9.23% in the number of crashes per treated turning movement analysis year, although the authors noted that this reduction was not statistically significant given the small sample size.

Another pedestrian countermeasure, the leading pedestrian interval, is a relatively new development and hasn't been studied extensively. King (2000) looked at its implementation in New York City, where right-turn-on-red is permitted (as opposed to in New Zealand, where the equivalent left-turn-on-red is not permitted). The leading pedestrian interval gives a red arrow to turning traffic for the length of the green man phase, allowing pedestrians to get part way into the crosswalk before conflicting vehicular traffic is released, thus improving their visibility. King did a before-and-after analysis of 26 locations that had had this interval implemented, with five years of before and five years of after data. The study noted a 22% decrease in pedestrian–vehicle crashes in crosswalks, and a 12% decrease in turning vehicle crashes over the five-year period. Other US studies have shown an increase in crashes with the implementation of the leading pedestrian interval, but it has been speculated that this increase is due more to the permitted right-turn-on-red rather than the countermeasure itself.

Another area of interest, but in which there has been little research carried out, is the treatment of left turns at signals and the follow-on effects on pedestrians and cyclists. Left turns are typically accommodated with a slip lane (which can be signalised but typically is not) or just using the same alignment as the adjacent lanes. In order to more safely accommodate pedestrians at signalised intersections in the US, the Pedestrian Safety Guide and Countermeasure Selection System (PEDSAFE) (2002) recommends a new slip-lane design similar to that in common use in Australasia, where the turning vehicle approaches at a 55–60 degree angle to give way when turning. PEDSAFE notes that visually impaired pedestrians still have trouble navigating slip lanes regardless, although no research is presented documenting the safety effects of left-turn treatments.

2.5.4 Physical improvements

Wundersitz (2009) studied the impacts on crash rates of the addition of gantry-mounted signal displays to intersections in Adelaide. Most of Wundersitz's literature review was based on American research, as there does not appear to be significant research from other parts of the world into these physical improvements. The reader is cautioned, however, that the approach to signal display in the US is markedly different than that in Australasia (and so is the accompanying driver expectation), so comparisons are somewhat difficult. American research quoted by Wundersitz suggested that a combination of post-mounted secondary signal displays and gantry-mounted signal displays led to a reduction in crashes of all types. The sole Australian research quoted by Wundersitz actually showed a 21% increase in overall crashes at black-spot sites in Victoria after the installation of gantry-mounted signals. The other results were all based on American studies and will not be repeated here due to the aforementioned difficulty in Australasian applicability. Further research in New Zealand is needed in this area.

It is interesting to note, though, that the use of gantry-mounted signal displays is declining in the UK (Trim and Barak-Zai 2008) due to regulations imposed by the European Union that extend the liability of safety during signal operation and maintenance onto the signal designers. Because there is concern about the risk to maintenance personnel from lamp replacement, the use of gantry-mounted signals is being supplanted by tall posts with repeated signal heads.

Sayed et al (2007) looked at the implementation of visibility improvements at signals in British Columbia, Canada (so the reader should bear in mind the earlier discussion on US signal design). Improvements included larger lens sizes (up to 300mm), new target boards, reflective target boards (similar to those already in use in Australasia), and additional signal heads across 139 improved intersections, compared with 85 control group intersections. Sayed et al used generalised linear modelling to evaluate the installations where all improvements were implemented. The research found a crash rate reduction of more than 7% for all crashes, 8.5% for property damage only, 6% for daytime crashes, and 6.6% reduction for night-time crashes.

In Victoria, Australia, Monash University published a report dictating guidelines for replacing signalised intersections with roundabouts (Corben 1989). While this physical change is so substantial as to be beyond the scope of this study, it is interesting to note that the study quotes multiple previous studies that demonstrated that:

> *... roundabouts exhibit superior safety performance to intersection signals ... where a roundabout is more appropriate than intersection signals, taking into account [vehicular] traffic mix, traffic flows, and turning movements, as well as intersection geometry.*

This report only considered vehicular traffic and did not look at the safety impacts to cyclists or pedestrians of replacing signals with roundabouts.

3 Sample selection

3.1 Introduction

The original scope of this research study consisted of the inclusion of low- and high-speed signalised intersection sites from five New Zealand cities: namely Auckland, Wellington, Christchurch, Hamilton and Dunedin. In addition, VicRoads indicated that they would be interested in participating in this study. Low- and high-speed signalised intersections from Melbourne were thus also identified and included in the sample set for this study.

The inclusion of the Melbourne sites was beneficial as it provided a more diverse set of sites for the study to draw data from, and was thus likely to lead to the development of a more robust model. Additional funding for data collection and analysis at the Melbourne sites was provided by VicRoads. Beca has previously collected data for 31 high-speed intersections in Melbourne, and some of these have been included in the list of selected intersections.

The utilisation of data from New Zealand sites predated the changes to the give-way rule on 25 March 2012. Also, Melbourne/Victoria has different road rules from New Zealand, including different give-way rules.

3.2 Site selection considerations

Signalised intersection sites were selected primarily from a desktop assessment of road maps and aerials. The following are certain key considerations that were used while selecting sites for this study:

- Only three-arm and four-arm traffic signals were included in the sample set.
- Intersections lying on one-way streets were excluded.
- Intersections not on the SCATS® system were also excluded.
- Low-speed (50, 60 and 70km/h on all approaches) sites were identified for each of the New Zealand centres and for Melbourne. In addition, high-speed (80 and 100km/h on at least two approaches) sites were identified for Auckland, Wellington and Melbourne.
- The surrounding land-use type was considered while selecting sites. Intersections that were selected were located in a diverse set of land-use environments, including central business districts, commercial and industrial areas, residential suburbs, and high-speed rural environments.
- Efforts were made to limit the number of signalised intersections with free left turns. However, a significant number of sites in the sample set still had free left turns on one or more approaches.
- Where possible, intersections that were known to lie along a coordinated traffic signal corridor were selected, so that the effects of this coordination could be analysed while developing the models.
- Traffic signal sites at locations associated with higher-than-usual gradients (eg Dunedin) were included in the sample set.

As part of previous research undertaken by Beca, data had already been collected at around 30 high-speed sites in Auckland, Wellington and Melbourne. These sites were included in the sample set for this study. In addition, some additional high-speed sites were identified for which additional data collection was undertaken.

3.3 Further exclusions from the sample set

It was recognised that some of the intersections selected during the desktop assessment may have undergone significant changes over the five-year study period (2004–2008). These needed to be excluded, as the implementation of significant changes was likely to affect the crash risk of vehicles at that site.

The following changes were deemed to be significant from the perspective of safety implications at the site:

- changes to intersection geometry (eg addition of traffic lanes, changes in intersection layout and geometry)
- changes to signal phasing (eg addition of protected turning phases)
- the addition of signal displays or mast arms.

It was noted that most of the signal aspects in Auckland had been upgraded to LED during the last five years. Although the upgrade of signal lamps to LEDs was expected to lead to an improvement in the level of safety, it was decided not to exclude the Auckland sites where this upgrade had been implemented, since this was a city-wide effect in the Auckland region, and thus would not result in irregular changes in signal performance across the region.

Traffic signal engineers from the respective councils (and from the Traffic Management Unit (TMU) in the case of Auckland) were contacted and requested to identify any changes that had been made at the initial list of selected sites. In some cases, such as Auckland, this history is stored in a central database which made the task of identifying changes quite simple. However, for most cities, the signal engineers had to rely on physical or electronic document archives, and on memory, for isolating intersections that had undergone significant changes.

The following local authorities were liaised with for the purpose of site selection and data collection:

- **Auckland**: Auckland City Council, North Shore City Council, Manukau City Council, Waitakere City Council, NZTA TMU
- **Wellington**: NZTA
- **Christchurch**: Christchurch City Council
- **Hamilton**: Hamilton City Council
- **Dunedin**: Dunedin City Council
- **Melbourne**: VicRoads.

Table 3.1 shows the number of intersections that were excluded from each location and the final number of intersections selected for data collection from each city.

Table 3.1 Selected intersections by location

Location	Initial number of selected intersections	Exclusions	Final number of selected intersections	Number of approaches at selected intersections
Auckland	127	38	89	324
Christchurch	66	13	53	205
Dunedin	14	3	11	43
Hamilton	27	10	17	66
Melbourne	69	11	58	214
Wellington	44[a]	34	16	37
Total			**244**	**889**

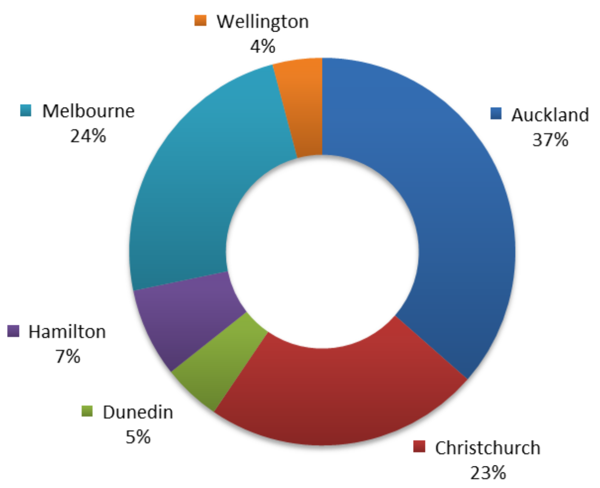
a) In Wellington, a set of 44 state highway traffic signals, maintained by the NZTA, was selected for the study. Most of these were found to have undergone significant changes during the five-year period from 2003 to 2008, and a final set of 16 sites was selected.

3.4 Selected intersections, by type and location

3.4.1 Intersections, by location

Figure 3.1 depicts the proportion of sites selected for inclusion in the sample set, by location.

Figure 3.1 Intersections, by location



Auckland accounted for a significant proportion (37%) of sites in the sample set. This was an expected trend, given the large number of signalised intersections that were located within the greater Auckland area. Melbourne (24%) and Christchurch (23%) also each accounted for a large proportion of the selected intersections.

Sites in Auckland were located in areas that were under the jurisdiction of different local councils. The following figure provides a breakdown of sites according to the respective council area.

The complete list of signalised intersection sites selected for inclusion in this research is attached in appendix A of this report.

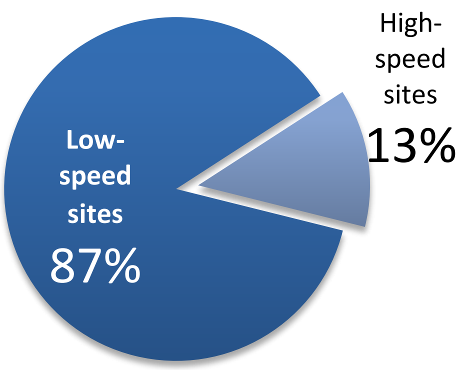
3.4.2 Sites, by type

Table 3.2 provides a summary of the number of low-speed and high-speed approaches at each location. For the purpose of this research, signalised intersection sites were classified as high-speed If they had a speed limit of 80kph or above on at least two approaches. Figure 3.2 illustrates the proportion of sites classified by speed environment.

Table 3.2 Low-speed and high-speed approaches

Location	Low-speed sites	High-speed sites	Total selected intersections
Auckland	81	8	89
Christchurch	53	0	53
Dunedin	11	0	11
Hamilton	17	0	17
Melbourne	37	21	58
Wellington	8	2	10
Total	**207**	**31**	**238**

Figure 3.2 Proportion of low-speed and high-speed sites



The sample set consisted of both three-arm and four-arm intersections. Five-arm and six-arm signalised intersections were not included in the sample set for this study. Table 3.3 provides a breakdown of sites by the number of arms.

Table 3.3 Low-speed and high-speed sites

Location	Three-arm intersections	Four-arm intersections	Total
Auckland	28	61	89
Christchurch	7	46	53
Dunedin	0	11	11
Hamilton	1	16	17
Melbourne	16	42	58
Wellington	5	5	10
Total	**57**	**181**	**238**

4 Data collection

4.1 Introduction

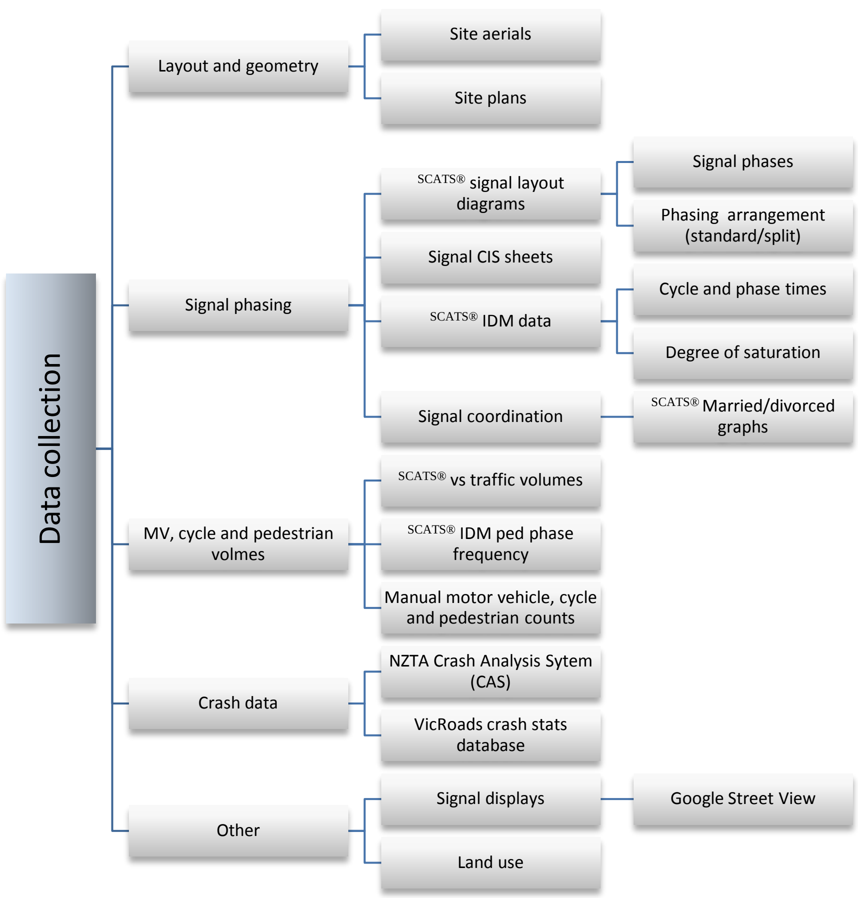
Data collection on a wide range of physical and operational characteristics of signalised intersections was undertaken as part of this study. Data was collected for each individual approach at the selected signalised intersection sites. This enabled a finer analysis of the data and of individual crash-causing movements, by approach, and enabled more accurate crash prediction models to be built.

The data requirements of this study can be grouped under the following five categories:

- signal layout and geometry
- signal operation (signal phasing and coordination)
- motor vehicle, pedestrian and cycle volumes
- crash data
- miscellaneous (signal displays and land use).

Figure 4.1 presents a summary of the data sources that were utilised to collect data for each of the above categories. Further details on each of the above data categories are provided in the subsections below.

Figure 4.1 Data categories and sources



Data on a number of different variables was collected within each of the categories listed in figure 4.1. These variables were subsequently tested during the development of the crash prediction models. Figure 4.2 lists all variables, by type, for which data was collected.

Figure 4.2 Variables tested during modelling

Volumes
- Motor vehicle (SCATS®)
- Cycle volumes (manual counts)
- Pedestrian volumes (SCATS®)

Layout
- Distance from upstream intersection
- Presence of free left-turn lane
- Presence of shared turns (left or right)
- Presence of shared left-turn lane
- Presence of shared right-turn lane
- High-speed approaches

Geometry
- Total number of approach lanes
- Number of lanes for through traffic
- Total width of approach lanes
- Presence of raised median/central island
- Intersection depth
- Number of exit lanes
- Distance of exit lane merging
- Distance from upstream intersection
- Length of right-turn bay/lane
- Offset of right-turning bays
- Dominant land use

Phasing
- Green time for through movement
- Yellow time for approach
- All-red time for approach
- Cycle time for intersection
- Degree of saturation
- Number of times pedestrian phase was called (SCATS)
- Coordination with upstream junction
- Type of phasing: standard/split/combined
- Right-turn phasing type: fully protected/partially protected/ filtered
- Presence of advance detectors on approach

Signal displays
- Low height of signal posts
- Presence of overhead mast arm
- Number of signal displays
- Number of aspects on signal displays
- Left side
- Right side
- Mast arm

Bus bays/parking
- Distance from limit line back to parking
- Presence of bus bay within 100m of approach
- Presence of bus bay within 100m of exit

Pedestrians
- Presence of pedestrian crossing
- Left-turn pedestrian protection
- Type of pedestrian crossing on Free Left Turn (if present)
- Barnes dance arrangement

Cyclists
- Presence of cycle facilities
- Presence of approach facilities
- Presence of storage facilities

4.2 Signal layout and geometry

Signal layout and geometry data was collected through site maps, aerial imagery and spatial tools such as Google Earth. Site imagery available through Google Street View was also utilised to collect data on signal posts, displays and aspects. Although most site imagery on Google Street View dates back to 2007/2008, this was not considered to be an issue with data collection, as the selected sites had already been checked to ensure that no significant changes had been implemented within the last five years.

Data on signal layout and geometry was further subdivided into the categories outlined in figure 4.3.

Figure 4.3 Intersection layout and geometry data

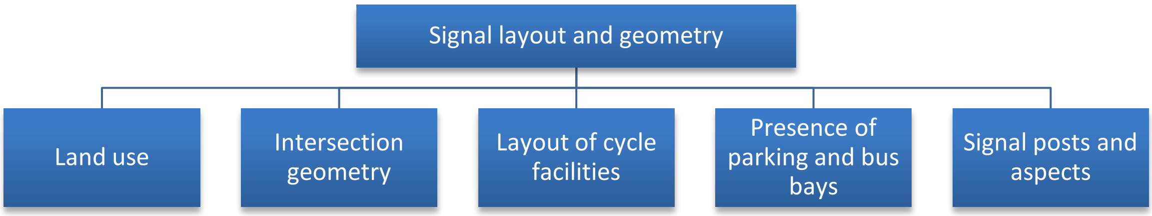


Table 4.1 lists the variables for which data was collected for each category.

Table 4.1 Layout and geometry data variables

Variable	Comments
Land use	
Land-use type	Aerial imagery was used to determine the dominant surrounding land-use type at the site, whether residential/commercial or industrial.
Intersection geometry	
Distance from upstream intersection	Distance to upstream intersection for each approach.
Upstream intersection type	Whether signalised, priority, roundabout, signalised pedestrian crossing, etc.
Number of approach lanes	Total number of lanes for approaching traffic at the individual arm of intersection. This does not include any free-left-turn lanes.
Number of through lanes	Number of lanes for through traffic.
Approach lanes width	Total width of approach lanes. Does not include width of free-left-turn lanes.
Raised median/central island	Whether a raised median or central island was present on the approach.
Intersection depth	The crossing distance from the approach to the opposite approach (see section 4.2.1).
Lane layout type	The arrangement of approaching lanes (see section 4.2.2).
Number of exit lanes	Number of exit lanes on the same approach.
Distance of exit lane merging	Distance from limit line to the end of the merge line for exit lanes.
Pedestrian crossing	Presence of a pedestrian crossing over or parallel with the approach.
Length of right-turn bay/lane	Length of right-turn bay/right-turn lane at the approach, measured from the limit line back to the farthest point of full width of the turning bay or lane.
Offset of right-turn bays	The offset of opposing right-turn bays/right-turn lanes.
Free-left-turn pedestrian crossing	Type of pedestrian crossing amenity provided on a free left turn (if present) – eg no ped facilities, zebra crossing, raised zebra crossing, signalised pedestrian crossing.
Advance detectors	Whether advance detector loops were present at the approach.
Layout of cycle facilities (if present)	
Approach	Location/layout of approaching cycle lanes (see section 4.2.3).
Storage	Presence of cycle storage facilities (eg advanced cycle boxes) (see section 4.2.3).
Presence of parking and bus bays (if present)	
Parking within 100m	Distance from the limit line to any parking provisions within 100m of the approach.
Bus bay within 100m	Presence of a bus bay within 100m from the limit line on the approach lanes and/or exit lanes of each approach.

Variable	Comments
Signal posts and displays (refer to signal hardware terminology definitions)	
Height of signal posts	The height of signal posts, which affects the height at which signal displays are fixed. Classified as low (eg in certain CBD areas) or normal.
Overhead mast arm	Whether an overhead mast arm with signal displays was present on the approach side of the intersection.
Number of signal displays	The number of signal displays visible to a driver approaching the intersection. Includes signal displays on both the approach and exit sides of the intersection.
Number of aspects on signal displays	The number of lamps on each signal display on the approach side of the intersection. Recorded for signal displays on the left side of approach, and on the median and mast arm (if present) (see section 4.2.4).

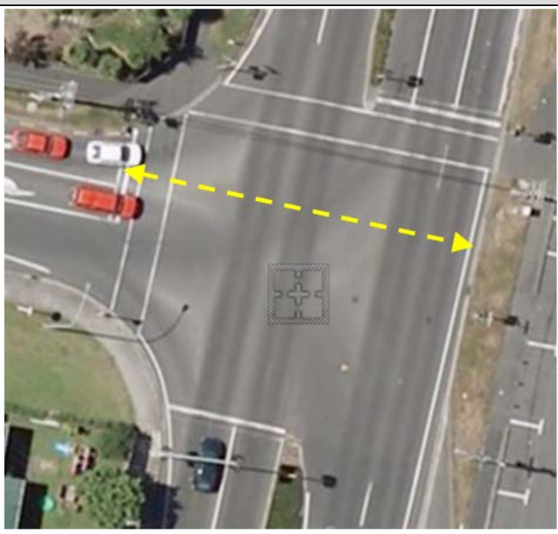
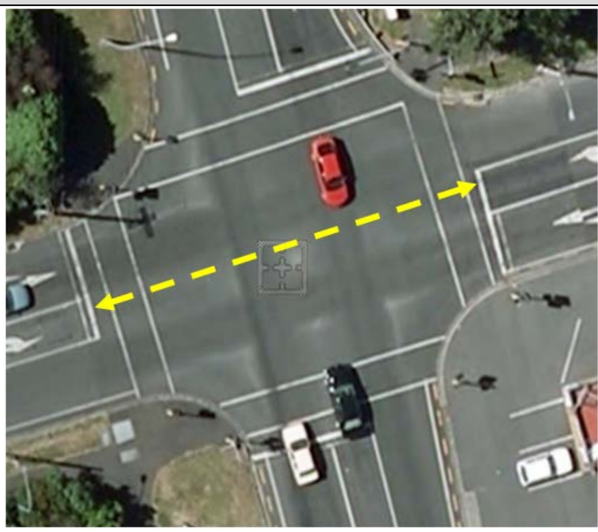
Sections 4.2.1–4.2.4 provide further details on the variables listed in the table above.

4.2.1 Intersection depth

The intersection depth defines the distance to be covered by an approaching vehicle for reaching the opposite approach. It is measured as the distance between the limit lines on the approach arm and the exit arm that is geometrically opposite to the approach. In the case of T-junctions, the intersection depth for the minor arm is measured as the distance from the limit line of the approach to the geometrically opposite edge of the intersection.

Intersection depths for three-arm and four-arm junctions are shown in figure 4.4.

Figure 4.4 Intersection depth measurements

Three-arm intersection (minor arm)	Four-arm intersection
	

4.2.2 Lane layout type

Various lane arrangements were observed at the selected sites. These lane arrangements are primarily a result of differences in the number of left-turn/right-turn lanes, presence of a free-left-turn lane or presence of shared turning lanes for traffic. It was considered appropriate to group the different lane layouts into certain common lane arrangement types. The lane layout types were subsequently used to determine whether free left turns or lanes with shared movements were present on the approach.

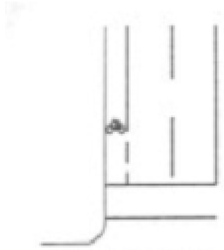
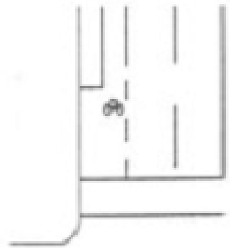
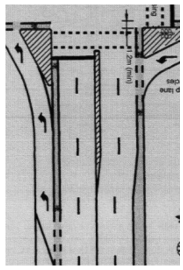
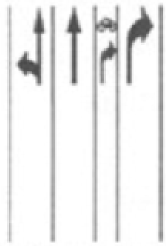
The coding system utilised for classifying lane layouts at intersection approaches is provided in appendix B of this report.

4.2.3 Layout of cycle facilities – approach and storage

The presence of facilities for cyclists at the selected intersections was recorded. These were divided into the following two categories:

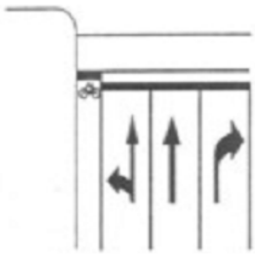
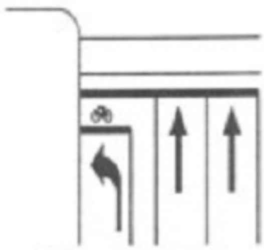
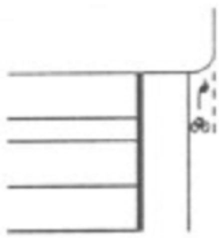
- Facilities at the approach, such as cycle lanes at the intersection – different layouts of the cycle lane with respect to the traffic lanes were recorded, as shown below:

Table 4.2 Cycle facilities – approach

Kerb side	Car side	Car side with slip lane	Right side
			

- Facilities for storage, such as cyclist storage facilities, eg advanced stop boxes. These were categorised as follows:

Table 4.3 Cycle facilities – storage

Advanced stop box	Expanded	Hook turn
		

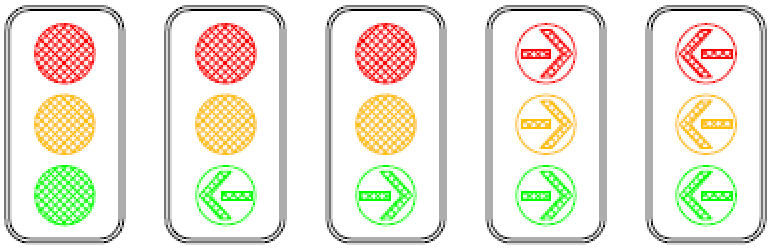
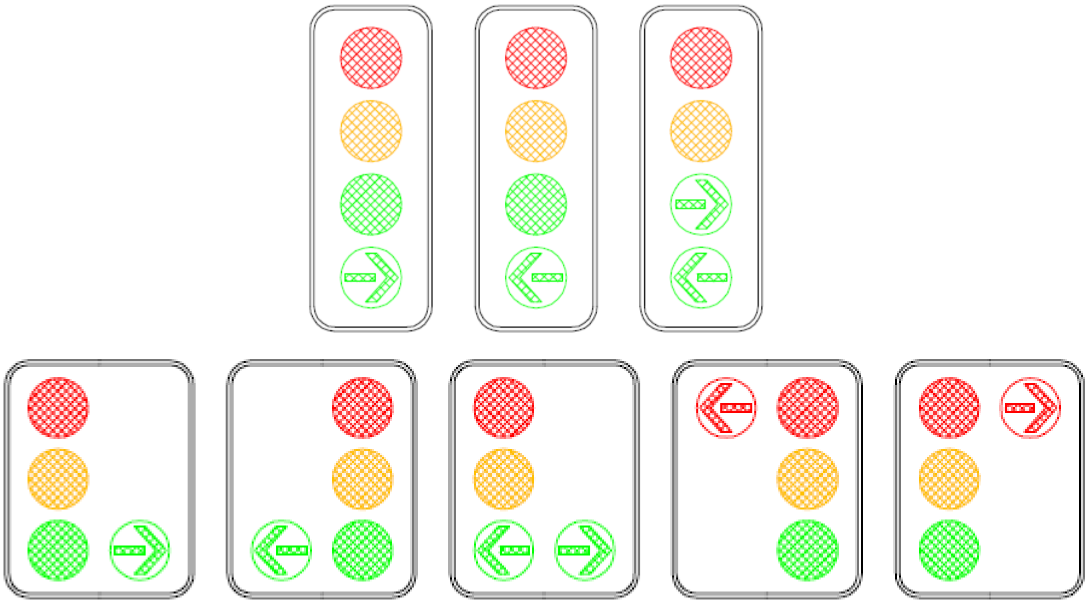
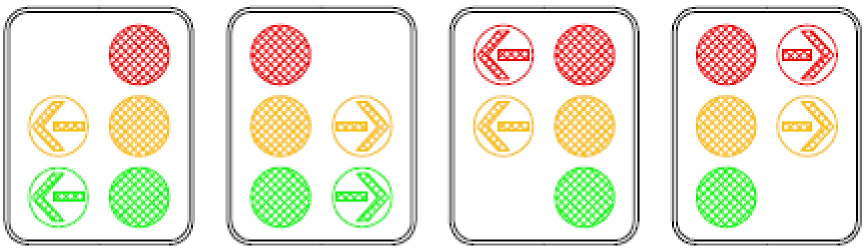
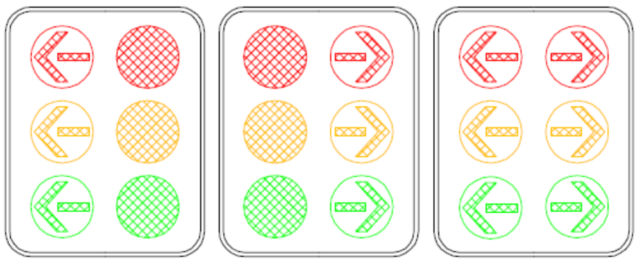
4.2.4 Number of aspects on signal displays

The number of aspects on each traffic signal post on the approach side of the intersection was recorded. This was done for signal displays on the left side of approach, and on the median and mast arm (if present), as shown in figure 4.5.

Figure 4.5 Location of signal displays on left side of approach, median and mast arm (Google Street View image)

Table 4.4 shows typical layouts of three-, four-, five- and six-aspect traffic signal displays that are employed in New Zealand.

Table 4.4 Layouts of three-, four-, five- and six-aspect traffic signal displays

Three-aspect	
Four-aspect	
Five-aspect	
Six-aspect	

4.3 SCATS® data collection

SCATS® detector loop traffic counts and Intersection Diagnostic Monitor (IDM) data (described in section 4.4) were collected for a two-week period for each site. SCATS® data collection for Christchurch, Hamilton, Dunedin, Wellington and Melbourne was done over the last two weeks of February 2010.

Data collection for Auckland was split into two phases, due to the large number of sites. This was required since SCATS® imposes a limit on the number of sites for which data can be recorded simultaneously. The Auckland sites were thus grouped into two sets, with data collection for the first set done along with that of the Christchurch, Hamilton, Dunedin and Melbourne sites during the last two weeks of February 2010, while data for the remaining set was done over the first two weeks of March 2010.

4.4 Traffic, pedestrian and cycle volumes

4.4.1 Traffic counts

Traffic counts from the SCATS® system were collected for each of the sites included in the study. In addition, short-duration (15min) manual turning counts were also undertaken at approaches where lanes with shared movement were present, to identify turning volume proportions within the SCATS® detector loop data for that lane. Manual traffic counts were also undertaken for free left turns in cases where SCATS® detector loops were not present.

Manual traffic counts were undertaken at each site during the same period as that of the SCATS® data collection. Since the purpose of the manual counts was solely to identify the proportion of traffic turning on shared lanes (and of free-left-turning volumes, where required), the use of short-duration counts was considered satisfactory.

A more detailed description of the SCATS® detector loop count data, along with an analysis of the error inherent in them, is provided in section 6.2.

4.4.2 Pedestrian and cycle counts

Manual cycle-turning volumes were extracted for all Christchurch sites from the traffic counts database maintained by Christchurch City Council. In addition, cycle counts for 18 intersections were obtained from the Cycle Monitoring study prepared by Gravitas Research for the Auckland City Transport Strategy in June 2009.

Short-duration (15min) counts of turning cyclists and crossing pedestrians were also conducted at all of the 11 sites located in Dunedin, and at 10 intersections in Melbourne that were identified to have medium-to-high cyclist and pedestrian volumes.

4.5 Signal-phasing information

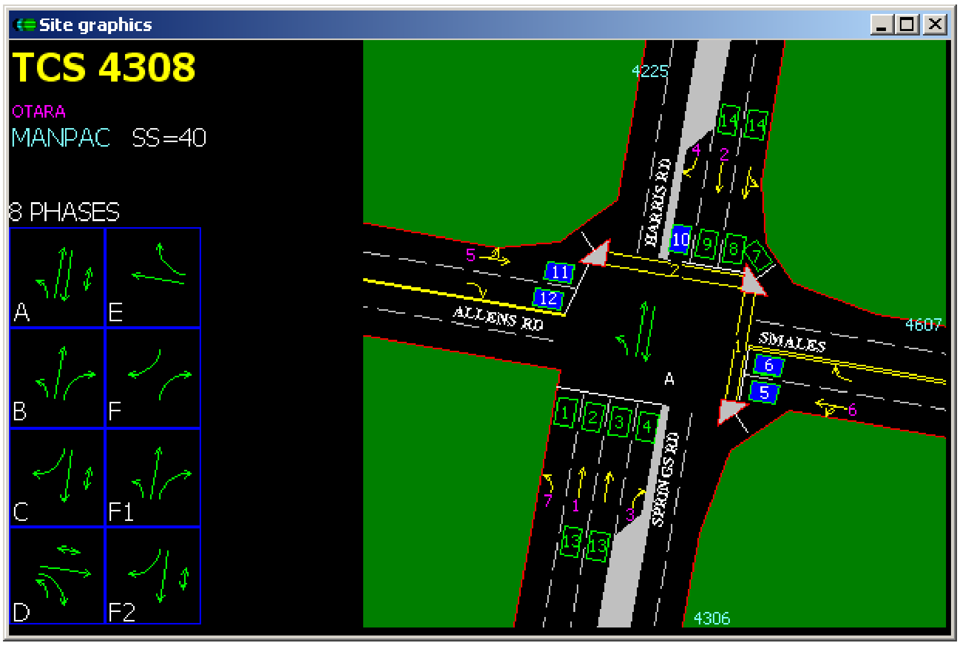
In addition to geometric characteristics of signalised intersections, this study aims to assess the effects of signal phasing on safety. The SCATS® system was used as the primary source of data on signal phasing.

The following subsections provide details on the various phasing data that were collected.

4.5.1 Signal phases

SCATS® intersection layout diagrams showing the possible phases that can be run at a particular intersection, along with location and numbers of detector loops, were obtained for all sites. Figure 4.6 shows an example SCATS® phasing diagram.

Figure 4.6 SCATS® signal layout diagram



4.5.2 Signal cycle times and phase times

The SCATS® system is capable of recording detailed intersection phasing data in the form of IDM files. Each IDM file contains a detailed cycle-by-cycle description of the operation of a signalised intersection during a 24-hour period. These files can subsequently be analysed and processed to obtain information on average/minimum/maximum signal cycle times, average phase times and the number of times that pedestrian phases were activated.

IDM data for a 14-day period was collected for all selected sites. Due to the large amount of data that was collected, traffic volume data was used to determine the most representative day, and the IDM data for this day was then processed to give the required inputs for the crash prediction models. Data was extracted for both the 24-hour period, as well as for the AM and PM peak periods for the selected day. Based on traffic volumes and green times, the average degree of saturation was also calculated for each intersection approach.

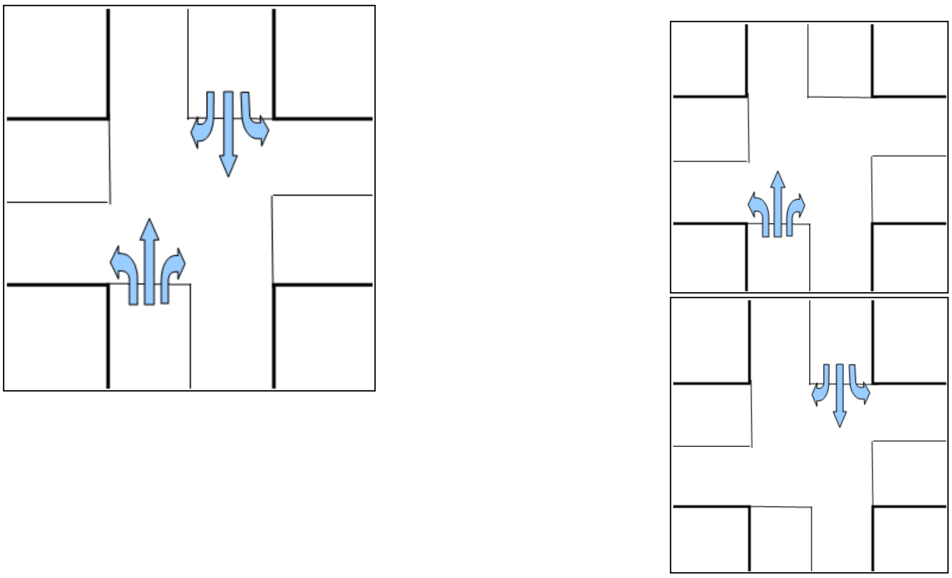
A more in-depth description of the analysis of SCATS® IDM data is provided in section 6.1.

4.5.3 Signal-phasing sequences

The IDM data for each site, along with the respective SCATS® signal layout diagram and Controller Information Sheets (CISs), were used to record information on the following variables:

- Phase type – Signals may have either a standard phasing sequence, whereby opposite sets of approaches are released simultaneously, or a split phasing sequence, whereby an individual approach has its own green phase. This is depicted in figure 4.7. Some signals may have both standard as well as split phases operating at them, depending on time of day. These were also noted.

Figure 4.7 Standard vs split phasing sequences



Standard phasing

Split phasing

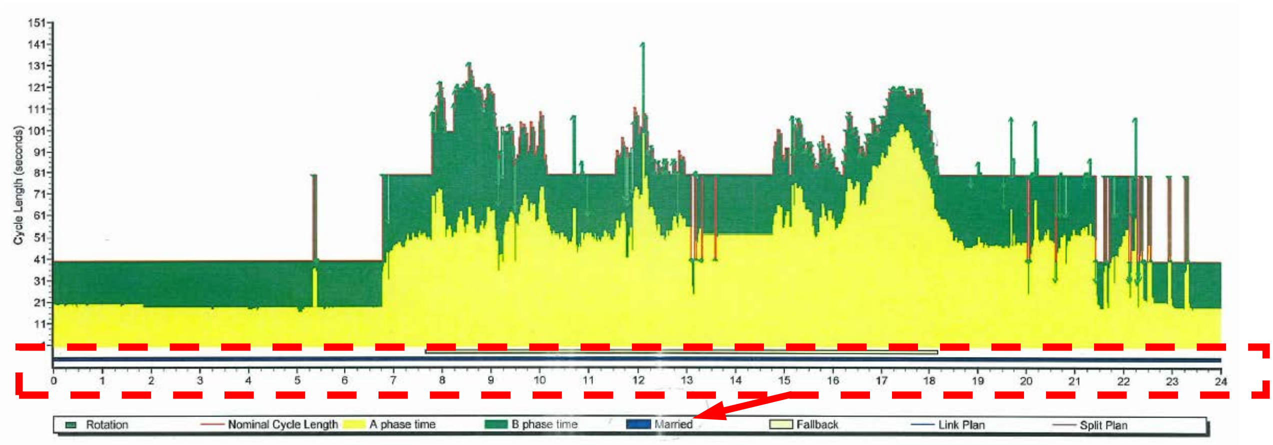
- Right-turn phase – The type of right-turn phasing – ie whether the right turn in question operates as a protected, partially protected (ie part-time filter) or filtered right turn – was also assessed. Information on whether partially protected right turns operated as leads or lags was found to be difficult to obtain, given the large number of sites.

4.5.4 Signal coordination

Signal coordination along a corridor may often have an effect on red-light running rates at individual intersections. To analyse the effect of signal coordination on crash rates, data on whether a signalised intersection was 'married' or 'divorced' (ie whether or not it was coordinated with other sets of signals on the SCATS® system) during the peak and off-peak periods was collected.

SCATS® Traffic Reporter outputs, an example of which is shown in figure 4.8, were used to determine signal coordination. The continuity of the 'married' status line at the bottom of the output graph provides an indication of the coordination status of the signal at various times during the day.

Figure 4.8 Example SCATS® Traffic Reporter output showing 'married'/'divorced' status



The ‘married’/’divorced’ status for each intersection was recorded for the peak and off-peak periods. It was also noted that although a signalised intersection may be ‘married’ during a certain time period, this may not translate into reasonable traffic coordination for all approaches of that intersection. Information on upstream intersections was thus also looked at to determine which approaches of the intersection were coordinated. An intersection approach was determined to be coordinated during a given period if it was ‘married’ for more than 50% of the period, and there was a signalised intersection within 600m upstream of the limit line on the approach.

4.6 Master database

A master relational database, which contains all data that has been collected as part of this study, was set up. Data was collected and entered according to the approach of an individual intersection, with each row of the database containing information for a single approach. The format of the database allows convenient addition of more data variables or sites, should this be undertaken in the future.

Figure 4.9 shows a screenshot of the master database.

Figure 4.9 Master database

						Vehicle Phasing							Ped Treatment			Layout	Layout	Layout	Layout	Layout	Layout	Cycle Facilities		
						Phase type			RT Phasing															
No.	Site ID	Ref	Location	Site Name	Arm	Standard 2-way phase	Split phase	Combined phase	RT Present?	Protected RT	Partially protected RT	Filter RT	Ped Phase	LT Phase present?	Barnes dance?	Raised Median / Central Island	Free left turn lane	Shared left	Shared right	Shared lanes	Ped crossing on FLT?	Present?	Approach facilities?	Storage facilities?
						Phasin_std	Phasin_split	Phasin_cmb	RT_present	RT_fprot	RT_pprot	RT_filter	Ped_present	LT_phase	Barnes_dance	Med_island	FLT	Shared_LT	Shared_RT	Shared_turns	FLT_ped	Cycle_fac	Cyc_app	Cyc_storage
2	12071	12071Tama	AKL	Quay / Tamaki . The Str	Tamaki E	1	-1	-1	-1	-1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
2	12071	12071The S	AKL	Quay / Tamaki . The Str	The Strand S	-1	1	-1	1	1	-1	-1	1	1	-1	-1	1	-1	-1	-1	-1	-1	-1	-1
2	12071	12071Quay	AKL	Quay / Tamaki . The Str	Quay W	-1	-1	1	1	1	-1	-1	-1	-1	-1	1	-1	-1	-1	-1	-1	-1	-1	-1
3	12904	12904Stanl	AKL	Stanley / Grafton	Stanley N	-1	-1	1	1	1	-1	-1	1	-1	-1	1	-1	-1	-1	-1	-1	-1	-1	-1
3	12904	12904Graft	AKL	Stanley / Grafton	Grafton S	1	-1	-1	-1	-1	-1	-1	-1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
3	12904	12904Graft	AKL	Stanley / Grafton	Grafton W	-1	1	-1	1	1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
4	12030	12030Orak	AKL	Remuera / Orakei / Asc	Orakei N	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	1	-1	1	-1	1	-1	1
4	12030	12030Remu	AKL	Remuera / Orakei / Asc	Remuera E	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	-1	-1	-1	-1	1	-1	1
4	12030	12030Asco	AKL	Remuera / Orakei / Asc	Ascot S	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	-1	-1	-1	-1	1	-1	1
4	12030	12030Remu	AKL	Remuera / Orakei / Asc	Remuera W	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	1	-1	1	-1	1	-1	1
5	12154	12154Bond	AKL	Bond / New North / San	Bond N	-1	1	-1	1	1	-1	-1	1	1	-1	-1	-1	1	1	1	-1	-1	-1	-1
5	12154	12154New N	AKL	Bond / New North / San	New North E	-1	-1	1	1	1	-1	-1	1	1	-1	-1	1	-1	-1	-1	-1	-1	-1	-1
5	12154	12154Sandr	AKL	Bond / New North / San	Sandringham S	-1	1	-1	1	1	-1	-1	1	1	-1	-1	1	-1	1	1	-1	-1	-1	-1
5	12154	12154New N	AKL	Bond / New North / San	New North W	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
6	12164	12164Capta	AKL	Church / Captain Spring	Captain Springs N	-1	-1	1	1	-1	-1	1	-1	1	-1	-1	-1	1	1	1	-1	-1	-1	-1
6	12164	12164Churc	AKL	Church / Captain Spring	Church E	-1	-1	1	1	-1	1	-1	-1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
6	12164	12164Capta	AKL	Church / Captain Spring	Captain Springs S	-1	-1	1	1	-1	1	-1	1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
6	12164	12164Churc	AKL	Church / Captain Spring	Church W	-1	-1	1	1	-1	1	-1	1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
7	12172	12172O'Ror	AKL	Church / Hugo Johnsto	O'Rorke N	-1	-1	1	1	1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
7	12172	12172South	AKL	Church / Hugo Johnsto	South Eastern Hwy	-1	-1	1	1	1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
7	12172	12172Hugo	AKL	Church / Hugo Johnsto	Hugo Johnston S	-1	-1	1	1	1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
7	12172	12172Churc	AKL	Church / Hugo Johnsto	Church W	-1	-1	1	1	1	-1	-1	1	1	-1	1	1	-1	-1	-1	-1	-1	-1	-1
8	12138	12138Domi	AKL	Dominion/ Walters / Va	Dominion N	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
8	12138	12138Valley	AKL	Dominion/ Walters / Va	Valley E	-1	1	-1	1	1	-1	-1	1	1	-1	-1	-1	1	1	1	-1	-1	-1	-1
8	12138	12138Domi	AKL	Dominion/ Walters / Va	Dominion S	-1	-1	1	1	1	-1	-1	1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
8	12138	12138Walte	AKL	Dominion/ Walters / Va	Walters W	-1	1	-1	1	1	-1	-1	1	1	-1	-1	-1	1	1	1	-1	-1	-1	-1
9	12123	12123Great	AKL	Great North / Alford	Great North N	-1	-1	1	1	1	-1	-1	1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
9	12123	12123Great	AKL	Great North / Alford	Great North S	1	-1	-1	-1	-1	-1	-1	-1	1	-1	-1	-1	1	-1	1	-1	-1	-1	-1
9	12123	12123Alford	AKL	Great North / Alford	Alford W	-1	1	-1	1	1	-1	-1	1	1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1

5 Data analysis

5.1 Distribution of predictor variables

Sections 5.1.1–5.1.7 present the distributions of certain important predictor variables that were used as inputs in modelling.

5.1.1 Traffic volume

Figure 5.1 shows the distribution of daily traffic volume (AADT) entering the signalised intersection from the selected approaches. The sample set consists of a range of low-, medium- and high-volume approaches, with a limited number of approaches having less than 1000 or more than 30,000 vehicles per day.

Figure 5.1 Distribution of daily traffic volume (AADT)

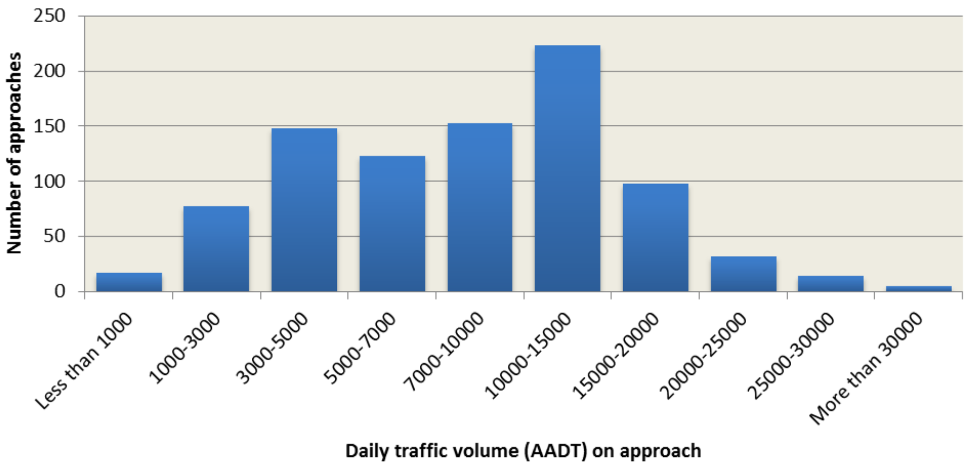
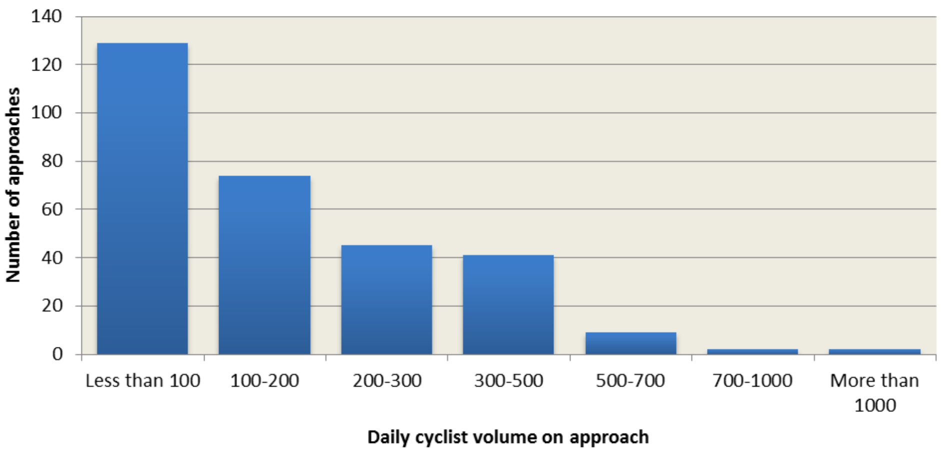


Figure 5.2 shows the distribution of daily cycle volumes for the 82 intersections where manual cycle counts were available. The majority of approaches were observed to have fewer than 200 cyclists per day; however some approaches with significant cyclist numbers were also seen.

Figure 5.2 Distribution of daily cyclist volumes



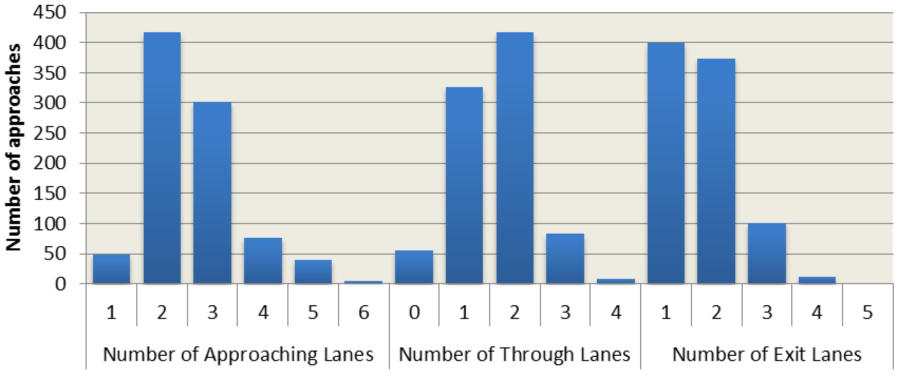
5.1.2 Intersection geometry

5.1.2.1 Number of lanes

Figure 5.3 depicts the distributions of the total number of approach lanes, number of lanes for through traffic and number of exit lanes on approaches in the sample set. Most approaches had either two or three lanes for approaching traffic; however, some of the larger intersections in the sample had four, five or even six approach lanes. A number of approaches also had a single entry lane that was shared by all turning movements.

The majority of approaches (47%) were found to have two lanes for through traffic, while 36% had one through lane. Seven percent of approaches had no through-traffic lane, indicating that these were probably the minor legs of T-intersections. Most intersections also had either one or two exit lanes.

Figure 5.3 Distribution of number of traffic lanes

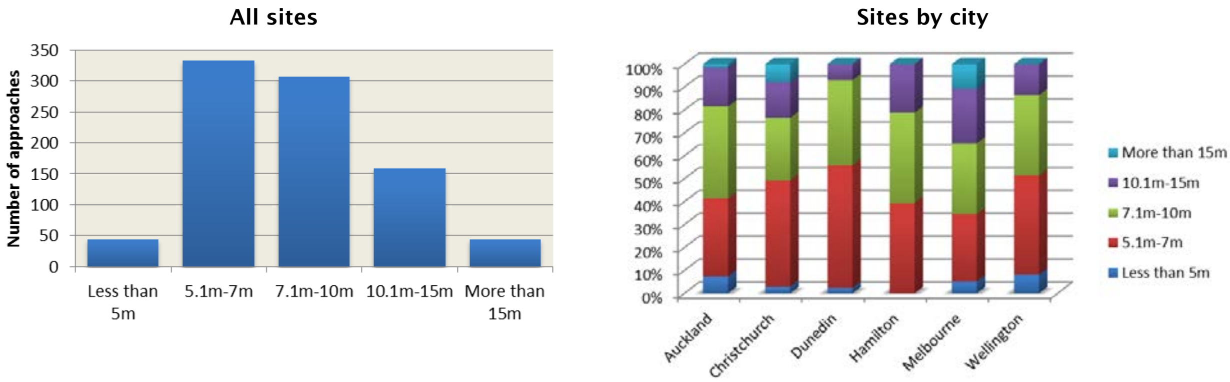


5.1.2.2 Approach width

Figure 5.4 shows the distribution of total approach widths. Seventy-one percent of approaches were between 5m and 10m wide, while 19% were between 10m and 15m wide. A number of narrower (less than 5m) and wider (more than 15m) approaches were also included.

Three approaches were found to have an approach width in excess of 20m. These were all located at the Ferntree Gully/Stud Street intersection in Melbourne.

Figure 5.4 Distribution of approach width

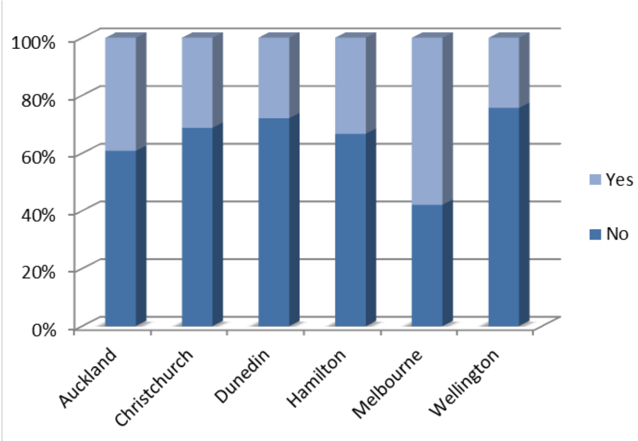


The comparison of approach widths by city suggests that Dunedin and Wellington have a higher proportion of narrower approaches than the other cities. Melbourne was observed to have the widest approaches, followed by Auckland and Hamilton.

5.1.2.3 Presence of raised median or central island

A raised median or central island was present at 358 of the 889 approaches included in the sample. Figure 5.5 indicates that 60% of Melbourne approaches had a median/central island, while this proportion was lower for approaches in the New Zealand cities.

Figure 5.5 Presence of raised medians or central islands

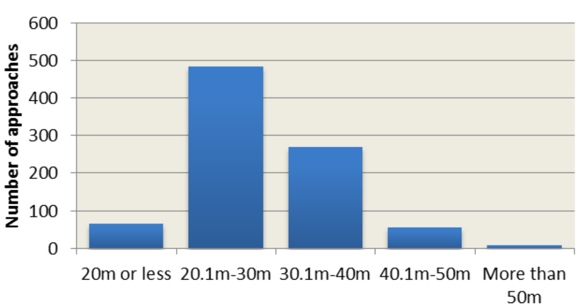


5.1.2.4 Intersection depth

The intersection depths of the selected approaches (intersections) varied between 17m and 58m, with the majority of intersections having depths between 20m and 40m, which together accounted for 74% of all approaches.

Intersections in Wellington (which were all located on state highways) and Dunedin had the lowest intersection depths, while Melbourne and Auckland intersections were observed to have the largest. Christchurch stood out as having a significant proportion (21%) of intersections with depths of 20m or less, although it did have a number of larger intersections as well.

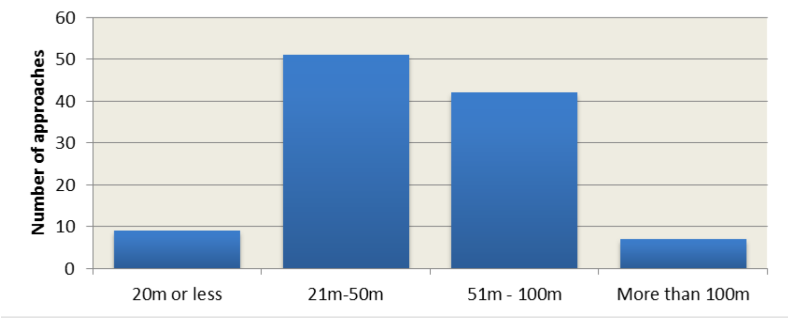
Figure 5.6 Distribution of intersection depths



5.1.2.5 Distance to exit merge

Twelve percent of approaches at the selected intersections had a merge on the exit side of the approach. In a majority of cases, this merge was located at a distance of between 20m and 100m from the start of the exit lanes on the approach. Nine approaches had merges that were located within 20m.

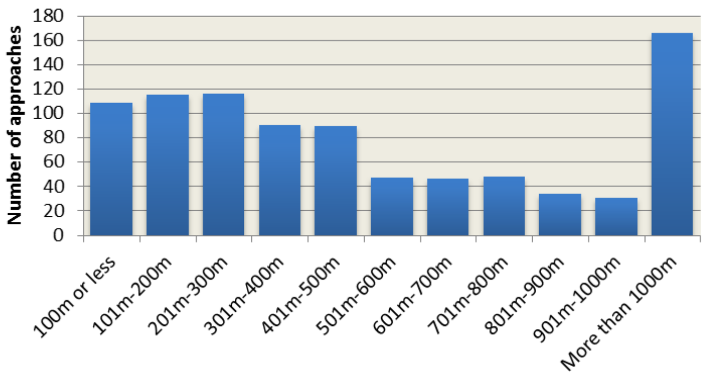
Figure 5.7 Distribution of distance to exit lane merge



5.1.2.6 Distance from upstream intersection

Figure 5.8 indicates a broad range in mid-block lengths upstream of the selected intersections. Twelve percent of approaches were located at or within 100m of an upstream intersection, while 19% were more than 1km away.

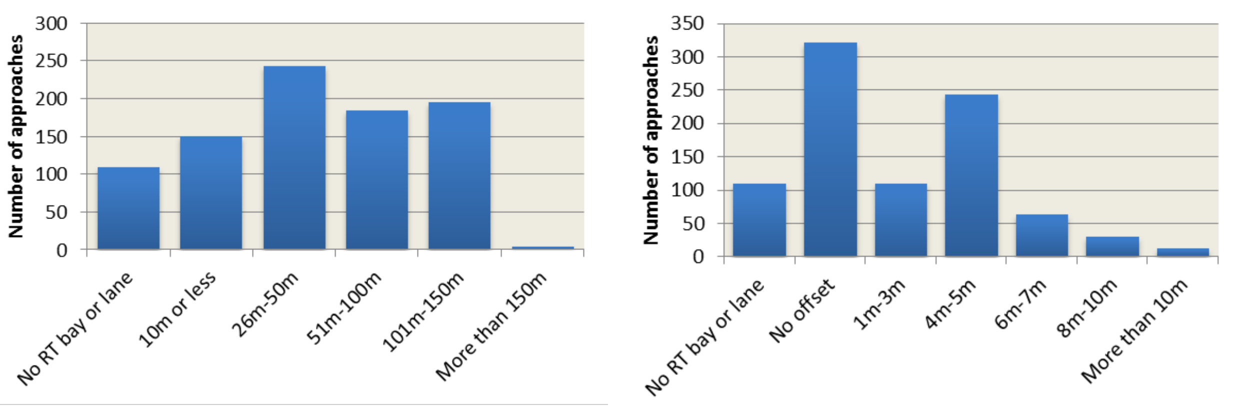
Figure 5.8 Distribution of distance from upstream intersection



5.1.2.7 Right-turning bay or lane

Most right-turning bays/short right-turning lanes at the selected approaches (when present) were observed to be less than 150m long, with 50% of these having lengths of 50m or less. Right-turning bays/lanes at 40% of approaches, where these were present, were observed to be well aligned with a zero offset. Offsets of 4m or 5m were also quite common (31% of approaches), while a few had offsets of more than 10m.

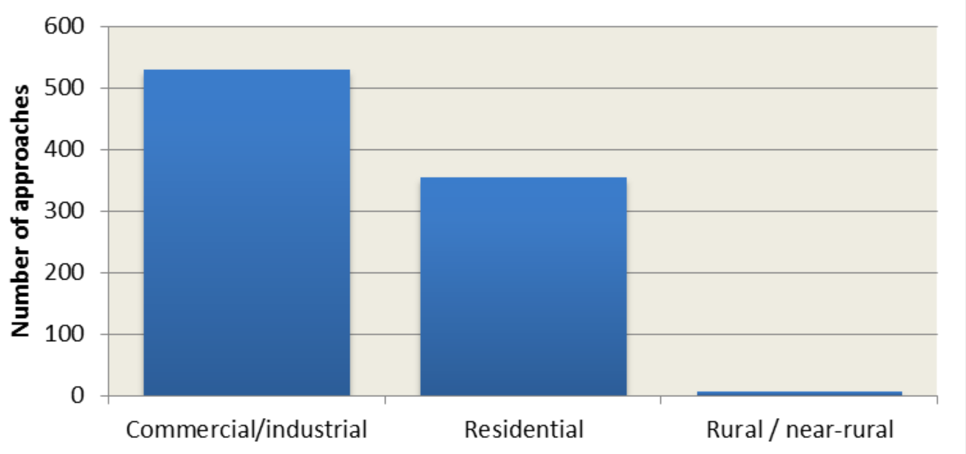
Figure 5.9 Distribution of length and offset of right-turning bay or lane



5.1.2.8 Land use

Sixty percent of the selected signalised intersections were located in areas that had a predominantly commercial/industrial land use. Less than 1% of sites were located in rural areas.

Figure 5.10 Distribution of land use

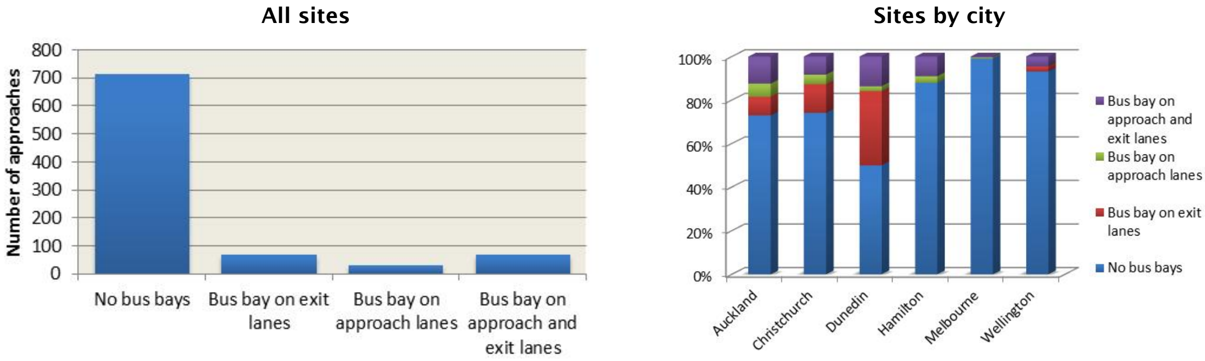


5.1.3 Bus bays and parking

5.1.3.1 Bus bays

Out of the 889 selected approaches, 700 (80%) did not have a bus bay located within 100m on either the entry or exit side of the approach, while 8% of approaches had a bus bay on both the approach as well as the exit. Most bus bays were located on the exit side (8%) rather than on the approach (4%).

Figure 5.11 Presence of bus bays

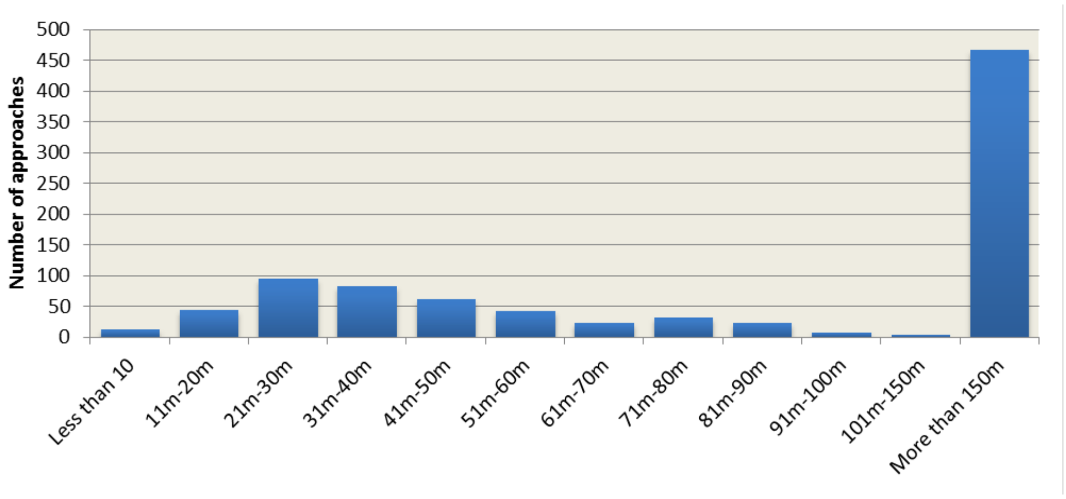


The comparison by city shown in figure 5.11 shows that Dunedin had a higher proportion of approaches with bus bays, while Melbourne did not have specially marked bus bays on almost all of its approaches.

5.1.3.2 Upstream parking

Figure 5.12 shows that upstream parking on the majority of approaches (52%) was located more than 150m from the limit line. Twenty-seven percent of approaches had upstream parking between 20m and 50m, while 6% had parking within 20m of the limit line.

Figure 5.12 Distribution of distance to parking

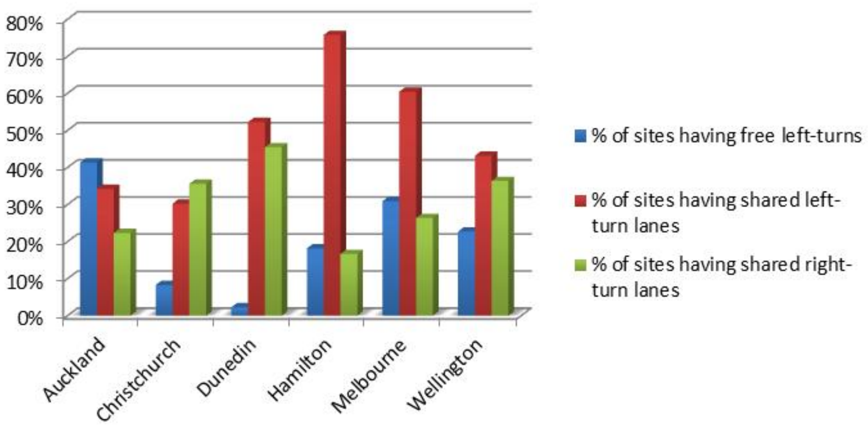


5.1.4 Lane layout

Figure 5.13 illustrates the percentage of sites within each city that had free left turns or shared left-turn/through or right-turn/through lanes. Free left turns for motor vehicles were observed to be quite common at the Auckland (40%) and Melbourne (30%) sites, in contrast to the Dunedin sites, which hardly had any free left turns.

Hamilton, Melbourne and Dunedin had shared lanes for the through/left-turning movement in a majority of cases. Shared lanes for through/right-turning traffic were less common in Auckland, Hamilton and Melbourne.

Figure 5.13 Lane arrangements, by city

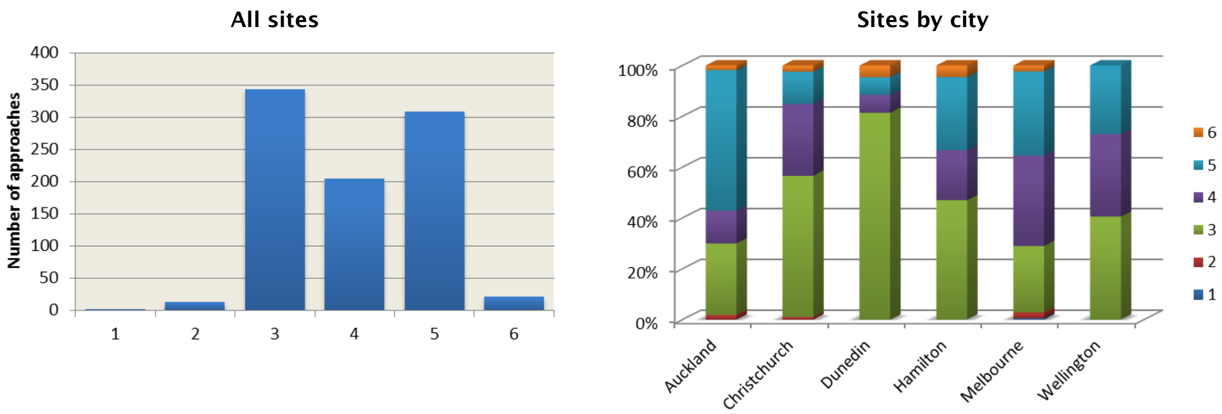


5.1.5 Signal displays

5.1.5.1 Number of unique signal displays

Most approaches had between 3 and 5 traffic signal displays visible to drivers approaching the intersection. Very few of the signalised intersection approaches in Dunedin and Christchurch had 5 or more signal displays, while the majority of approaches in Auckland (55%) had 5 signal displays.

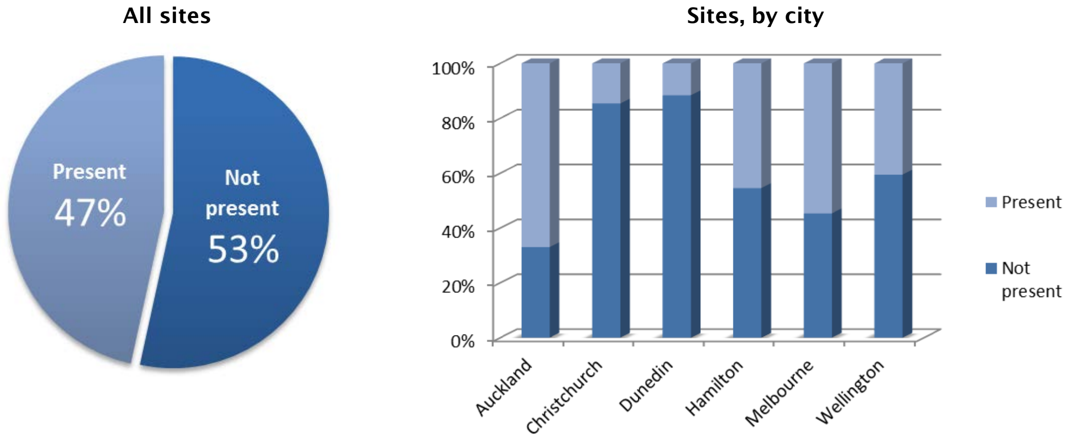
Figure 5.14 Number of signal displays



5.1.5.2 Mast arm on approach

Signal displays mounted on a mast arm were present at 47% of approaches in the sample set. Sixty-seven percent of Auckland approaches had a mast arm on the same side as the approach, compared with only 15% and 12% for Christchurch and Dunedin approaches respectively.

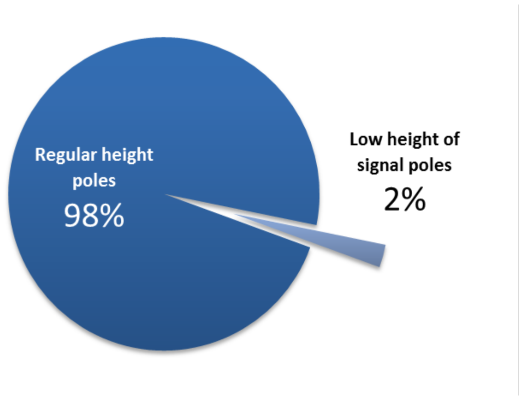
Figure 5.15 Presence of mast arm



5.1.5.3 Height of signal posts

The height of signal posts was observed to be uniformly consistent between sites throughout the different cities. However, 2% of approaches were found to have signal display posts that were significantly shorter than the norm. These were mostly located in dense commercial areas where the height of the post was constrained by shop canopies or other space-limiting architecture.

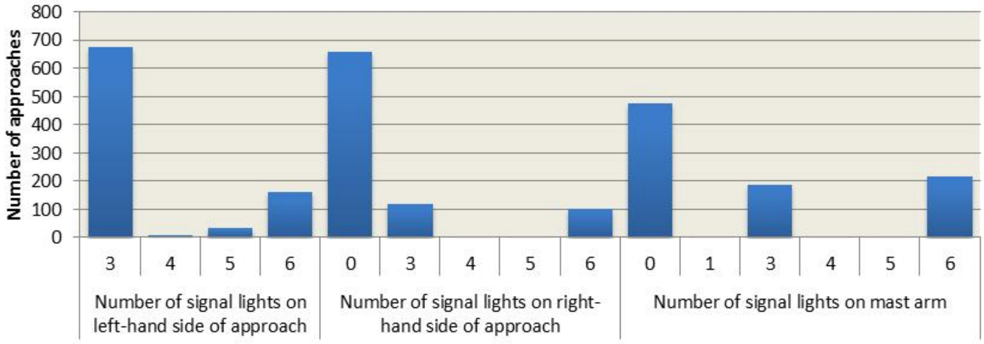
Figure 5.16 Height of signal posts



5.1.5.4 Number of aspects on signal displays

Figure 5.17 depicts the number of signal aspects on signal displays located on the left and right side of the approach, and on mast arms (if present). The majority of signal displays were observed to have either 3 or 6 aspects, depending on the kind of phasing sequence at the intersection.

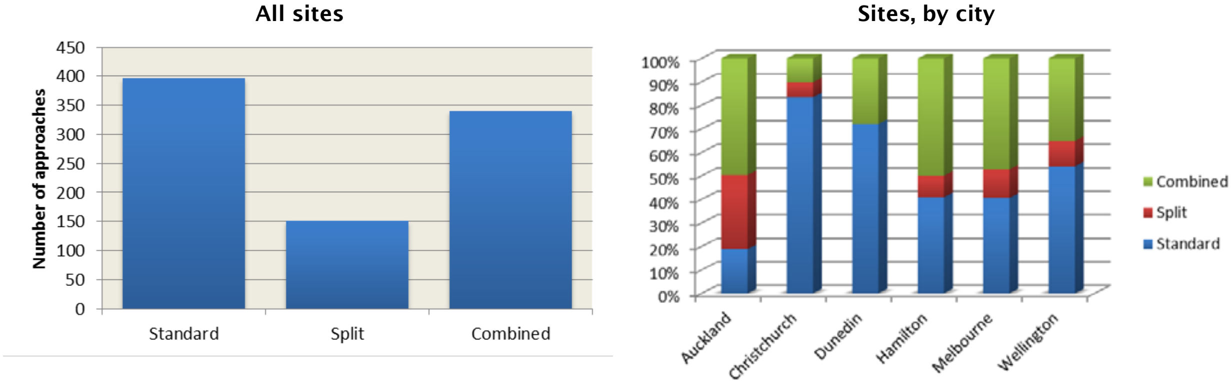
Figure 5.17 Number of signal lamps



5.1.6 Signal phasing

Figure 5.18 plots the phasing types against the corresponding number of approaches for each. The majority of approaches (43%) had a standard phasing sequence involving opposite approaches being given the green signal simultaneously. Seventeen percent of approaches had a split phasing sequence, while 40% operated as both standard as well as split phases (referred to as combined phasing) depending on traffic volume and time of day.

Figure 5.18 Distribution of phase type

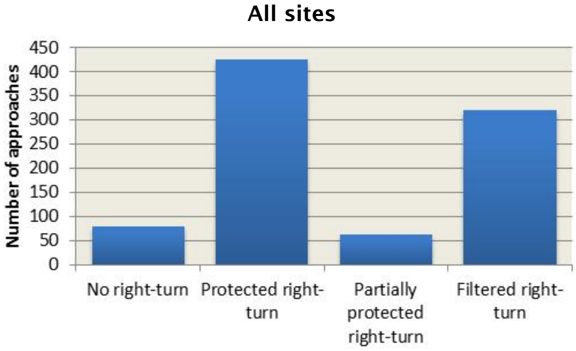


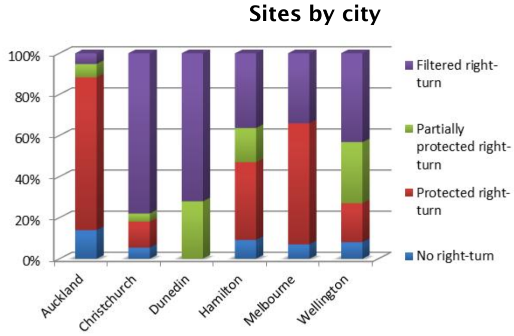
The comparison by city shown in figure 5.18 indicates that 83% and 72% of approaches in Christchurch and Dunedin respectively operated using the standard phasing sequence. Thirty-one percent of approaches in Auckland had split phasing, while approximately 50% of Auckland, Hamilton and Melbourne approaches could operate in both standard and split phasing sequences.

5.1.6.1 Type of right-turn phase

The majority of approaches in the sample set had either a fully protected right-turning phase (48%) or a filtered right turn (36%). The proportion of approaches with a partially protected right turn or those not having any right-turn movement was relatively low, at 7% and 9% respectively.

Figure 5.19 Distribution of right turn phase type



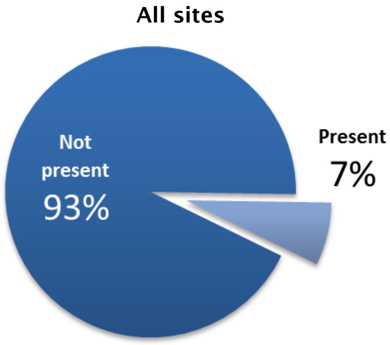


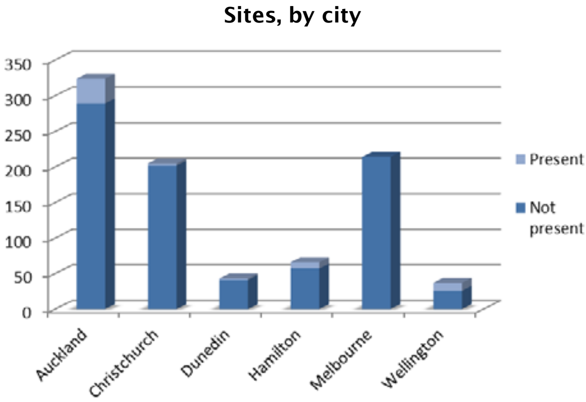
The second part of figure 5.19 shows that right-turn phases in Auckland and Melbourne were primarily fully protected, which reflects the larger intersection size and higher traffic volumes at intersections selected from these cities, while those in Christchurch and Dunedin were filtered. Dunedin, Hamilton and Wellington had higher proportions of partially protected right turns, compared with the other cities. The Hamilton sites had similar proportions of fully protected (38%) and filtered (36%) right turns.

5.1.6.2 Advanced detector loops

Advanced detector loops were located at 7% of approaches in the sample set. Use of advanced detector loops was found to be more common at approaches in Auckland, Wellington and Hamilton.

Figure 5.20 Presence of advanced detector loops



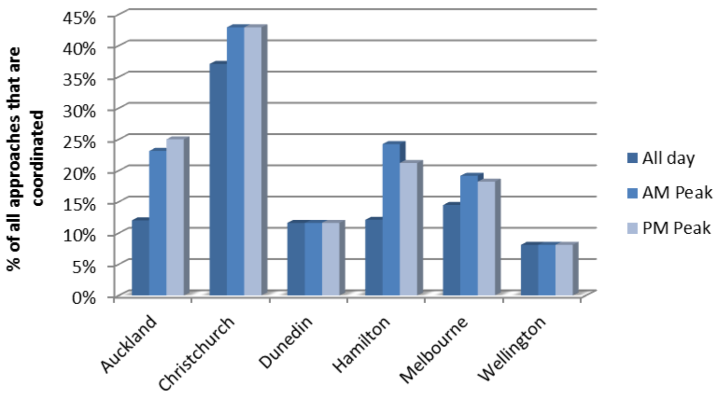


5.1.6.3 Coordination with upstream intersection

Coordination with upstream intersections was assessed using SCATS® 'married'/'divorced' data, along with type and distance of the upstream intersection. Figure 5.21 shows that compared with intersections in other cities, a larger percentage of Christchurch intersections (38%) remained coordinated during the whole day as well as during the AM and PM peak periods (44%).

A number of Auckland and Hamilton intersections appeared to be set up for coordination during peak periods only. In contrast, coordinated signalised intersections in Dunedin and Wellington remained in that state throughout the day. While 16% of Melbourne intersections remained coordinated for the whole day, some intersections were coordinated during the AM (20%) and PM (19%) peak periods only.

Figure 5.21 Coordination with upstream intersection

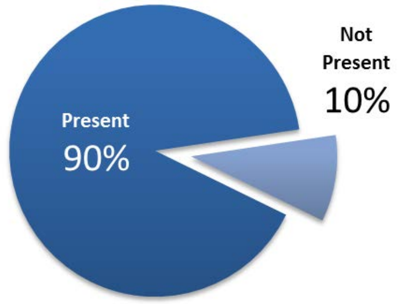


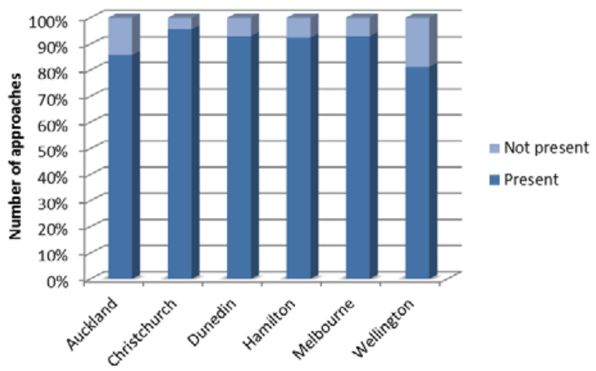
5.1.7 Facilities for pedestrians and cyclists

5.1.7.1 Pedestrian crossings

A pedestrian crossing was present on 90% of the approaches in the sample set.

Figure 5.22 Presence of a pedestrian crossing



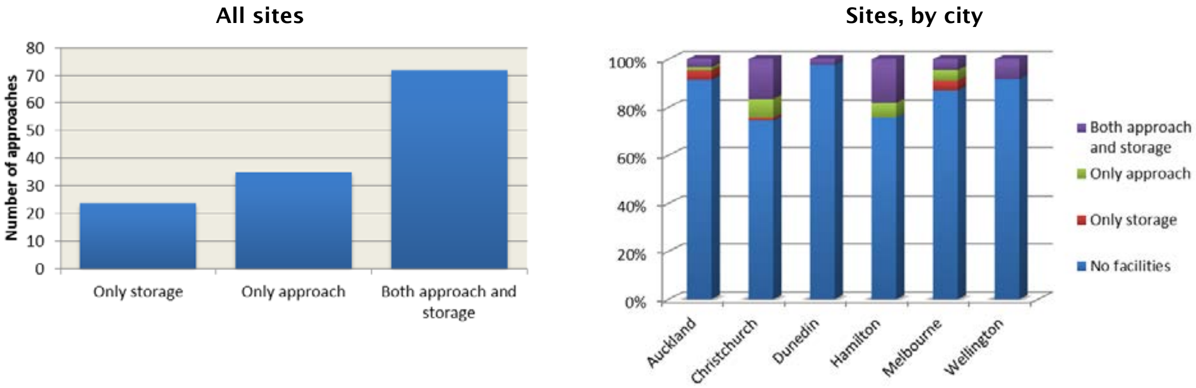


5.1.7.2 Cycle facilities

Cycle facilities were present at only 131 (15%) of approaches in the sample set. Of the approaches where cycle facilities were present, 3% had only storage facilities for cyclists (such as advanced or expanded storage boxes for cyclists), 4% had cycle lanes, while 8% had both cycle lanes as well as storage facilities.

On a city level, signalised intersections in Christchurch and Hamilton were observed to be more cycle friendly, with a larger proportion of sites providing facilities for cyclists.

Figure 5.23 Provision of cycle facilities



5.2 Variable correlations

Table 5.1 explains significant correlations between variables. Identification of correlated variables is required to avoid having two or more significantly correlated variables in the same prediction model, since in such cases the variability in one variable does, to a certain extent, predict the variability in the correlated variable.

The full correlation matrix for all variables is attached as appendix C of this report.

Table 5.1 Variable correlations

Variable 1	Variable 2	Correlation factor	Comments
Signal phasing			
Phasin_std	RT_fprot	-0.87	Approaches with standard phasing, most commonly filtered right turns and not fully protected right turns.
Phasin_std	RT_filter	0.81	
RT_fprot	RT_filter	-0.75	
Phasin_std	Phasin_cmb	-0.71	
Phasin_cmb	Lamps_mastarm	0.57	A mast arm is often provided at approaches with a ‘combined’ phasing sequence.
Phasin_cmb	RT_filter	-0.57	Fully protected right turns are more commonly used at intersections that operate in the ‘combined’ phasing sequence.
Phasin_cmb	RT_fprot	0.54	
Phasin_cmb	Num_signal_displays	0.49	More signal displays are present at approaches with a combination of standard and split phasing.
Phasin_split	RT_fprot	0.46	Right turns (when present) are fully protected in a split phasing sequence.
RT_fprot	Cyctime_day	0.45	Intersections with fully protected right turns usually have higher cycle times.
RT_filter	Cyctime_day	-0.45	Intersections with filtered right turns usually have lower cycle times.
Phasin_std	Lamps_mastarm	-0.43	Approaches with the standard phasing sequence do not generally have mast arms.
Phasin_std	Deg_sat	-0.38	Standard phasing is usually associated with approaches that have a lower degree of saturation.

Variable 1	Variable 2	Correlation factor	Comments
Intersection geometry and layout			
Num_app_lanes	App_width	0.93	More approach lanes results in increased approach width. Approaches with more lanes for through traffic usually have more exit lanes. Larger intersections, ie those with multiple through lanes, also have multiple exit lanes.
Num_thr_lanes	Num_exit_lanes	0.72	
Num_app_lanes	Num_thr_lanes	0.71	
Num_thr_lanes	App_width	0.69	
Num_app_lanes	Num_exit_lanes	0.67	
App_width	Num_exit_lanes	0.66	
Num_exit_lanes	q_app	0.59	Intersections with higher traffic volumes are generally provided with more approach and exit lanes.
Num_app_lanes	q_app	0.58	
App_width	q_app	0.57	
App_width	Num_signal_displays	0.56	More signal displays are present on wider approaches.
Num_app_lanes	Num_signal_displays	0.56	
Num_thr_lanes	q_app	0.56	More through lanes are provided at approaches with higher traffic volumes.
Med_island	Lamps_right	0.55	Signal displays are generally provided on medians/central islands.
Num_thr_lanes	Num_signal_displays	0.54	More signal displays are present on wider approaches.
App_width	Med_island	0.53	A raised median or central island is often present on wider approaches.
App_width	Lamps_right	0.53	Wider approaches often have a signal display on the right side of the approaching traffic lanes.
Num_exit_lanes	Num_signal_displays	0.51	Intersections with multiple exit lanes have more signal displays.
Num_app_lanes	Med_island	0.50	A raised median or central island is often present on wider approaches.
Cycle and bus facilities			
Cyc_app	Cyc_storage	0.79	Approaches with cycle lanes usually also have storage facilities for cyclists.
Cycle_fac	Cyc_app	0.58	
Cycle_fac	Cyc_storage	0.54	
App_bus	Exit_bus	0.52	Bus bays are often provided on both the entry and exit sides of the approach.
Signal displays			
Mast_arm	Lamps_mastarm	0.90	Presence of a mast arm results in increasing the number of unique signal displays.
Mast_arm	Num_signal_displays	0.75	
Num_signal_displays	Lamps_mastarm	0.74	
Num_signal_displays	q_app	0.53	Busier intersections have more signal displays, and also often have mast arms on approaches.
Mast_arm	q_app	0.45	

Variable 1	Variable 2	Correlation factor	Comments
Signal timing/other			
Green	q_app	0.42	Higher green times are often provided for busier approaches.
Deg_sat	q_app	0.41	Busier approaches are more saturated.
Cycle time	Deg_sat	0.39	No clear relationship between cycle time and degree of saturation.

6 Analysis of SCATS® data

6.1 SCATS® detector loop counts

6.1.1 Analysis methodology

SCATS® detector loop count data was extracted from the SCATS® system for a two-week period during February and March 2010 (see section 4.3 for data collection periods for each city). This data, in the form of detailed text outputs, reflects the number of vehicles that cross each detector loop at a given site, each hour over the two-week period.

SCATS® detector loop counts for each site were imported into an Excel spreadsheet for manipulation. A Visual Basic macro was then used to convert the unformatted raw data into a format suitable for analysis. Vehicle counts were summed by detector and time period (AM peak, PM peak, interpeak and whole day) for each analysis day.

To minimise the amount of missing and erroneous data from some SCATS® detector loops, a single day during the two-week period (Tuesday of the second data collection week, 23 February 2010) was selected for most sites. On this day, 6.4% of all detectors had missing or erroneous data. Data from these erroneous detectors was removed from the final sample set, and replaced with scaled data from other days where available.

The detector counts were subsequently converted to turning movement (left, through and right) counts. SCATS® signal layout diagrams were used to understand the lanes/movements that were collected by each detector. Some detectors were located within lanes where shared movements were present (for example, a shared through and right-turning lane). Short-duration (15min) traffic count surveys for each peak period were used to split the detector counts for these 'shared' lanes into the associated movement counts.

As a final step, the SCATS® counts were scaled back to 2006 (the midpoint of the crash period 2004–2008). The annual growth factors used for each city are shown in table 6.1. Hence, the 2006 counts were lower than the 2010 counts obtained from SCATS®.

Table 6.1 Growth factors used (VicRoads website www.vicroads.vic.gov.au)

City	Annual growth rate
Auckland City	1.5%
Manukau, North Shore, Waitakere	2.5%
Christchurch	2.0%
Dunedin	1.5%
Hamilton	2.0%
Melbourne#	1.5%
Wellington	2.0%

6.1.2 Error in SCATS® detector loop counts

SCATS® detector loop counts have been shown to underreport actual vehicle volumes. This under-reporting often occurs during periods of high congestion, where gaps between vehicles are small and vehicles travel at slow speeds.

To understand this error, manual traffic count datasets were collected in Christchurch and Melbourne in February 2010, and compared with SCATS® counts. Figure 6.1 shows the comparison between the raw SCATS® data and the manual traffic counts.

Figure 6.1 Comparison between manual counts and SCATS® counts, by movement

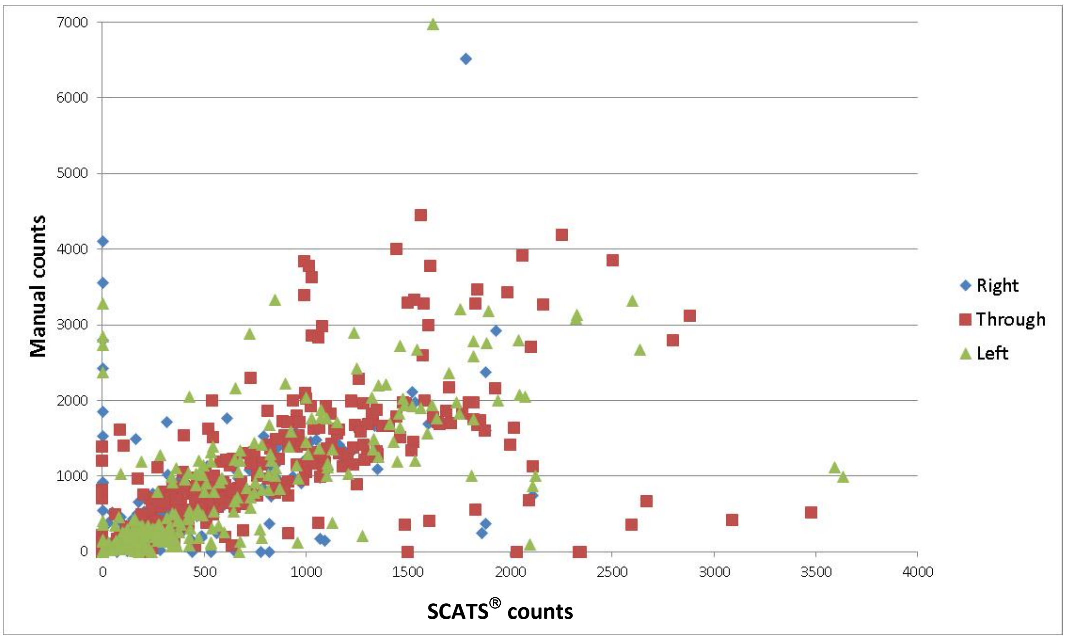
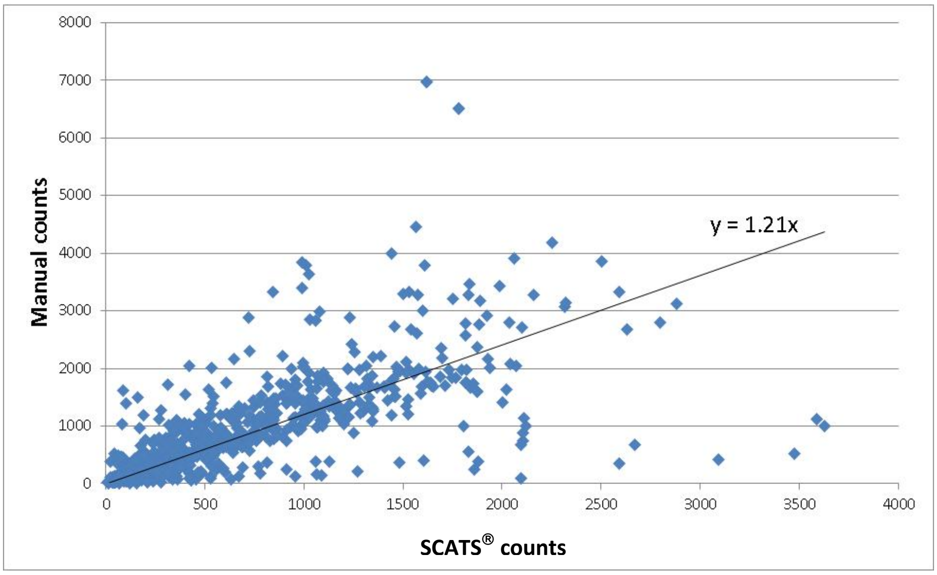


Figure 6.1 demonstrates the large scatter between the manual counts and the SCATS® counts. Right-turning movements show the greatest scatter, while left-turning movements show a cluster below a SCATS® count of 500 vehicles. Cases where manual counts or SCATS® counts were zero were removed from the error analysis. These were points where either survey results were inaccurate, or the SCATS® data contained errors.

Data was initially split into two bins – SCATS® counts less than, or greater than, 1000. An error-scaling factor for the 'less than 1000 vehicles' bin was found to fit the data well. However, the factor for cases where the SCATS® counts were greater than, or equal to, 1000 did not produce a good fit. It is believed that detector loop error increases significantly as vehicle volumes increase.

As a result, these bins were removed and the data was analysed as a complete set. The resulting plot with a linear fitted function is shown in figure 6.2.

Figure 6.2 Comparison between manual counts and SCATS® counts



The results show that on average, SCATS® counts needed to be scaled up by 21%. The SCATS® traffic volume data was thus increased by a factor of 1.21 to reflect the actual volume of motor vehicles more accurately.

6.2 SCATS® IDM data

6.2.1 Analysis methodology

SCATS® IDM data contains a detailed record of the operation of the signalised intersection, including the phases that are called, times at which they are called, green, yellow and all-red times. As discussed in section 6.1.1, IDM data for the Tuesday in the second data collection week was imported into an Excel spreadsheet for manipulation.

A Visual Basic macro was used to extract relevant information on signal operation from the IDM data. The macro picked up keywords in the IDM file and extracted the associated time within each phase at which the signal turned green, yellow or red. The total green, yellow and all-red times were thus recorded over each time period (AM peak, PM peak, interpeak and whole day) for each phase. The number of times that a phase was called within a period was also recorded. The detailed phase-by-phase information was then summarised for each analysis period.

The average green time for the through movement on each approach was calculated from the SCATS® data. This green time for the through movement did not include the extra green time that was available to phases allowing left- or right-turning movements. However, the overall cycle time of the intersection included green times for all movements, whether through, left or right turning.

The average yellow and all-red times for an approach (for each period) were also extracted from the IDM data for each time period.

6.2.2 Outputs: signal timing

Figure 6.3 shows the distribution of green time for the through movement for all selected sites, by period. 65% of approaches had an average green time less than or equal to 40s. The PM peak was further skewed to the right, with higher average green times.

Figure 6.3 Distribution of green time

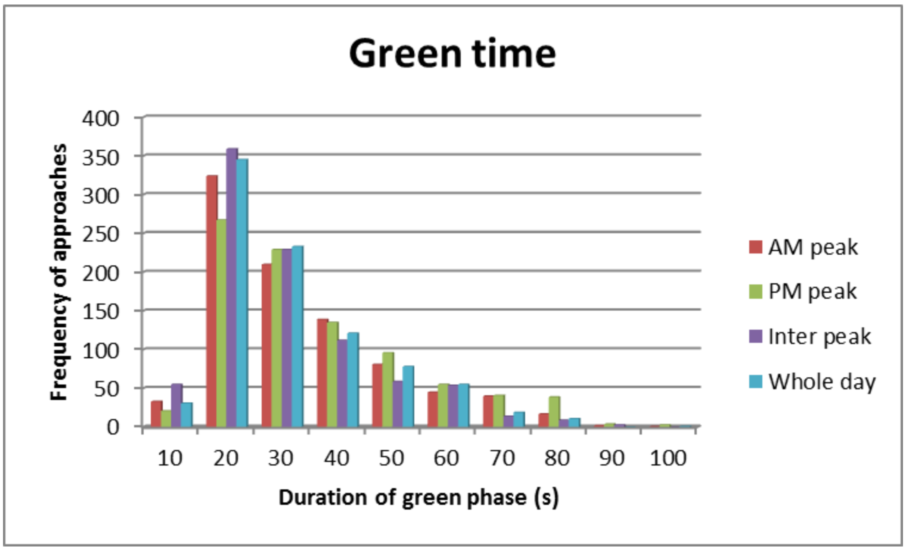


Figure 6.4 shows the distribution of yellow time for all selected sites, by period. The figure shows that 95% of approaches had yellow time greater than or equal to 4s. There was little difference between the periods.

Figure 6.4 Distribution of yellow time

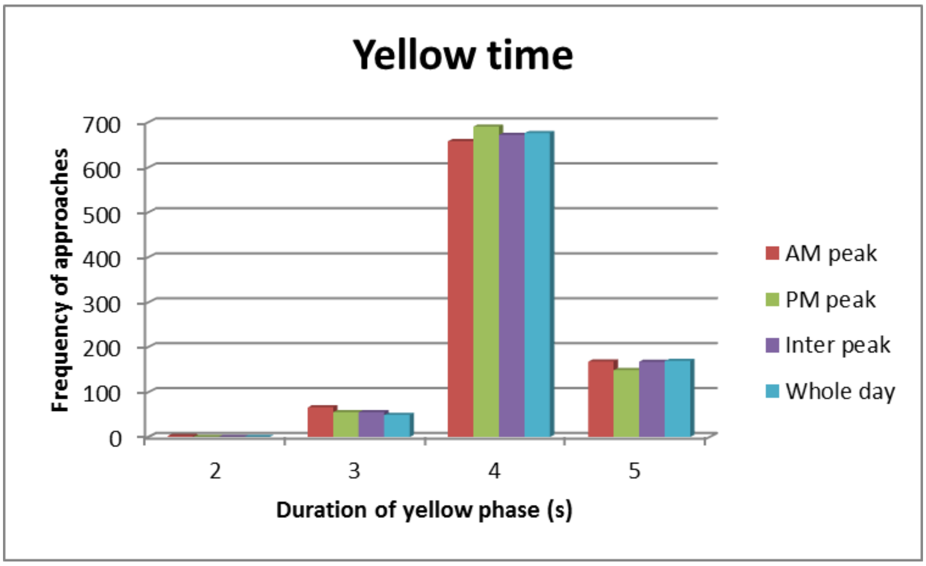


Figure 6.5 shows the variation in yellow time, by city. A larger proportion of sites in Hamilton, Auckland and Wellington are seen to have yellow times of 5 seconds.

Figure 6.5 Distribution of yellow time, by city (seconds)

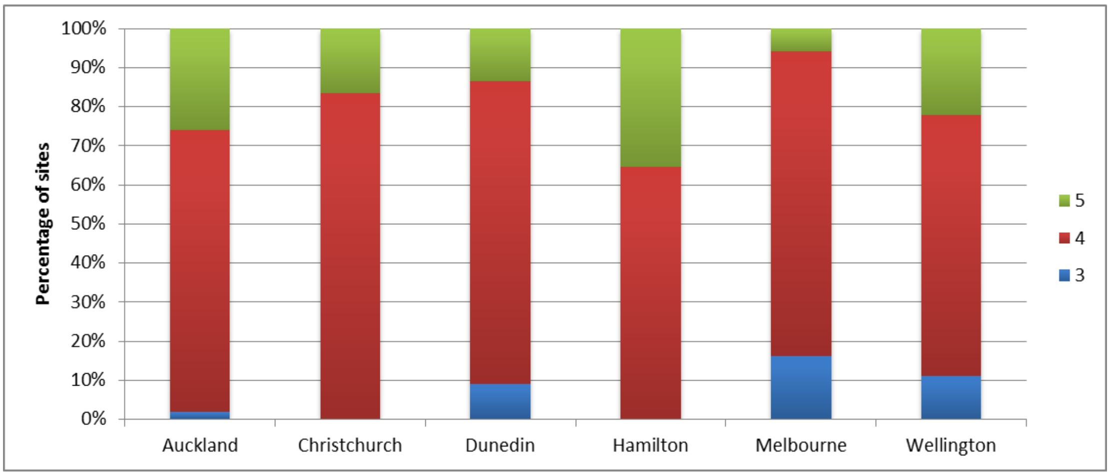


Figure 6.7 plots the distribution of all-red time for all selected sites, by period. The figure shows that AM and PM peak periods had shorter all-red times than other periods.

Figure 6.6 Distribution of all-red time

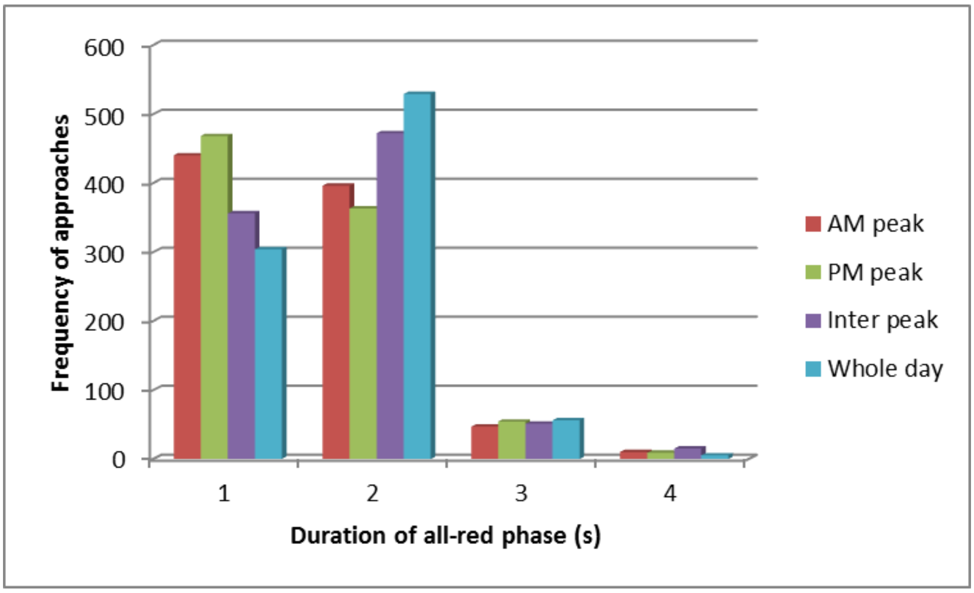
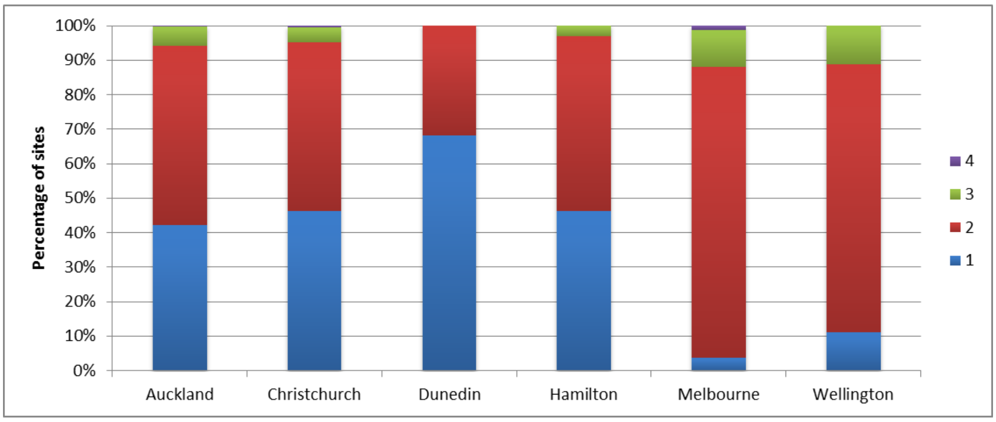


Figure 6.58 shows the variation in all-red time, by city. With the exception of Hamilton and Dunedin, a small proportion of sites in the other cities are observed to have all-red times of three seconds or longer. This indicates that there seems to be a policy of reducing lost time during peak periods to improve signal throughput.

Figure 6.7 Distribution of all-red time, by city (seconds)



6.2.3 Degree of saturation

Degree of saturation is the ratio of the average number of vehicles through an approach in a set period of time divided by the capacity of that approach. This is calculated using equation 6.1 below. A degree of saturation of 1 occurs when the number of vehicles using an approach is equal to the link capacity. For a signalised intersection the capacity is related to the green time allocated to that approach.

$$DS = \frac{V}{(\%GT \times Lanes) \times Cap} \qquad \text{(Equation 6.1)}$$

Where:

DS = degree of saturation

V = SCATS® vehicle count per hour (all the through lanes on the approach)

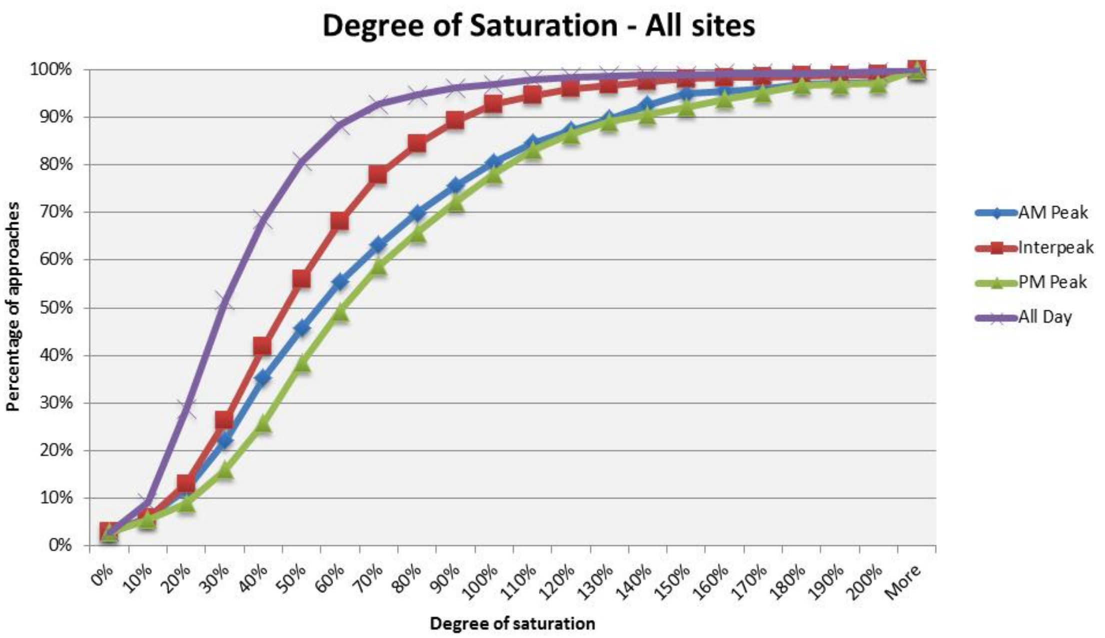
%GT = percentage of green time versus total cycle time (for the through lanes)

Lanes = number of through lanes

Cap = link capacity (assumed as 1700pcus/hour/lane) (passenger car units).

Figure 6.8 shows the degree of saturation for all selected sites, by period. The figure shows that almost 20% of the selected intersection approaches were over-saturated in the AM and PM peak periods. This figure drops to less than 10% for the interpeak and all-day periods.

Figure 6.8 Degree of saturation – all sites



Figures 6.9, 6.11 and 6.12 show the degree of saturation by city for the all-day, AM and PM peak periods. The figures show that intersection approaches located in Auckland and Melbourne were the most congested, while those in Dunedin and Christchurch were the least congested.

Figure 6.9 Degree of saturation – all day

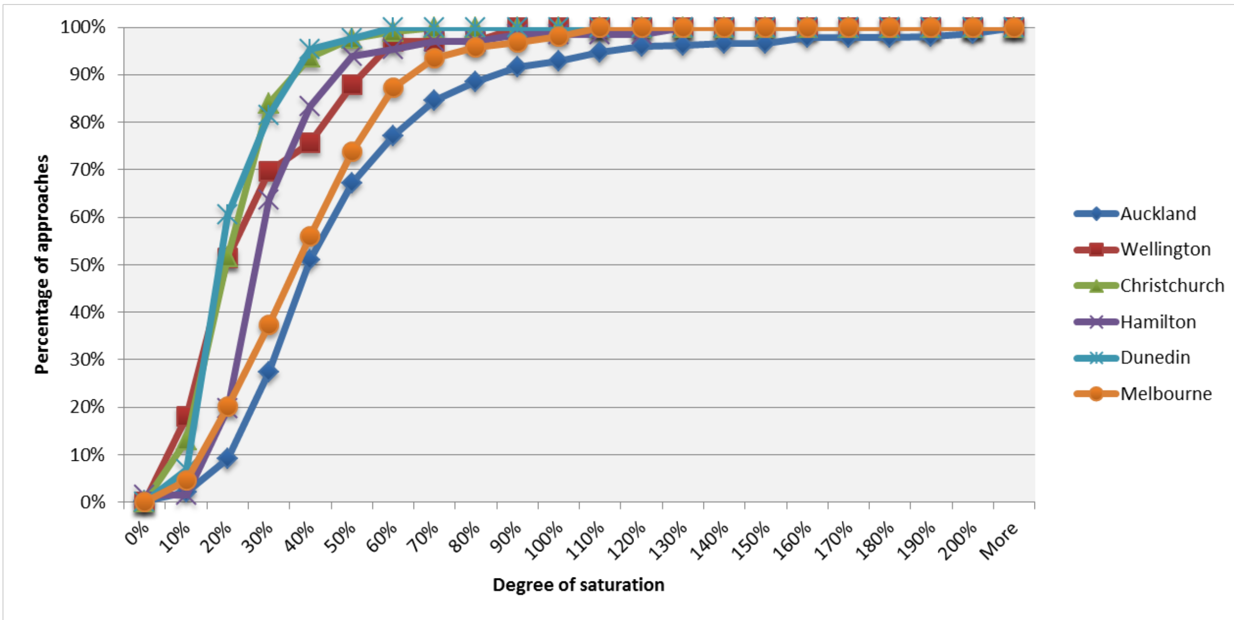


Figure 6.10 Degree of saturation – AM peak, by city

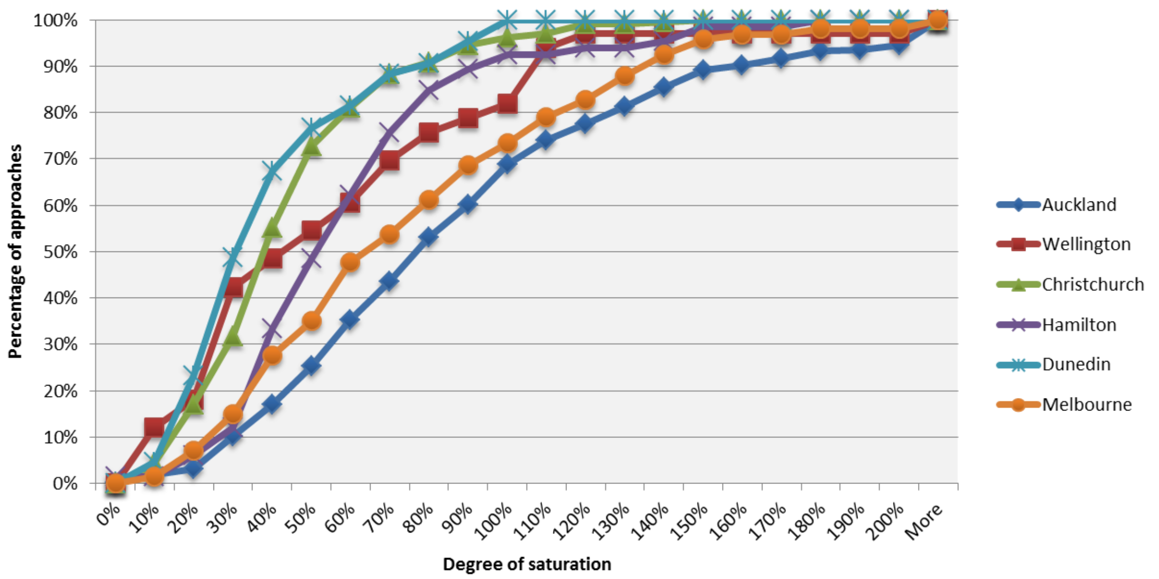
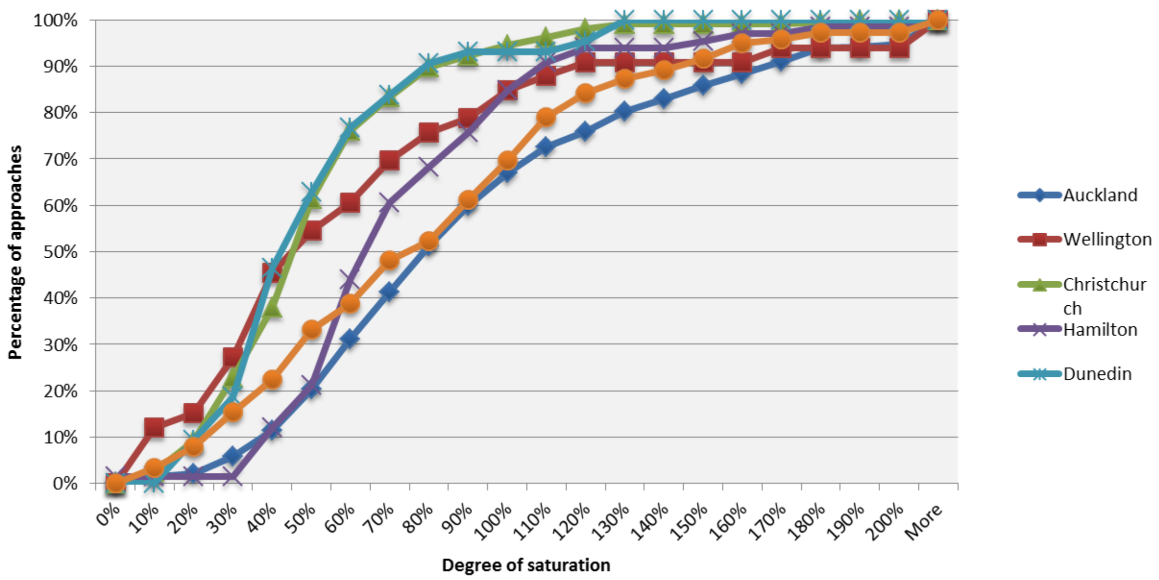


Figure 6.11 Degree of saturation: PM peak, by city



6.3 Pedestrian statistics from SCATS®

6.3.1 Analysis methodology

Data on the proportion of pedestrian phases called per time period were also extracted from the SCATS® IDM data. The IDM data records every incidence when a pedestrian crossing phase is called. Although this does not correspond to the number of crossing pedestrians, the IDM data can be used as a surrogate for the pedestrian volume across an approach.

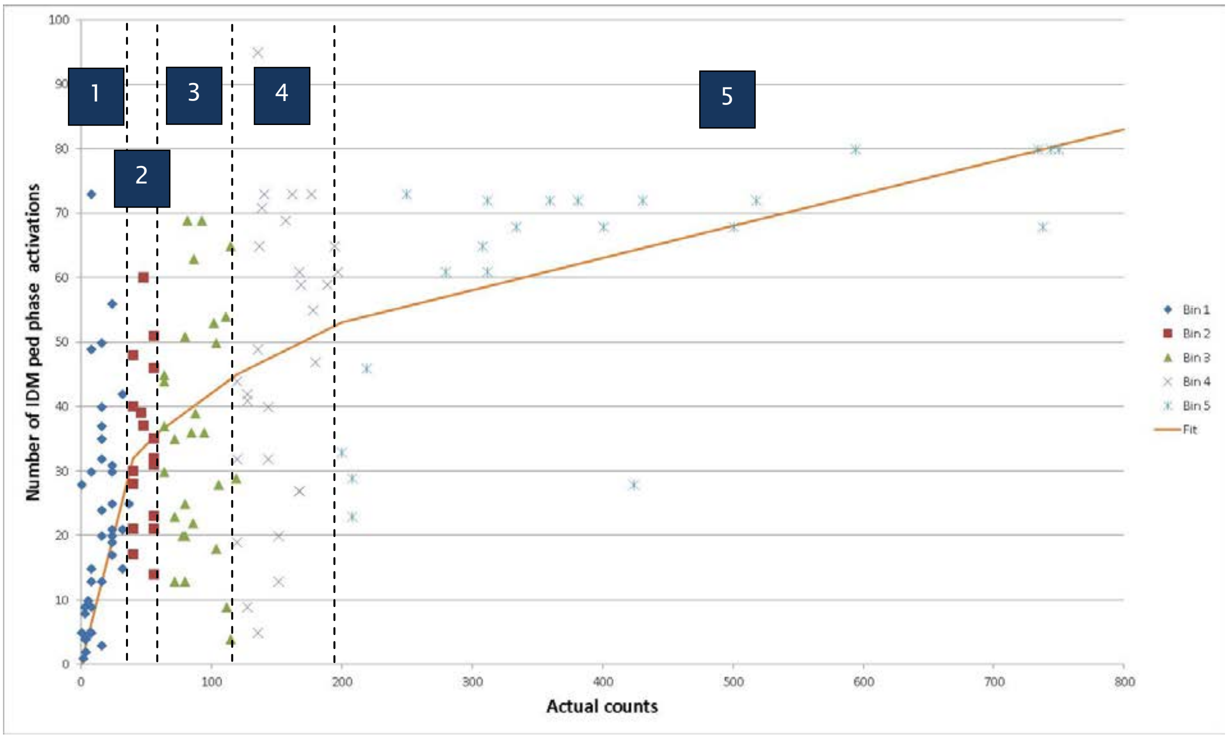
To develop this relationship between the proportion of pedestrian phases called and pedestrian volume, pedestrian count surveys were undertaken and SCATS® data on proportion of pedestrian phases called was collected at 11 sites in Dunedin and 9 sites in Melbourne. Pedestrian count data from these intersections was then plotted against the frequency of occurrence of the pedestrian phase for these sites, as indicated by the SCATS® IDM data. This comparison is shown in figure 6.12. The pedestrian counts were subsequently grouped into five bins corresponding to the level of pedestrian usage: low, medium-low, medium, medium-high and high. Each bin comprised one-fifth of the data. The start and end pedestrian flows within each range are shown in table 6.2.

Table 6.2 Range of pedestrian usage bins

Bins	Starting pedestrian flow per hr	Ending pedestrian flow per hr
1	0	40
2	41	60
3	61	120
4	121	200
5	201 or greater	-

A non-increasing continuous linear piecewise function was produced to show how each pedestrian flow bin relates to the IDM pedestrian phase data. The developed function is also shown in figure 6.12.

Figure 6.12 Comparison between IDM pedestrian phase data and actual pedestrian counts

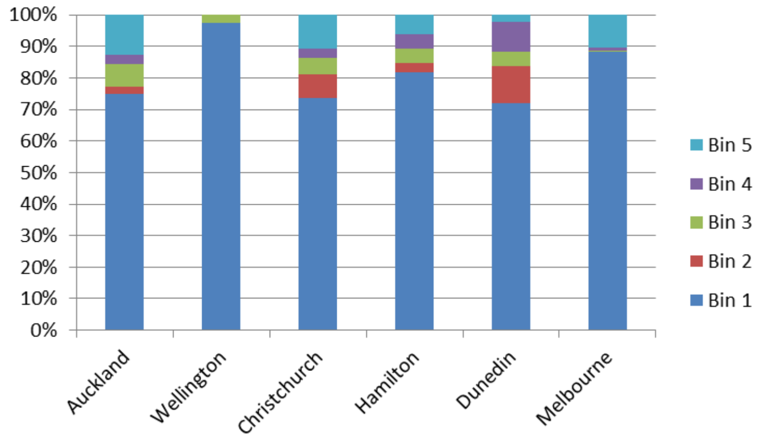


The function shown in figure 6.12 was finally used to classify the selected sites (through IDM pedestrian data for each) into one of the pedestrian usage bins. These bins were subsequently used for modelling.

6.3.2 Outputs

Figure 6.13 depicts the level of pedestrian usage (through classification in pedestrian usage bins) at the selected intersections. Most selected intersections were at low pedestrian-usage sites, although Auckland, Christchurch and Melbourne had a number of sites with heavy pedestrian demands.

Figure 6.13 Pedestrian usage at intersections, by city



7 Crash data

Crash data for the selected New Zealand and Australian sites was collected for the five-year period from January 2004 to December 2008. New Zealand crash data was extracted from the national Crash Analysis System (CAS) database maintained by the NZTA. Crash data for the Melbourne sites was provided by VicRoads from the Victoria Crash Stats database. Although crash data for 2009 had been uploaded on to the CAS database in time for the data analysis phase of this study, the data could not be included because at the time, 2009 crash data for Melbourne was not yet available.

7.1 Crash data, by year

Crash data for the selected New Zealand sites was available from 1992 onwards, due to the relative ease of extracting this information from CAS. Figure 7.1 illustrates the number of crashes, by severity and by year.

Figure 7.1 Crashes at New Zealand sites, by year

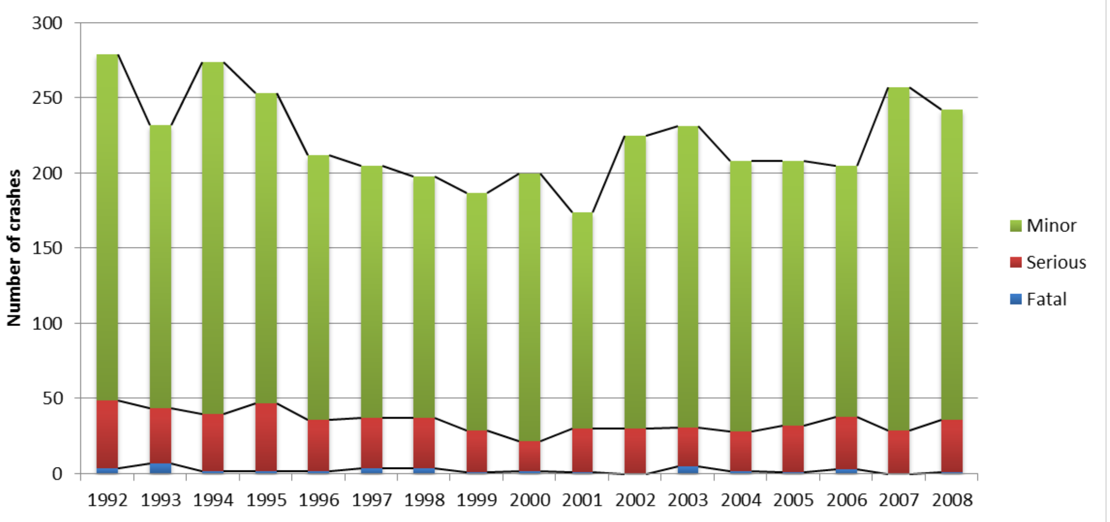


Figure 7.2 depicts crashes, by severity, for the selected Melbourne sites, for the 2004–2008 period.

Figure 7.2 Crashes at Melbourne sites, by year

7.2 Crash data: whole day

7.2.1 Motor vehicle crashes

Table 7.1 shows the main crash types at signalised intersections in New Zealand and Australia, and provides the number of crashes included in the sample set for each.

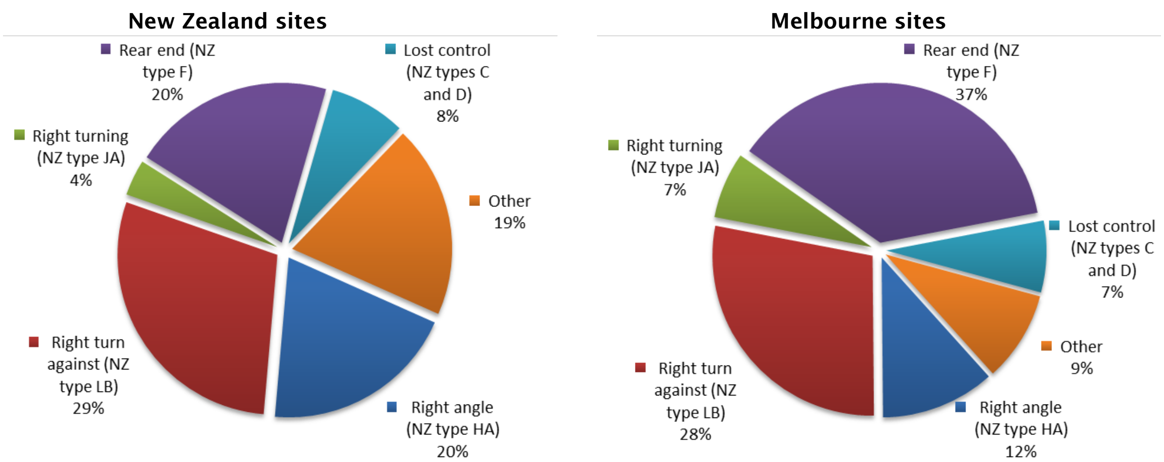
Table 7.1 Motor vehicle crashes at selected sites (2004–2008)

Crash type[a]	Number of crashes (NZ sites)	Number of crashes (Melbourne sites)
Right-angle (NZ type HA)	179	43
Right-turn-against (NZ type LB)	263	105
Right-turning (NZ type JA)	33	25
Rear-end (NZ type F)	186	139
Lost-control (NZ types C and D)	70	27
Other	177	34
Total injury crashes	**908**	**373**

a) See appendix D for description of crash movement codes.

Figure 7.3 illustrates the proportion of crashes, by type, for the New Zealand and Melbourne sites. Right-turn-against and rear-end crashes formed a large proportion of all crashes in both centres, although the New Zealand sites seemed to have a higher proportion of right-angle crashes (20%) than Melbourne (12%). However, Melbourne had almost double the number of rear-end crashes (37%) than New Zealand (20%).

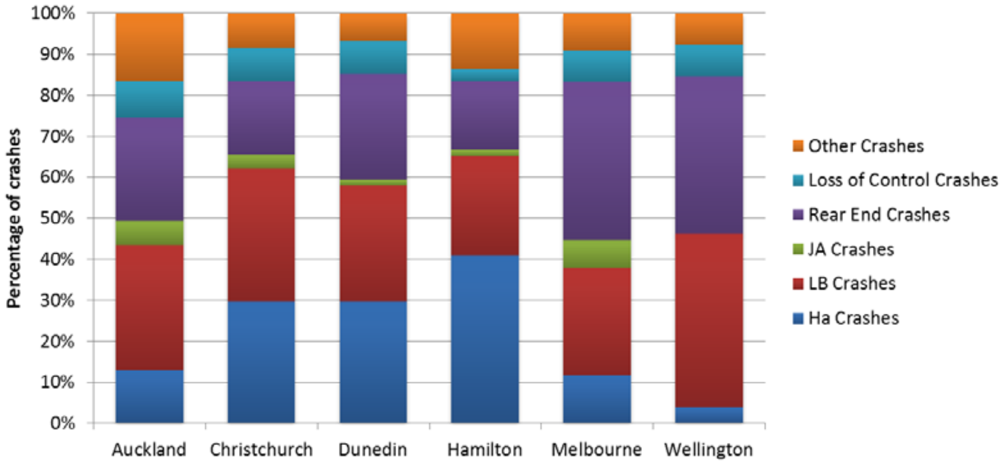
Figure 7.3 Proportion of motor vehicle crashes, by type (2004–2008)



7.2.2 Motor vehicle crashes, by city

The proportion of crashes in each crash type is observed to vary by city, as illustrated in figure 7.4.

Figure 7.4 Proportion of motor vehicle crash types, by city



7.2.3 Cycle-motor vehicle crashes

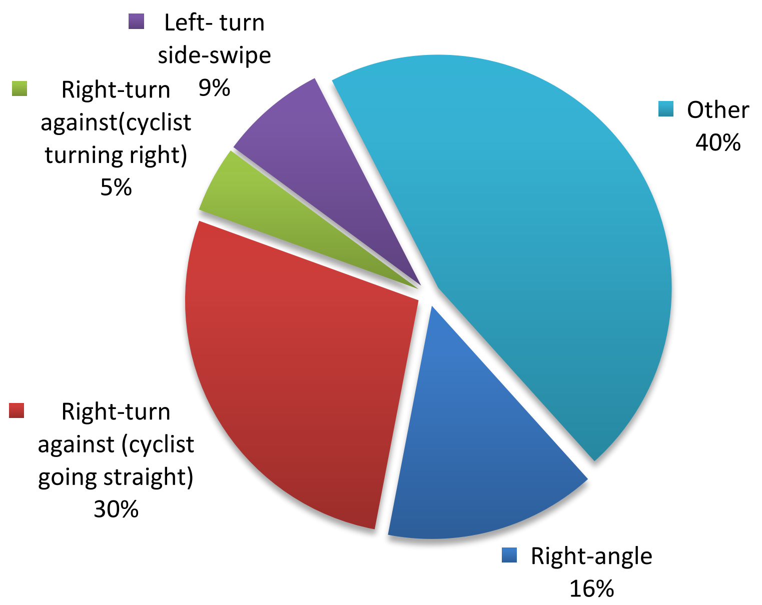
Table 7.2 shows the number of cycle-motor vehicle crashes, by type, at the selected intersections. The New Zealand sites had more crashes than Melbourne; however, this was expected because of the higher cyclist numbers seen at some of the New Zealand sites than in Melbourne.

Table 7.2 Cycle-vehicle crashes at selected sites (2004-2008)

Crash type	Number of crashes (NZ sites)	Number of crashes (Melbourne sites)
Right-angle (NZ type HA)	15	1
Right-turn-against (cyclist going straight) (NZ type LB)	28	2
Right-turn-against (cyclist turning right) (NZ type LB)	5	0
Left-turn side-swipe (NZ type GB, AC)	8	N/A
Other	38	12
Total injury crashes	**94**	**15**

Figure 7.5 depicts the proportion of cycle-motor vehicle crashes at all selected sites. Right-turn-against (cycle going straight), right-angle and left-turn side-swipe crashes collectively constituted the majority of all cycle-motor vehicle crashes.

Figure 7.5 Proportion of cycle-motor vehicle crashes, by type (2004-2008)



7.2.4 Pedestrian-motor vehicle crashes

Table 7.3 shows the number of pedestrian-vehicle crashes, by type. Crashes involving right-angle collisions between pedestrians and motor vehicles were the most common crash type. Collisions between pedestrians and right-turning vehicles, although relatively frequent in New Zealand, were not recorded in the Melbourne crash data.

Table 7.3 Pedestrian-vehicle crash types at selected sites (2004-2008)

Crash type	NZ crash code	Number of crashes (NZ sites)	Number of crashes (Melbourne sites)
Right-angle	NA+NB	102	29
Right-turning vehicle	ND+NF	38	0
Total injury crashes		**140**	**29**

Figure 7.6 illustrates the proportions of pedestrian-vehicle crashes, by type.

Figure 7.6 Pedestrian-vehicle crashes, by type (2004-2008)

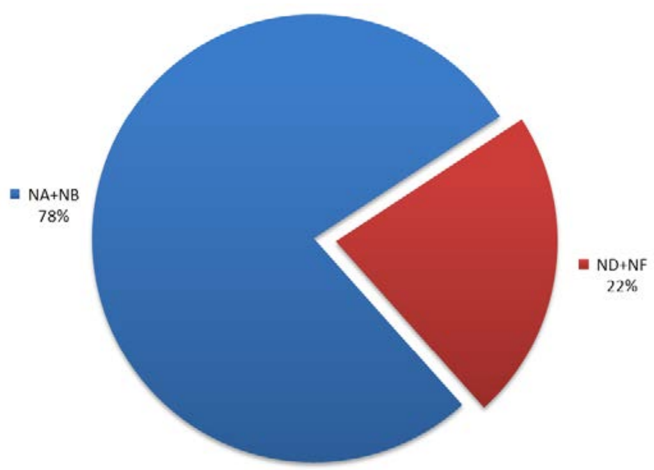
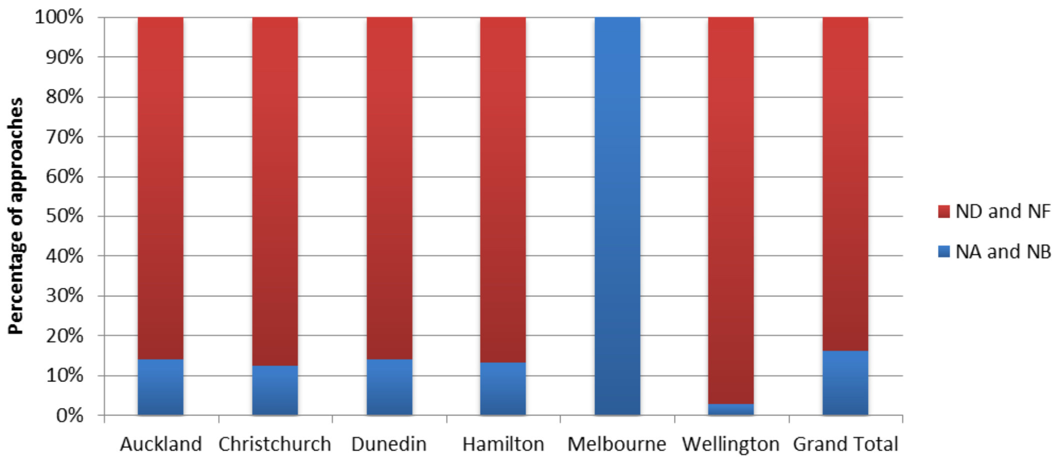


Figure 7.7 Proportion of pedestrian-vehicle crash types, by city



7.3 Peak-period crashes

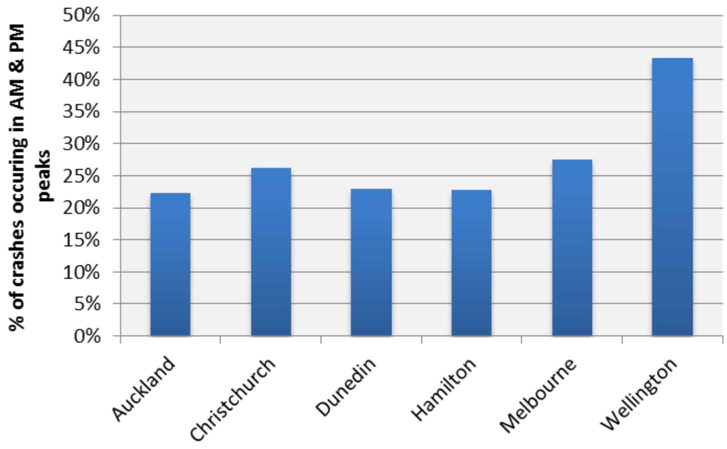
Table 7.4 shows the percentage of crashes for each of the major signalised intersection crash types that occurred during the AM (7–9) and PM (4–6) peak periods. While between 18% and 23% of right-angle, right-turn-against, right-turning and loss-of-control crashes took place during peak periods, the occurrence of rear-end (35%) and loss-of-control (27%) crashes was especially high during peak periods.

Table 7.4 Percentage of crashes occurring in AM and PM peak periods, by type (2004–2008)

Crash Type	Number of crashes: All-Day	Number of crashes: AM & PM peaks	% of crashes occuring in AM & PM peaks
Right angle (NZ type HA)	217	39	18%
Right turn against (NZ type LB)	344	80	23%
Right turning (NZ type JA)	57	11	19%
Rear end (NZ type F)	313	111	35%
Lost control (NZ types C and D)	96	26	27%
Other	135	29	21%

Out of all the various cities, the Wellington intersections (which all lie on state highways) had the highest rate of peak-period crashes.

Figure 7.8 Percentage of crashes occurring in AM and PM peak periods, by city (2004–2008)



7.4 Daytime/night-time crashes

Figure 7.9 illustrates the proportion of daytime and night-time crashes at the New Zealand and Melbourne sites. The proportion was similar in both countries, with around 36%–37% of all crashes occurring in dark conditions or at twilight.

Figure 7.9 Proportion of daytime and night-time crashes

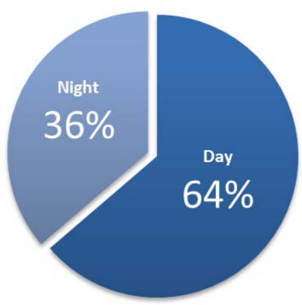
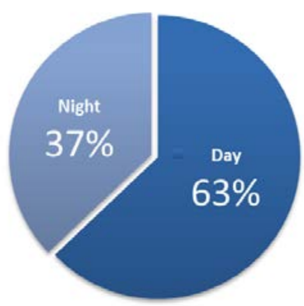
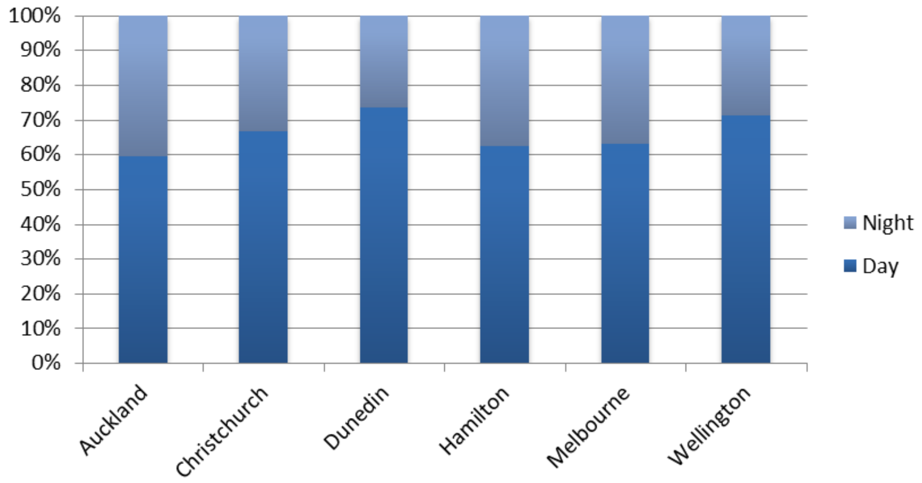
NZ sites	Melbourne sites
	

Figure 7.10 presents a further breakdown of daytime/night-time crashes, by city. The Auckland, Hamilton and Melbourne sites had higher proportions of night-time crashes than the other cities.

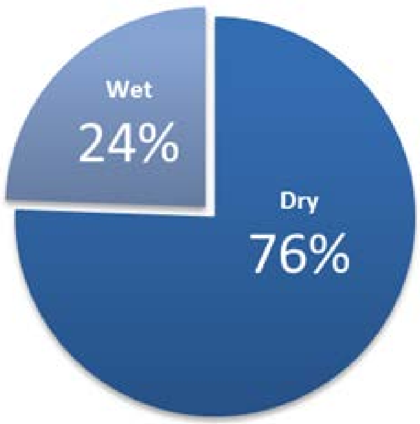
Figure 7.10 Proportion of daytime and night-time crashes, by city



7.5 Wet/dry crashes

Figure 7.11 shows that wet-road crashes accounted for 24% of crashes at the selected signalised intersections in New Zealand.

Figure 7.11 Proportion of dry- and wet-road crashes at New Zealand sites



8 Safety effects of predictor variables

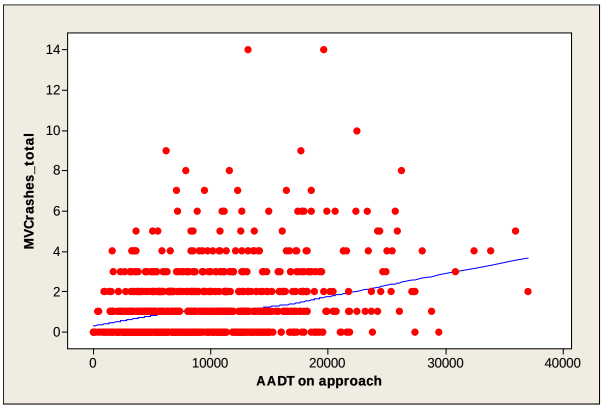
This section presents an analysis of the effect of key physical and operational characteristics of traffic signals on safety, based on data from the intersections in the sample set for this study. The red dots represent the crashes observed on each intersection approach, and the line represents the predicted crashes.

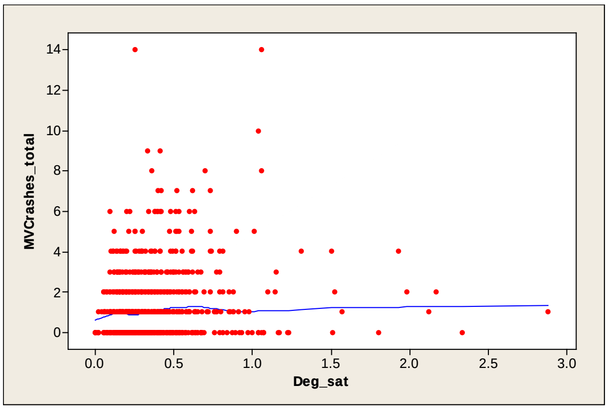
8.1 Motor vehicle crashes

8.1.1 Traffic volume

Figure 8.1 shows that sites with a higher traffic volume generally have more crashes. The scatter plot for degree of saturation shows that intersections that are between 50% and 75% saturated have higher crash rates than the rest, although crash numbers reduce slightly as the intersection approaches saturation. The number of crashes is observed to increase slightly as the intersection becomes significantly over-saturated.

Figure 8.1 Crashes vs traffic flow

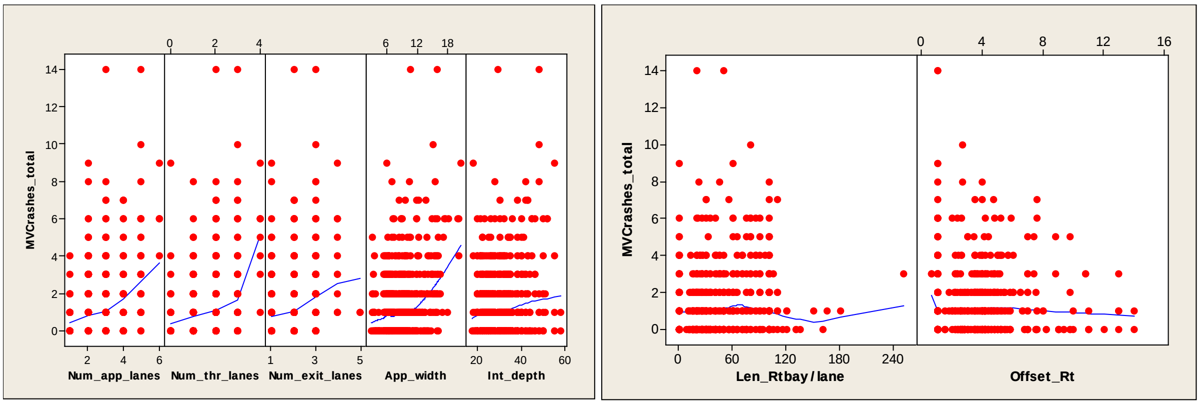




8.1.2 Intersection geometry

The scatter plots in figure 8.2 show that larger intersections, ie those with more approaching and exit lanes and greater intersection depths, have more crashes. Intersections with right-turn bays or exclusive right-turning lanes that are longer than 60m are associated with lower crash numbers than those sites with shorter right-turning bays/lanes.

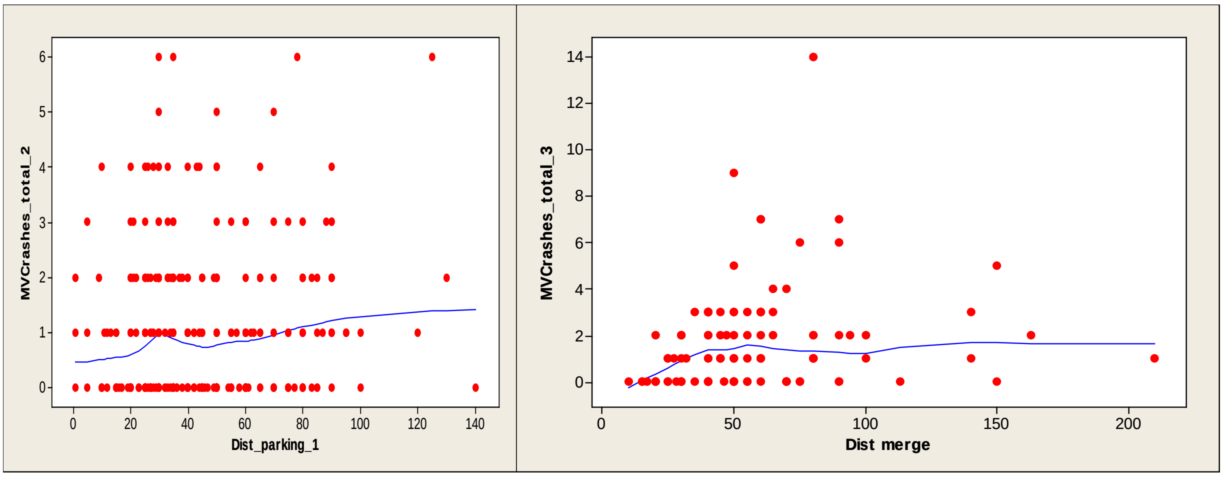
Figure 8.2 Crashes vs intersection geometry variables



8.1.3 Presence of parking and exit merges

Figure 8.3 shows that intersections that have parking within 30-40m of the limit line are often associated with a high number of crashes. The plot for distance to exit merge does not display a clear relationship with crashes.

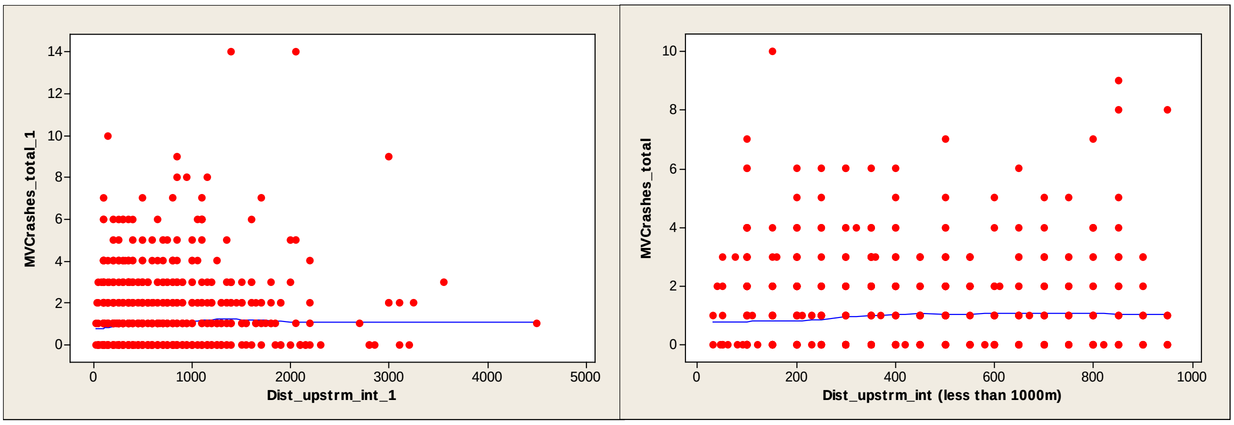
Figure 8.3 Crashes vs distance to parking, distance to merge



8.1.4 Mid-block lengths

Distance from upstream intersection did not seem to have a clear relationship with crashes. Figure 8.4 displays scatter plots for all approaches, and for approaches with an upstream intersection within 1000m. Neither graph displays a clear effect of distance between intersections on crashes.

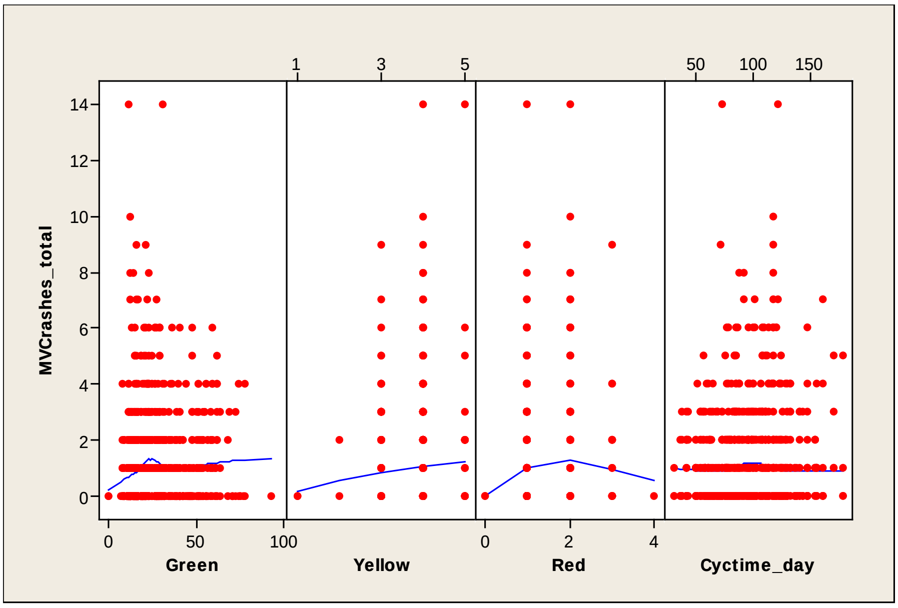
Figure 8.4 Crashes vs distance from upstream intersection



8.1.5 Signal timing

Figure 8.5 indicates that intersections in the sample set that had cycle times of 90-100sec were associated with more crashes. Intersections with longer yellow times were also observed to have more crashes; however, this was probably due to the fact that larger intersections usually tend to have longer yellow times. The scatter plot for all-red time indicates that sites where the all-red time had been extended beyond the more commonly used two seconds were associated with fewer crashes.

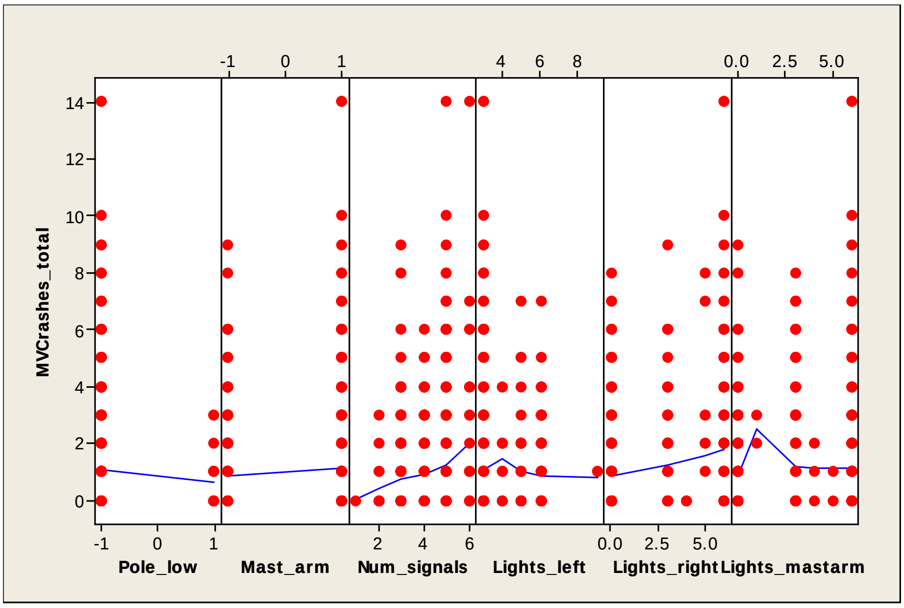
Figure 8.5 Crashes vs signal timing variables



8.1.6 Signal displays and aspects

The scatter plots shown in figure 8.6 indicate that sites that had more crashes also had more signal displays, and usually had a mast arm as well. This again was likely to be a consequence of larger intersections (which generally have more crashes) being provided with multiple signal aspects and mast arms.

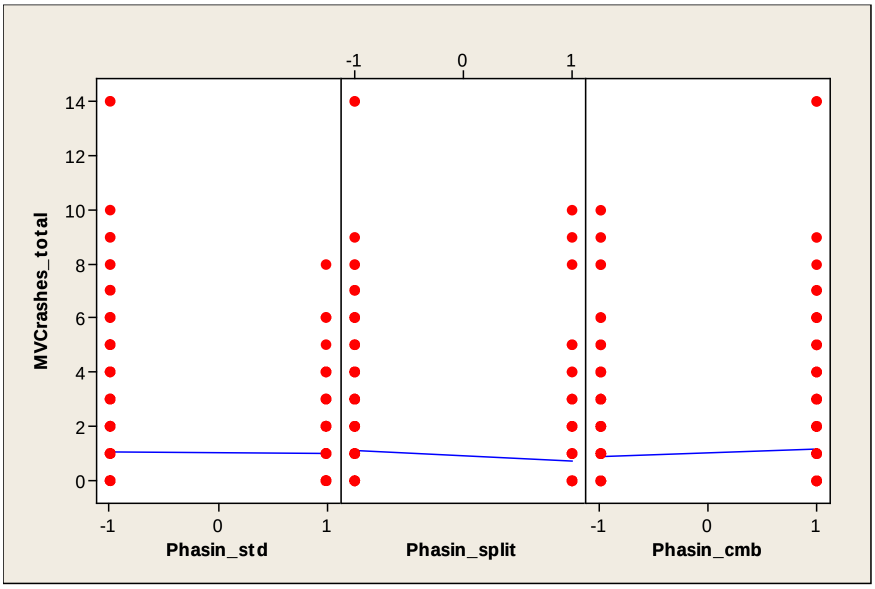
Figure 8.6 Crashes vs signal display and aspect variables



8.1.7 Signal phasing

Figure 8.7 does not display sufficient variation in the number of all crashes with phasing sequence to merit any conclusions.

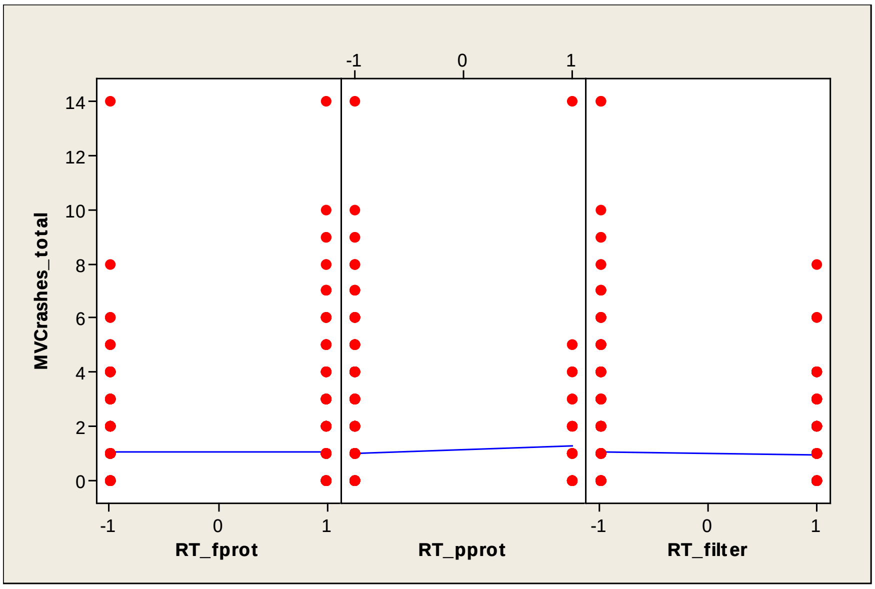
Figure 8.7 Crashes vs signal phasing variables



8.1.8 Right-turn phasing

The scatter plots shown in figure 8.8 do not show any definite relationship between the type of phasing sequence for right-turning vehicles and crashes.

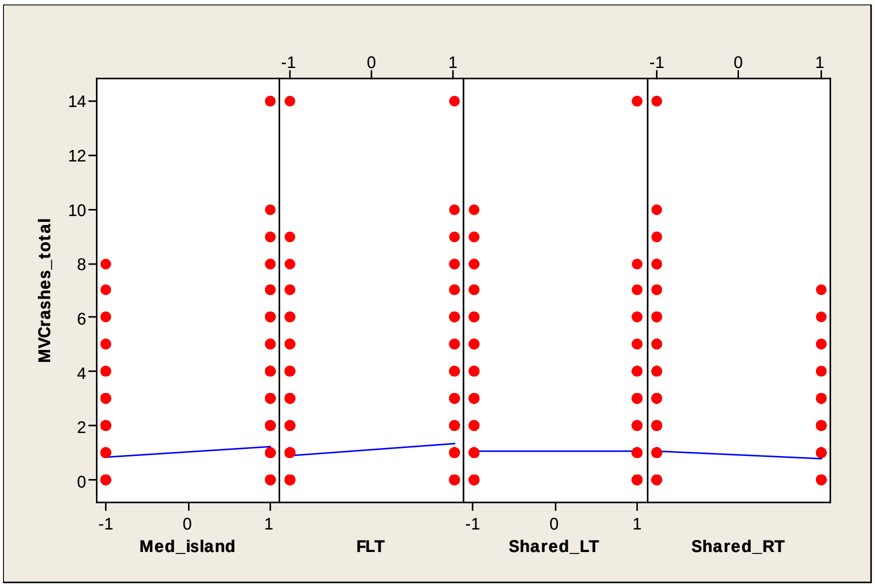
Figure 8.8 Crashes vs right-turn phasing variables



8.1.9 Layout

Figure 8.9 shows that sites that had a median/central island or free left turn for vehicles had slightly more crashes than those that did not. No clear relationship is seen in the plot for shared left-turning lanes. The plot for shared right-turning lanes indicates that approaches where shared right-turn/through lanes are provided appear to have slightly fewer crashes, which is expected since they are usually present at smaller intersections with lower traffic volumes.

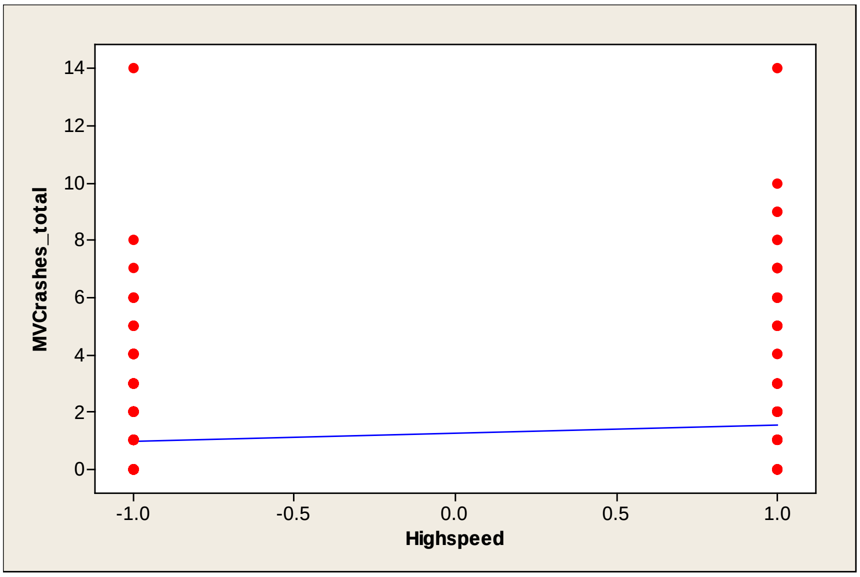
Figure 8.9 Crashes vs signal layout variables



8.1.10 Speed environment

The scatter plot between speed environment and crashes shown in figure 8.10 suggests that more crashes occur at approaches with speed limits of 80kph or more.

Figure 8.10 Crashes vs speed environment

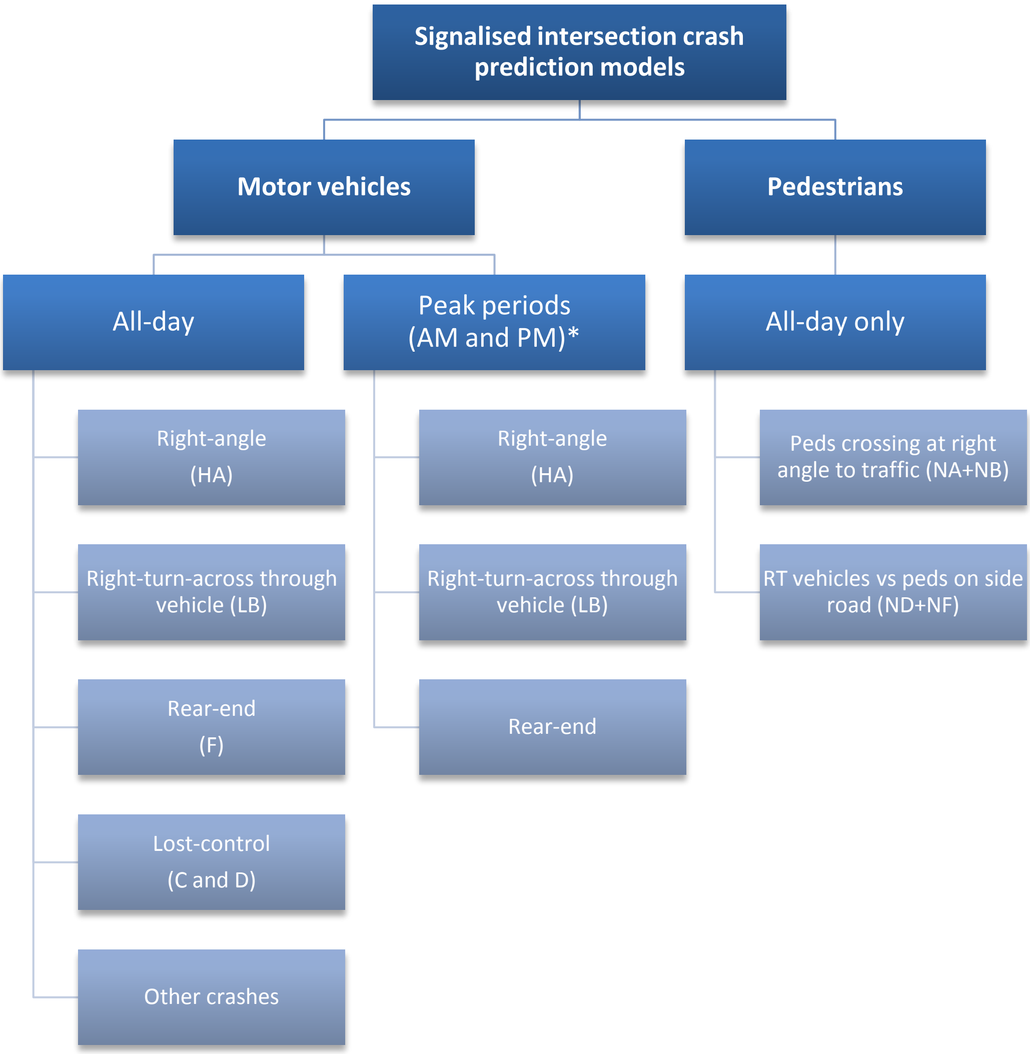


9 Crash prediction models

9.1 Models developed

Figure 9.1 presents the various categories of crash prediction models that were developed as part of this study. Models were developed for the main crash types to understand how changes in signal geometry and phasing affect each crash type. This also enables an assessment of how certain interventions may improve the safety of a particular crash type while negatively affecting that of other crash types.

Figure 9.1 Crash prediction models developed[a]



a) Two-hour weekday peak periods were used for developing the peak-period models. The time periods used were AM (7:00–9:00) and PM (16:00–18:00).

The data collected during this study did not prove to be sufficient to build well-fitting models for cycle-vehicle crashes. As a result, no cycle-vehicle crash models have been reported on.

9.2 Crash-modelling methodology

Crash prediction models are mathematical models that relate crashes to road user volumes and other road layout and operational features. They are cross-sectional regression models. With crashes being discrete events, and typically following a Poisson or negative binomial distribution, traditional regression analysis methods such as linear regression are not suitable. The models used in crash prediction are developed using generalised linear modelling methods. Generalised linear models were first introduced to road crash studies by Maycock and Hall (1984), and extensively developed in Hauer et al (1989). These models were further developed and fitted by Turner (1989), using crash data and traffic counts in the New Zealand context for motor-vehicle-only crashes. Cross-section models do have some limitations, which need to be considered when interpreting the result of the models. Refer to Road Safety Research.com for more details on cross-section modelling (www.oocities.org/haver@rogers.com/download.htm).

The aim of this modelling exercise is to develop relationships between the mean number of crashes (as the response variable), and traffic flows, as well as non-flow predictor variables. Typically the models take the multiplicative form:

$$A = b_0 x_1^{b_1} \ldots x_i^{bi} e^{b_{i+1}x_{i+1}} \ldots e^{b_n x_n} \qquad \text{(Equation 9.1)}$$

Where

A is the fitted annual mean number of crashes

the x_1 to x_i are measurement variables, such as average daily flows of vehicles

the x_i+$_1$ to x_n are categorical variables, recording the presence, for example, of a cycle installation

the $b_1,\ldots,b_n$ are the model coefficients.

9.2.1 Model development process

Once a functional model form has been selected, in this case the power model, generalised linear models are then developed for each crash type, using either a negative binomial or Poisson distribution error structure.

Software has been developed in Minitab in order to fit such models (ie to estimate the model coefficients). For this study, the popular Bayesian Information Criterion (BIC) was used as the preferred criterion to decide when the addition of a new variable was worthwhile. Goodness-of-fit testing of all models was also undertaken, using software that has been written in the form of Minitab macros. A detailed description of the modelling methodology adopted is given in appendix E of this report.

9.2.2 Model interpretation

Once models have been developed, in some simple cases the relationship between crashes and predictor variables can be interpreted. Caution should always be exercised when interpreting relationships, as two or more variables can be highly correlated. However, the modelling process described in the previous sections usually means that variables in the ‘preferred’ models are not highly correlated because the method acknowledges that adding a variable correlated to those already in an existing model does not improve the fit of the model, compared with the addition of important non-correlated variables. Likewise, functional forms that deviate from a power function are also difficult to interpret. In these situations it is always best to plot the relationship.

In models with a power-function form, where the variables are not correlated, an assessment of the relationship can be carried out. For a typical model with a power-function form and two continuous variables (such as flows or speeds) the equation takes the following form:

$$A = b_0 x_1^{b_1} x_2^{b_2}$$ (Equation 9.2)

Where:

A is the annual mean number of crashes

x_1, x_2 are continuous flow or non-flow variables

b_0, b^1 and b^2 are model parameters.

In this model form, the parameter b_0 acts as a constant multiplicative value. Different b_0 coefficients were produced in this study, one for Melbourne and the other for Christchurch, which allowed the relative safety performance of each city to be compared. If the number of reported injury crashes is not dependent on the values of the two-predictor variables (x_1 and x_2), then the model parameters b^1 and b^2 are zero. In this situation the value of b_0 was equal to the mean number of crashes. The value of the parameters b^1 and b^2 indicate the relationship that a particular predictor variable has (over its flow range) with crash occurrence. There are five types of relationship for this model form, as presented in figure 9.2 and discussed in table 9.1.

Figure 9.2 Relationship between crashes and predictor variable x for different model exponents (b^1)

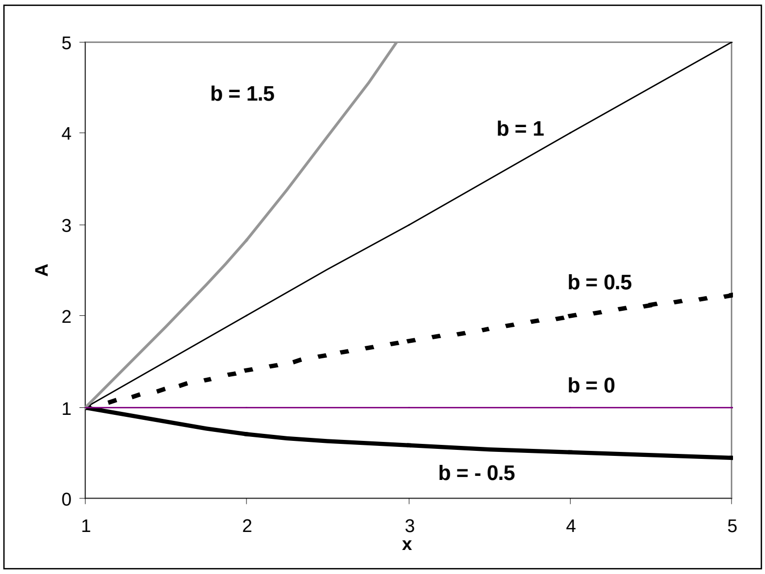


Table 9.1 Relationship between predictor variable and crash rate

Value of exponent	Relationship with crash rate
bi > 1	For increasing values of the variable, the number of crashes will increase at an increasing rate.
bi = 1	For increasing values of the variable, the number of crashes will increase at a constant (or linear) rate.
0 < bi < 1	For increasing values of the variable, the number of crashes will increase at a decreasing rate.
bi = 0	There will be no change in the number of crashes with increasing values of the variable.
bi < 0	For increasing values of the variable, the number of crashes will decrease.

Generally, models of this form have exponents between bi = 0 and bi = 1, with most flow variables having an exponent close to 0.5; ie the square root of flow. In some situations, however, parameters have a value outside this range.

In the case of models including a covariate (here, discrete variables with a small number of alternatives) a multiplier for different values of the variable is produced, and it is easy to interpret the relationship. This factor indicates how much higher (or lower) the number of crashes is if the feature (such as a cycle storage facility) is present. A factor of 1 indicates no effect on crash occurrence.

10 Motor vehicle crash models

The following section presents the models that were developed for each of the major crash types involving motor vehicles.

10.1 Right-angle crashes (NZ type HA)

Three models were developed for right-angle (HA) crashes. The first model takes into account all selected intersections in the six cities. Although the variation in crash rates and factors between cities resulted in this model displaying a poor goodness of fit, the model still provides a fair indication of the factors affecting safety at signalised intersections.

Due to the similarities observed between Auckland and Melbourne within this crash group, a separate model was developed specifically for these two cities. This model displays a high goodness of fit. This model also supports the conclusions of the all-cities model, and provides a more accurate prediction for Auckland and Melbourne intersections.

Finally, a combined morning and evening peak-period model representing sites from all six cities was also built.

10.1.1 All-cities model

Equation 10.1 presents the preferred model form.

$$A_{HA} = B_0 \text{ X } q_2^{0.311} \text{ X } (q_5+q_{11})^{0.362} \text{ X } \exp(0.356 \text{ X Number of approaching lanes}) \text{ X } (\text{Intersection depth})^{0.602} \text{ X } (\text{Cycle time})^{0.037} \text{ X } (\text{All-red time})^{-0.636} \text{ X } F_{\text{Split phasing}} \text{ X } F_{\text{Mast arm}} \text{ X } F_{\text{Coordinated}} \text{ X } F_{\text{Adv detector}} \text{ X } F_{\text{Shared turns}} \text{ X } F_{\text{Med island}} \quad \text{(Equation 10.1)}$$

Table 10.1 Model variables

Factor	Value	Description
$B_{0\ (\text{Auckland})}$	4.27E-05	Constant for Auckland
$B_{0\ (\text{Wellington})}$	2.08E-05	Constant for Wellington
$B_{0\ (\text{Christchurch})}$	8.69E-05	Constant for Christchurch
$B_{0\ (\text{Hamilton})}$	1.13E-04	Constant for Hamilton
$B_{0\ (\text{Dunedin})}$	1.54E-04	Constant for Dunedin
$B_{0\ (\text{Melbourne})}$	4.11E-05	Constant for Melbourne
$F_{\text{Split phasing}}$	0.69	Split phasing on approach
$F_{\text{Mast arm}}$	0.74	Presence of signal mast arm
$F_{\text{Coordinated}}$	1.31	Signal coordination with upstream intersection
$F_{\text{Adv detector}}$	2.06	Presence of advanced detector on approach
$F_{\text{Shared turns}}$	1.19	Lanes with shared movements (eg left-turn/through or right-turn/through) present on approach
$F_{\text{Med island}}$	0.67	Presence of raised median/central island on approach

Where:

A_{HA} = number of predicted HA injury crashes in 5 years

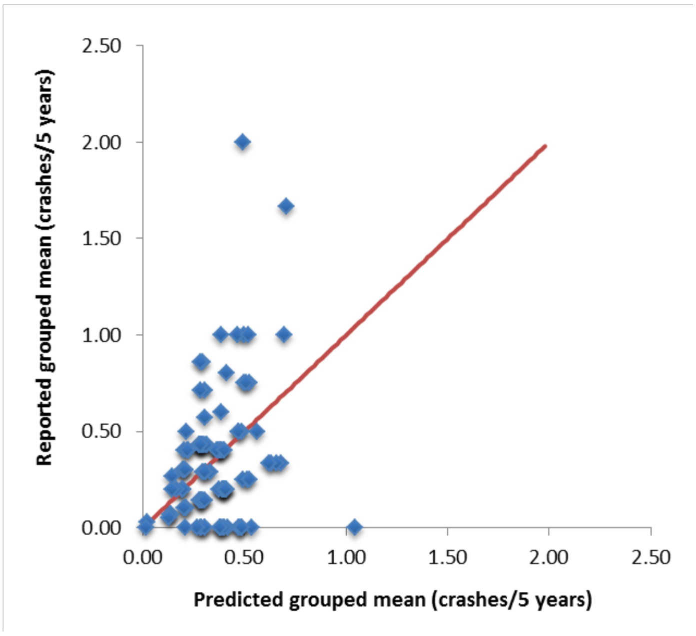
q_2 = daily volume of through vehicles on movement 2 (see approach movement coding) – refer to appendix B

q_5 = daily volume of through traffic coming from left side (movement 5 – see approach movement coding)

q_{11} = daily volume of through traffic coming from right side (movement 11 – see approach movement coding).

The error structure for this model was found to be negative binomial. Figure 10.1 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 10.1 has a p-value of 0.021, which indicates that the model did not have a very high goodness of fit.

Figure 10.1 Predicted vs reported grouped means of crashes



The magnitudes of the constant term (B_0) for the different cities in this model points to a significant variation in the number of type HA crashes between cities. This is also likely to be the primary cause of the large variation seen in model results and the resulting low goodness of fit.

However, the model does indicate the factors that appear to have had a significant effect on safety. Both intersection traffic flow volumes had similar coefficients. Larger intersections, ie those with more approach lanes and larger intersection depths, also had more type HA crashes. Split phasing and the presence of a mast arm or raised median/central island on the approach reduced the number of type HA crashes, while approaches having shared turns and traffic signals located along a coordinated route generally tended to have more type HA crashes.

Surprisingly, approaches with an advanced detector appeared to have twice the number of type HA crashes than those where these detectors were not present. This result is counterintuitive and further research, such as before-and-after studies, is required to determine the safety consequences. Given this result, this factor has been excluded from the Excel toolkit that has been prepared to assess the safety of traffic signals.

10.1.2 Auckland and Melbourne model

The B_0 values for Auckland and Melbourne in equation 10.1 were similar. Therefore a separate model for the Auckland and Melbourne sites was developed to limit some of the variation that was apparent in the first model.

Equation 10.2 presents the preferred model form.

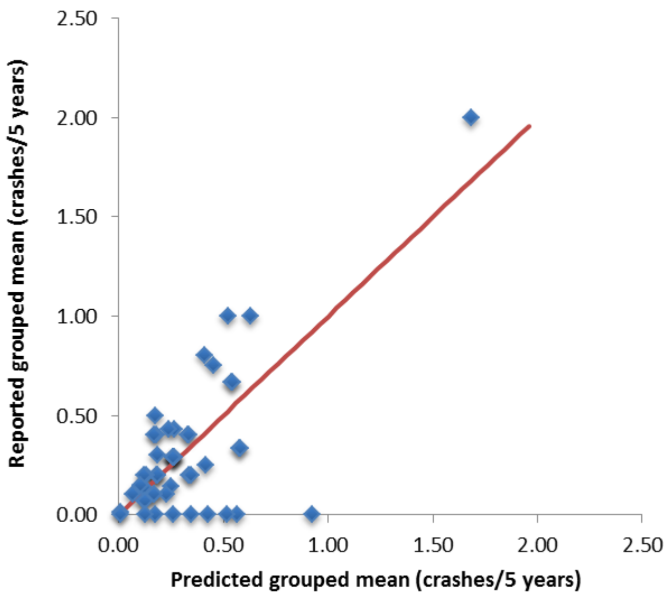
$$A_{HA\ (AKL,MEL)} = B_0 \times q_2^{0.455} \times (q_5+q_{11})^{0.47} \times \exp(0.397 \times \text{Number of approaching lanes}) \times (\text{Intersection depth})^{0.494} \times (\text{Cycle time})^{-0.286} \times (\text{All-red time})^{-1.321} \times F_{Split\ phasing} \times F_{Mast\ arm} \times F_{Coordinated} \times F_{Adv\ detector} \times F_{Shared\ turns} \times F_{Med\ island} \quad \text{(Equation 10.2)}$$

Table 10.2 Model variables

Factor	Value
B_0	2.18E-05
$F_{Split\ phasing}$	0.93
$F_{Mast\ arm}$	0.77
$F_{Coordinated}$	0.85
$F_{Adv\ detector}$	2.20
$F_{Shared\ turns}$	0.68
$F_{Med\ island}$	0.57

This model had a Poisson error structure. Equation 10.2 has a p-value of 0.19, which indicates that the model had a high goodness of fit. Figure 10.2 plots the predicted and reported grouped means of crashes.

Figure 10.2 Predicted vs reported grouped means of crashes



As expected, limiting the amount of spatial variation results in a more accurate model. The above model again indicates that both conflicting flows equally contributed to type HA crashes. Larger intersections had more crashes, although reduction in cycle time and all-red time had a greater positive effect on safety. The presence of split phasing, mast arms and raised medians reduced type HA crashes, although the magnitude of reduction for split phasing was lower than that predicted by the first model. The presence of an advanced detector again had a large negative effect on safety. However, in contrast to the model for all-cities, the presence of shared lanes and signal coordination resulted in a decrease in crashes for Auckland and Melbourne.

10.1.3 Peak-period model

Equation 10.3 presents the preferred model form for the peak-period model.

$$A_{HA\ (Peak)} = B_0 \text{ X } q_2^{0.156} \text{ X } (q_5+q_{11})^{0.381} \text{ X } \exp(0.788 \text{ X Number of approaching lanes}) \text{ X } (\text{Intersection depth})^{1.237} \text{ X } (\text{Cycle time})^{-0.945} \text{ X } (\text{All-red time})^{-2.528} \text{ X } F_{Split\ phasing} \text{ X } F_{Mast\ arm} \text{ X } F_{Coordinated} \text{ X } F_{Shared\ turns} \text{ X } F_{Med\ island} \quad \text{(Equation 10.3)}$$

Table 10.3 Model variables

Factor	Value	Description
B_0	6.61E-05	Constant
$F_{Split\ phasing}$	0.35	Split phasing on approach
$F_{Mast\ arm}$	0.56	Presence of signal mast arm
$F_{Coordinated}$	1.49	Signal coordination with upstream intersection
$F_{Shared\ turns}$	2.06	Lanes with shared movements (eg left-turn/through or right-turn/through) present on approach
$F_{Med\ island}$	1.19	Presence of raised median/central island on approach

Interestingly, the conflicting traffic flow from the left and right side of the main vehicle was significantly more important in the morning and evening peak periods than during the whole day. The effect of larger intersection size (more crashes), split phasing (fewer crashes) and shared turns (more crashes) was also more significant in the peaks. Presence of advanced detectors is not seen to have an effect in this model.

10.1.4 Summary: type HA crash models

Table 10.4 Summary of type HA crash models

Factor	Effect on type HA crashes		
	All cities, whole day	Auckland & Melbourne, whole day	Peak periods
Higher traffic volume	Increase	Increase	Increase
Larger intersections (approach lanes, intersection depth)	Increase	Increase	Increase
Longer cycle time	Decrease	Decrease	Decrease
Longer all-red time	Decrease	Decrease	Decrease
Split phasing	Decrease	Decrease	Decrease
Mast arm	Decrease	Decrease	Decrease
Coordinated signals	Increase	Decrease	Increase
Advanced detector	Increase	Increase	No effect
Shared turns (left-turn/through, right-turn/through or both)	Increase	Decrease	Increase
Raised median/central island	Decrease	Decrease	Increase

10.2 Right-turn-against crashes (NZ type LB)

Two models were developed for right-turn-against (LB) crashes. The first model considered sites from all six cities. The variation seen in type LB crashes between different cities was not as large as that found in other crash types, and the need for a separate model for Auckland and Melbourne intersections was not required.

A second model for predicting type LB crashes occurring in morning and evening peak periods was also developed.

10.2.1 All-day model

Equation 10.4 presents the preferred model form.

$$A_{LB} = B_0 \text{ X } q_7^{0.155} \text{ X } (1+\text{Length of RT bay or lane})^{-0.124} \text{ X } \exp(0.352 \text{ X Number of through lanes}) \text{ X } (\text{Degree of saturation})^{0.397} \text{ X } (\text{Cycle time})^{-0.683} \text{ X } F_{\text{Full RT Protection}} \text{ X } F_{\text{Shared RT}} \text{ X } F_{\text{Med island}} \text{ X } F_{\text{Cycle facilities}} \quad \text{(Equation 10.4)}$$

Table 10.5 Model variables

Factor	Value	Description
$B_{0\ (Auckland)}$	3.83	Constant for Auckland
$B_{0\ (Wellington)}$	4.10	Constant for Wellington
$B_{0\ (Christchurch)}$	4.41	Constant for Christchurch
$B_{0\ (Hamilton)}$	2.27	Constant for Hamilton
$B_{0\ (Dunedin)}$	4.16	Constant for Dunedin
$B_{0\ (Melbourne)}$	3.95	Constant for Melbourne
$F_{\text{Full RT Protection}}$	0.71	Fully protected right-turn phasing
$F_{\text{Shared RT}}$	0.72	Shared right-turn/through lane present on approach
$F_{\text{Med island}}$	1.22	Presence of raised median/central island on approach
$F_{\text{Cycle facilities}}$	1.35	Presence of cycle facilities (cycle lanes or storage) on approach

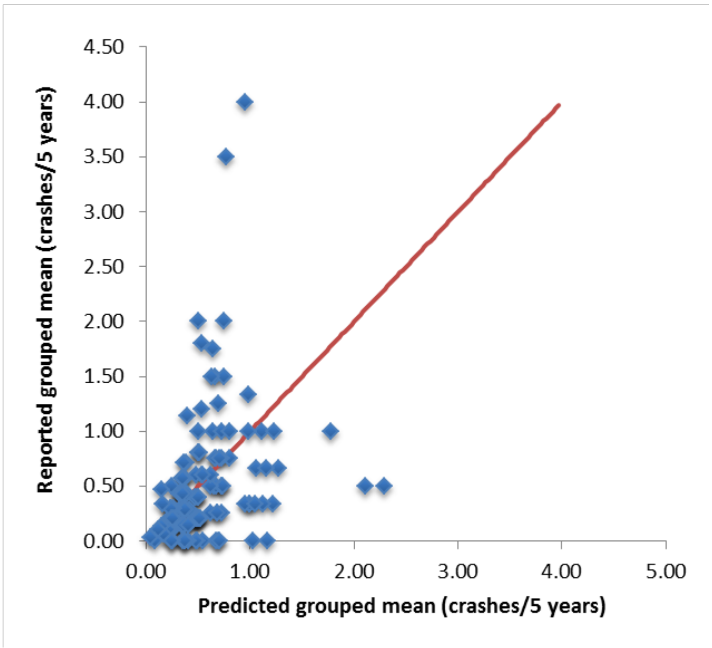
Where:

A_{LB} = number of predicted LB injury crashes in 5 years

q_7 = daily volume of right-turning vehicles on movement 7 (see approach movement coding).

The error structure for this model was found to be negative binomial. Figure 10.3 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 10.4 has a p-value of 0.041, which indicates that the model was just below the standard required for a statistically significant model.

Figure 10.3 Predicted vs reported grouped means of crashes



The model for type LB crashes suggests that the right-turning traffic volume was a more significant contributor in these crashes than the through-traffic volume. Wider approaches (ie those with more lanes for through traffic) were more prone to these crashes, although extending the length of right-turning bays or lanes resulted in fewer crashes. Degree of saturation was also observed to have a significant negative effect on safety for this crash type. As was the case with type HA crashes, longer cycle times also resulted in a reduction in type LB crashes. Fully protected right-turn phasing, and shared right-turn/through lanes, improved safety, while the presence of a raised median and cycle facilities resulted in higher crash rates.

10.2.2 Peak-period model

Equation 10.5 presents the preferred model form for the morning and evening peak periods.

$$A_{LB\ (Peak)} = B_0 \text{ X } q_7^{0.256} \text{ X } (1+\text{Length of RT bay or lane})^{0.111} \text{ X } \exp(0.26 \text{ X Number of through lanes}) \text{ X } (\text{Degree of saturation})^{0.41} \text{ X } (\text{Cycle time})^{-0.034} \text{ X } F_{Full\ RT\ Protection} \text{ X } F_{Shared\ RT} \text{ X } F_{Med\ island} \quad \text{(Equation 10.5)}$$

Table 10.6 Model variables

Factor	Value	Description
B_0	5.82E-03	Constant
$F_{Full\ RT\ Protection}$	0.24	Fully protected right-turn phasing
$F_{Shared\ RT}$	0.56	Shared right-turn/through lane present on approach
$F_{Med\ island}$	1.84	Presence of raised median/central island on approach

The right-turning traffic volume is observed to have had a greater increasing effect on crashes in the peaks than in the all-day period. Interestingly, longer right-turning bays/lanes resulted in a slight increase in crashes. Longer cycle times still reduced crashes, although the effect was quite diminished. The effect of full right-turn protection (fewer crashes), shared right-turn/through lanes (fewer crashes) and the presence of a raised median or central island (more crashes) was more pronounced than during the all-day period.

10.2.3 Summary: type LB crash models

Table 10.7 summarises the safety impacts of various factors identified in the type LB crash models.

Table 10.7 Summary of type LB crash models

Factor	Effect on type LB crashes	
	All cities, whole day	Peak periods
Higher right-turning traffic volume	Increase	Increase
More through lanes	Increase	Increase
Longer cycle time	Decrease	Decrease
Longer right-turn bay/lane	Decrease	Increase
Higher degree of saturation	Increase	Increase
Full right-turn protection	Decrease	Decrease
Shared right-turn/through lane	Decrease	Decrease
Raised median/central island	Increase	Increase
Cycle facilities	Increase	No effect

10.3 Rear-end crashes (NZ type F)

Models that utilised data from all selected intersections were initially developed for rear-end (F) crashes. However, a large degree of variation due to intersection size was observed in these model results. It was thus felt necessary to develop models based on the size of the signalised intersection.

Intersections were split into three size categories and crash prediction models were built for each. These categories were:

- small intersections – those having 1 or 2 approach lanes and an intersection depth of 25m or less
- large intersections – those having 3 or more approach lanes and an intersection depth of 40m or greater
- medium intersections – those not lying in either of the two above categories.

Table 10.8 shows the number of approaches that fell within each size category, along with the total number of type F crashes. Models for the three intersection sizes are presented in sections 10.3.1–10.3.3.

Table 10.8 Number of approaches and crashes, by intersection size classification

Intersection size	Number of approaches	Number of crashes
Small	201	36
Medium	611	184
Large	77	93

10.3.1 Rear-end models for medium-sized intersections

These intersections formed the majority of the sample set. Equation 10.6 presents the preferred model form for type F crashes at medium-sized intersections.

$$A_{\text{Rear end (medium)}} = B_0 \text{ X } q^{0.496} \text{ X } \exp(0.243 \text{ X Number of approaching lanes}) \text{ X } (\text{Lost time})^{0.209} \text{ X } F_{\text{Cycle facilities}} \text{ X } F_{\text{Standard phasing}} \text{ X } F_{\text{FLT}} \text{ X } F_{\text{High speed}} \text{ X } F_{\text{Approach bus bay}} \text{ X } F_{\text{Commercial}} \quad \text{(Equation 10.6)}$$

Table 10.9 Model variables

Factor	Value	Description
$B_{0\ (\text{Auckland})}$	9.56E-04	Constant for Auckland
$B_{0\ (\text{Wellington})}$	1.41E-03	Constant for Wellington
$B_{0\ (\text{Christchurch})}$	1.12E-03	Constant for Christchurch
$B_{0\ (\text{Hamilton})}$	9.29E-04	Constant for Hamilton
$B_{0\ (\text{Dunedin})}$	3.06E-03	Constant for Dunedin
$B_{0\ (\text{Melbourne})}$	1.16E-03	Constant for Melbourne
$F_{\text{Cycle facilities}}$	0.753	Presence of cycle facilities (cycle lanes or storage) on approach
$F_{\text{Standard phasing}}$	0.637	Standard phasing on approach
F_{FLT}	1.442	Presence of free-left-turn lane for motor vehicles
$F_{\text{High speed}}$	1.449	Speed limit of 80kph or more on approach
$F_{\text{Approach bus bay}}$	0.908	Presence of upstream bus bay within 100m of limit line
$F_{\text{Commercial}}$	0.900	Commercial land use

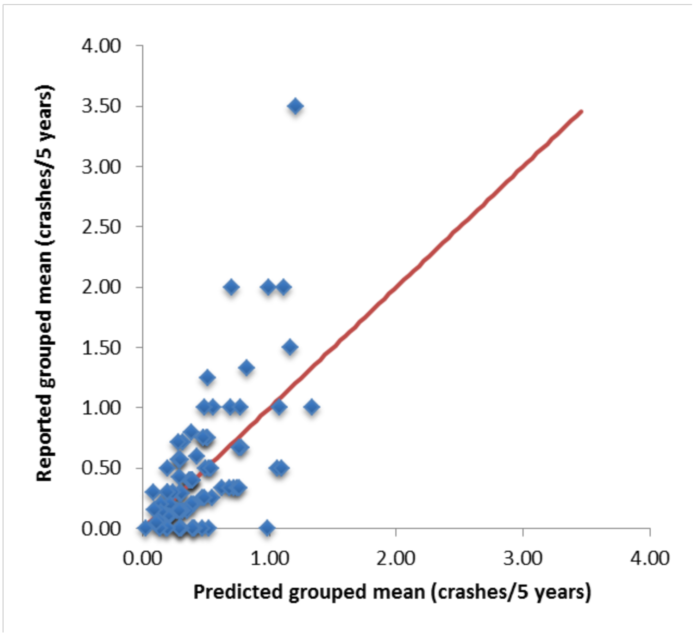
Where:

$A_{\text{Rear end (medium)}}$ = number of predicted rear-end injury crashes at medium-sized intersections (see definition above) in 5 years

q = total AADT entering the intersection from the approach.

The error structure for this model was found to be negative binomial. Figure 10.4 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 10.6 has a p-value of 0.047, which indicates that the model was close to the standard level required for a statistically significant model.

Figure 10.4 Predicted vs reported grouped means of crashes



Type F crashes showed a strong relationship with the total approach traffic volume (AADT). Intersections with more approach lanes were also seen to increase crash numbers. Although lost time had a positive coefficient, this was likely to be the result of variation within the sample set (the non-city covariate model showed a reduction in crashes with longer lost times).

The model results also indicate that intersections that operated using a standard phasing sequence, and approaches with cycle facilities, had fewer type F crashes. A high-speed environment and presence of a free left turn for motor vehicles negatively affected safety. The presence of an approach bus bay within 100m upstream of the approach limit line, and the commercial land-use environment, also led to slight reductions in type F crashes, possibly due to drivers observing more caution while driving in such environments.

10.3.2 Small intersections

Only 36 out of the 313 type F crashes in the sample set occurred at small intersections. Equation 10.7 presents the preferred model form.

$$A_{\text{Rear end (small)}} = B_0 \text{ X } q^{0.447} \text{ X } (1+\text{Length of right turn bay or lane})^{-0.259} \text{ X } (\text{Lost time})^{-3.424} \text{ X } F_{\text{Split Phasing}} \text{ X } F_{\text{Approach bus bay}} \text{ X } F_{\text{Cycle facilities}} \text{ X } F_{\text{FLT}} \quad \text{(Equation 10.7)}$$

Table 10.10 Model variables

Factor	Value	Description
$B_{0\ (Auckland)}$	1.38E+00	Constant for Auckland
$B_{0\ (Wellington)}$	6.58E-01	Constant for Wellington
$B_{0\ (Christchurch)}$	4.34E+00	Constant for Christchurch
$B_{0\ (Hamilton)}$	1.36E+00	Constant for Hamilton
$B_{0\ (Dunedin)}$	7.95E+00	Constant for Dunedin
$B_{0\ (Melbourne)}$	1.25E+00	Constant for Melbourne
$F_{Split\ Phasing}$	5.256	Split phasing on approach
$F_{Approach\ bus\ bay}$	1.309	Presence of upstream bus bay within 100m of limit line
$F_{Cycle\ facilities}$	0.706	Presence of cycle facilities (cycle lanes or storage) on approach
F_{FLT}	1.585	Presence of free-left-turn lane for motor vehicles

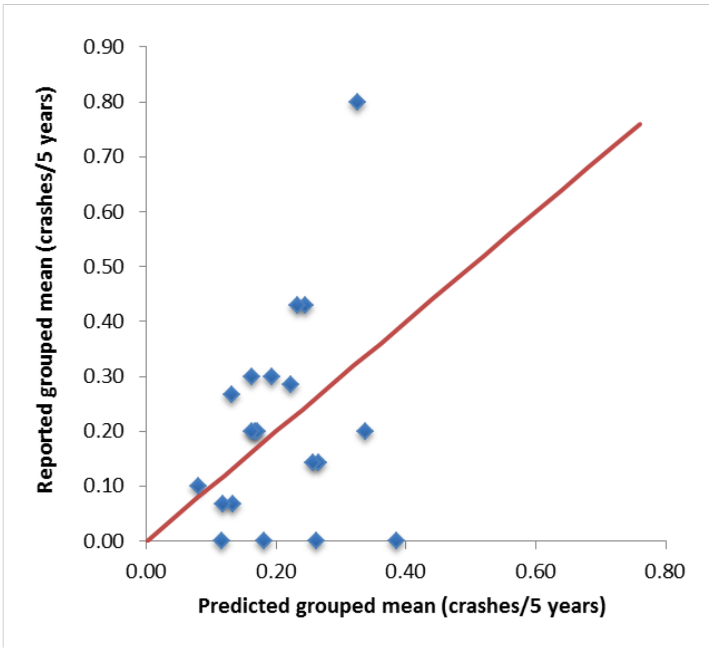
Where:

$A_{Rear\ end\ (small)}$ = number of predicted type F injury crashes at small intersections (see definition above) in 5 years

q = total daily traffic volume entering the intersection from the approach.

The error structure for this model was found to be Poisson. Figure 10.5 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 10.7 has a p-value of 0.022, which indicates that the model did not have a high goodness of fit. This is probably a result of the small sample size for this model.

Figure 10.5 Predicted vs reported grouped means of crashes



Traffic flow at small intersections shows a similar coefficient to that at medium-sized intersections. The number of approach lanes did not figure in this model, as most sites only had either one or two approach lanes, although longer right-turning bays positively affected safety. Longer lost times and the presence of cycle facilities reduced crashes, while the presence of upstream bus bays within 100m, and free left turns, increased crashes. The operation of intersection approaches using the split phasing sequence increased the number of type F crashes at small intersections.

10.3.3 Large intersections

The sample set for this model consisted of less than 9% of approaches; however, almost 30% of all type F crashes occurred at these approaches. The selected large intersections were located only in Melbourne (mostly high-speed intersections), Christchurch (low-speed intersections) and Auckland (both high-speed and low-speed intersections).

Equation 10.8 presents the preferred model form for type F crashes at large intersections.

$$A_{\text{Rear end (large)}} = B_0 \times q^{0.356} \times \exp(0.459 \times \text{Number of approaching lanes}) \times (1+\text{Length of right turn bay or lane})^{-1.142} \times (\text{Lost time})^{-1.739} \times F_{\text{High speed}} \times F_{\text{Standard phasing}} \times F_{\text{Cycle facilities}} \times F_{\text{FLT}} \times F_{\text{Commercial}} \qquad \text{(Equation 10.8)}$$

Table 10.11 Model variables

Factor	Value	Description
$B_{0\ (\text{Auckland})}$	3.92E+00	Constant for Auckland
$B_{0\ (\text{Christchurch})}$	7.74E-01	Constant for Christchurch
$B_{0\ (\text{Melbourne})}$	3.36E+00	Constant for Melbourne
$F_{\text{High speed}}$	0.985	Speed limit of 80kph or more on approach
$F_{\text{Standard phasing}}$	1.053	Standard phasing on approach
$F_{\text{Cycle facilities}}$	1.257	Presence of cycle facilities (cycle lanes or storage) on approach
F_{FLT}	1.227	Presence of free-left-turn lane for motor vehicles
$F_{\text{Commercial}}$	0.819	Commercial land use

Where:

$A_{\text{Rear end (large)}}$ = number of predicted rear-end injury crashes at large intersections (see definition above) in 5 years

q = total AADT entering the intersection from the approach.

The error structure for this model was found to be Poisson. No goodness-of-fit testing was undertaken for this model due to the limited sample set. However, the model results do indicate the increased contribution of intersection size in increasing the number of rear-end crashes at large intersections. The negative coefficient for lost time indicates that safety could be improved by extending the total yellow and all-red time at these intersections. In contrast to small and medium-sized intersections, the presence of cycle facilities was shown to cause more rear-end F crashes at large intersections. The presence of a free left turn also increased crashes, while intersections located in predominantly commercial areas had fewer crashes.

10.3.4 Peak-period models

Table 10.12 presents the combined morning and evening peak-period models that were developed for each intersection size.

Table 10.12 Peak period models by intersection size

Intersection size	Model	Model factors
Small	$A_{Rear\ end\ (small,\ peaks)} = 4.30E\text{-}03 \times q^{0.252} \times F_{Split\ phasing}$	$F_{Split\ phasing} = 2.33$
Medium	$A_{Rear\ end\ (medium,\ peaks)} = 7.89E\text{-}04 \times q^{0.457} \times \exp(0.277 \times \text{Number of approaching lanes} \times F_{High\ speed} \times F_{Standard\ phasing} \times F_{Cycle\ facilities} \times F_{Approach\ bus\ bay} \times F_{FLT} \times F_{Commercial}$	$F_{High\ speed} = 1.630$ $F_{Standard\ phasing} = 0.572$ $F_{Cycle\ facilities} = 0.754$ $F_{Approach\ bus\ bay} = 0.692$ $F_{FLT} = 1.604$ $F_{Commercial} = 0.653$
Large	$A_{Rear\ end\ (large,\ peaks)} = 6.42E\text{-}04 \times q^{1.181} \times \exp(0.465 \times \text{Number of approaching lanes} \times (1+\text{Length of right-turn bay or lane})^{-1.478} \times F_{High\ speed} \times F_{Standard\ phasing} \times F_{Cycle\ facilities} \times F_{FLT} \times F_{Commercial}$	$F_{High\ speed} = 1.756$ $F_{Standard\ phasing} = 1.257$ $F_{Cycle\ facilities} = 0.443$ $F_{FLT} = 0.788$ $F_{Commercial} = 0.925$

The total approach traffic volume during peak periods showed a significant relationship with crashes for medium- and large-sized intersections, while the effect for smaller and larger intersections was less pronounced. The standard phasing sequence improved safety at small and medium-sized intersections, but not at large intersections, where split phasing was more common.

The model coefficients also indicate that higher speeds on approaches were a more important factor during the peaks than during the all-day period, with more crashes occurring in high-speed environments. In contrast to the results of the all-day model, the presence of free-left-turn lanes at larger intersections reduced type F crashes during peak periods.

10.3.5 Summary: type F models

Table 10.13 summarises the safety impacts of various factors identified in the type F crash models.

Table 10.13 Summary of type F crash models

Factor	Effect on type F crashes		
	Small intersections	Medium intersections	Large intersections
Higher traffic volume on approach	Increase	Increase	Increase
More approach lanes	No effect	Increase	Increase
Longer lost time	Decrease	Increase	Decrease
Longer right-turn bay/lane	Decrease	No effect	Decrease
Split phasing	Increase	Increase	Decrease
Upstream bus bay within 100m	Increase	Decrease	No effect
Free left turns	Increase	Increase	Increase
Cycle facilities	Decrease	Decrease	Increase
High speed limit (>=80kph)	No effect	Increase	No effect

10.4 Loss-of-control crashes (NZ types C and D)

A single model covering the whole day and utilising data from all selected intersections was developed for loss-of-control (types C and D) crashes. Equation 10.9 presents the preferred model form.

$$A_{\text{Loss of control}} = B_0 \times q^{0.541} \times \exp(0.144 \times \text{Number of approaching lanes}) \times (\text{Cycle time})^{-0.704} \times (\text{Degree of saturation})^{0.447} \times F_{\text{Residential}} \times F_{\text{Split Phasing}} \times F_{\text{Upstream parking}} \times F_{\text{Exit merge}} \times F_{\text{FLT}} \times F_{\text{High speed}} \times F_{\text{Approach bus bay}} \quad \text{(Equation 10.9)}$$

Table 10.14 Model variables

Factor	Value	Description
$B_{0\ (\text{Auckland})}$	2.65E-02	Constant for Auckland
$B_{0\ (\text{Wellington})}$	2.44E-02	Constant for Wellington
$B_{0\ (\text{Christchurch})}$	9.12E-02	Constant for Christchurch
$B_{0\ (\text{Hamilton})}$	1.31E-02	Constant for Hamilton
$B_{0\ (\text{Dunedin})}$	1.11E-01	Constant for Dunedin
$B_{0\ (\text{Melbourne})}$	3.04E-02	Constant for Melbourne
$F_{\text{Split Phasing}}$	2.47	Split phasing on approach
$F_{\text{Upstream parking}}$	0.58	Presence of upstream parking within 100m of limit line
$F_{\text{Exit merge}}$	1.47	Merge present on exit side
F_{FLT}	1.17	Presence of free-left-turn lane for motor vehicles
$F_{\text{High speed}}$	1.57	Speed limit of 80kph or more on approach
$F_{\text{Approach bus bay}}$	1.60	Presence of upstream bus bay within 100m of limit line
$F_{\text{Residential}}$	0.75	Residential land use

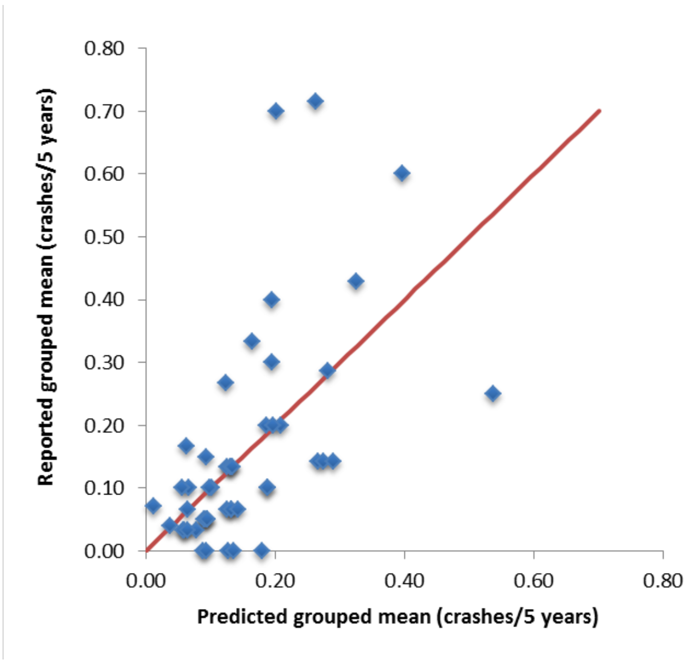
Where:

$A_{\text{Loss of control}}$ Number of predicted type C and D injury crashes in 5 years

q Total daily traffic volume entering the intersection from the approach.

The error structure for this model was found to be negative binomial. Figure 10.6 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 10.9 has a p-value of 0.062, which indicates that the model performed well against the standard goodness-of-fit criteria.

Figure 10.6 Predicted vs reported grouped means of crashes



Higher traffic volumes and wider approaches are observed to result in more type C and D crashes. The model also indicates that more type C and D crashes occurred at intersection approaches that were near-saturated or over-saturated. Increasing the cycle time could result in improved safety.

Fewer type C and D crashes were observed at approaches with parking within 100m of the limit line, suggesting more caution on the part of drivers approaching the intersection. Use of split phasing resulted in a large increase in crashes, while the presence of an exit merge, free-left-turn lane, upstream bus bay (within 100m) and speed limit of 80kph or more also caused more type C and D crashes. Sites located in residential areas had fewer crashes than those in commercial or industrial zones.

10.5 Other crashes

One model was developed for all other crash types that were not looked at individually. Separate peak-period models were not built in this case. Equation 10.10 presents the preferred model form.

$$A_{Other} = B_0 \times q^{0.262} \times (\text{Approach width})^{0.027} \times (\text{Cycle time})^{0.354} \times F_{FLT} \times F_{Coordinated} \times F_{Shared\ turns} \times F_{Split\ phasing} \times F_{Adv\ detector} \times F_{High\ speed} \times F_{Approach\ bus\ bay} \times F_{Upstream\ parking} \times F_{Exit\ merge} \times F_{Commercial} \quad \text{(Equation 10.10)}$$

Table 10.15 Model variables

Factor	Value	Description
$B_{0\ (Auckland)}$	1.87E-03	Constant for Auckland
$B_{0\ (Wellington)}$	1.46E-03	Constant for Wellington
$B_{0\ (Christchurch)}$	2.32E-03	Constant for Christchurch
$B_{0\ (Hamilton)}$	2.02E-03	Constant for Hamilton
$B_{0\ (Dunedin)}$	2.38E-03	Constant for Dunedin
$B_{0\ (Melbourne)}$	1.55E-03	Constant for Melbourne
$F_{Split\ Phasing}$	1.21	Split phasing on approach
$F_{Coordinated}$	0.71	Signal coordination with upstream intersection
$F_{Shared\ turns}$	1.26	Lanes with shared movements (left-turn/through, right-turn/through, or both) present on approach
$F_{Adv\ detector}$	0.44	Presence of advanced detector on approach
$F_{High\ speed}$	1.98	Speed limit of 80kph or more on approach
F_{FLT}	1.16	Presence of free-left-turn lane for motor vehicles
$F_{Approach\ bus\ bay}$	1.27	Presence of upstream bus bay within 100m of limit line
$F_{Upstream\ parking}$	0.70	Presence of upstream parking within 100m of limit line
$F_{Exit\ merge}$	0.65	Merge present on exit side
$F_{Commercial}$	1.83	Commercial land use

Where:

A_{Other} = number of other predicted injury crashes in 5 years

q = total daily traffic volume entering the intersection from the approach.

No goodness-of-fit testing was conducted for this model, due to the variety of crashes represented therein. However, the model results did provide an indication of the most important factors that affected ‘other’ type crashes at signalised intersections.

A range of factors appear to have prominence in this model, which was an expected outcome because of the variety of crash types included in the ‘other crashes’ category. Some of the key results of this model suggest that longer cycle times, split phasing, shared left-turn/through or right-turn/through lanes, high-speed environments and upstream bus bays within 100m increased crashes, while signal coordination, parking within 100m of the limit line, exit merge and advanced detector loops reduced crashes. Commercial areas had a significantly higher rate of ‘other’ crashes than residential areas.

11 Pedestrian–motor vehicle crash models

Models were developed for predicting the two most prominent crashes involving pedestrians – namely, right-angle (NZ types NA and NB) and right-turning/pedestrian crossing crashes (NZ types ND and NF).

11.1 Right-angle crashes (NZ type NA and NB)

Two models were developed for type HA crashes involving motor vehicles and pedestrians. The first model looked at all selected intersections. As in the earlier models, this model did not have a high goodness of fit due to the level of variation for sites in different cities.

A second model was then developed for the Auckland and Melbourne intersections, due to their apparent similarities.

11.1.1 All-cities model

Equation 11.1 presents the preferred model form.

$$A_{NA,NB} = B_0 \text{ X } q^{0.314} \text{ X } p^{0.364} \text{ X } \exp(0.16 \text{ X Number of approaching lanes}) \text{ X } (\text{All-red time})^{0.61} \text{ X } (\text{Cycle time})^{0.810} \text{ X } F_{\text{Cycle facilities}} \text{ X } F_{\text{Shared turns}} \text{ X } F_{\text{Split phasing}} \text{ X } F_{\text{Med island}} \quad \text{(Equation 11.1)}$$

Table 11.1 Model variables

Factor	Value	Description
$B_{0\ (Auckland)}$	3.84E-05	Constant for Auckland
$B_{0\ (Wellington)}$	1.28E-05	Constant for Wellington
$B_{0\ (Christchurch)}$	5.30E-05	Constant for Christchurch
$B_{0\ (Hamilton)}$	5.94E-05	Constant for Hamilton
$B_{0\ (Dunedin)}$	8.90E-05	Constant for Dunedin
$B_{0\ (Melbourne)}$	3.39E-05	Constant for Melbourne
$F_{Cycle\ facilities}$	0.513	Presence of facilities for cyclists (eg cycle lanes and/or storage boxes)
$F_{Shared\ turns}$	1.321	Presence of lanes with shared turning movements (eg left-turn/through, right-turn/through, or both)
$F_{Split\ phasing}$	0.741	Signal coordination with upstream intersection
$F_{Med\ island}$	0.767	Presence of raised median/central island on approach with pedestrian movement

Where:

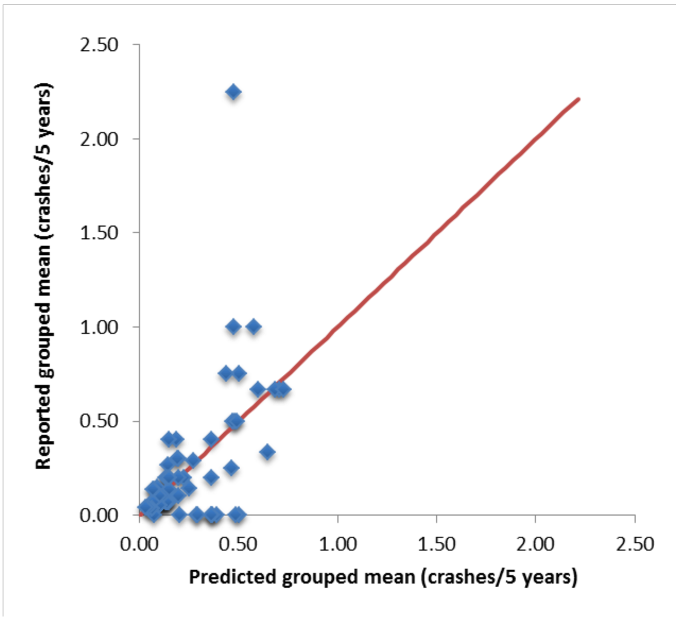
$A_{NA,NB}$ = number of predicted NA and NB injury crashes in 5 years

q = total daily traffic volume entering the intersection from the approach

p = pedestrian volume bin on the approach (on a scale of 1–5, with 1 being low and 5 being high).

The error structure for this model was found to be negative binomial. Figure 11.1 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 11.1 has a p-value of 0.036, which indicates that the model did not have a high goodness of fit.

Figure 11.1 Predicted vs reported grouped means of crashes



The coefficients for traffic volume and pedestrian volume bin in the above model are similar, which indicates that they were equally important factors. Wider approaches were predicted to have more type NA/NB crashes. The variable coefficients for cycle time and all-red time suggested that increasing the length of the signal cycle resulted in more pedestrian-vehicle crashes, possibly as a result of pedestrian frustration.

A split-signal-phasing sequence, the presence of a raised median and cycle facilities on the approach resulted in reduced crash numbers. In the case of the latter, drivers were probably more cautious of other road users around them, which led to a reduction in crashes involving pedestrians as well. On the other hand, the presence of lanes with shared turning movements caused more type HA/NB crashes.

11.1.2 Auckland and Melbourne model

The variation in B_0 values for Auckland and Melbourne in equation 11.1 were similar. A separate model for the Auckland and Melbourne sites was therefore developed to limit some of the variation that was apparent in the first model. Equation 11.2 presents the preferred model form.

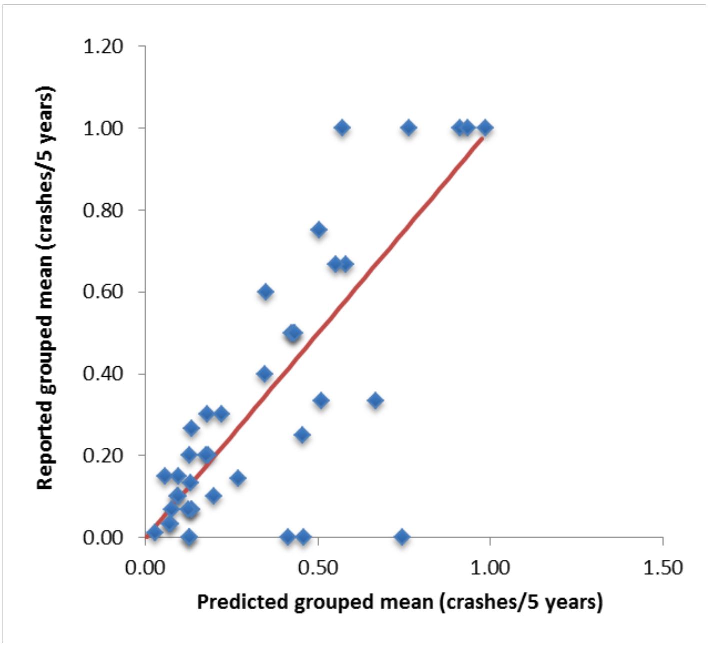
$$A_{HA,NB} = B_0 \text{ X } q^{0.188} \text{ X } p^{0.406} \text{ X } \exp(0.275 \text{ X Number of approaching lanes}) \text{ X (All-red time)}^{0.444} \text{ X (Cycle time)}^{0.646} \text{ X } F_{\text{Cycle facilities}} \text{ X } F_{\text{Shared turns}} \text{ X } F_{\text{Split phasing}} \text{ X } F_{\text{Med island}} \quad \text{(Equation 11.2)}$$

Table 11.2 Model variables

Factor	Value	Description
B_0	1.84E-04	Constant
$F_{\text{Cycle facilities}}$	0.673	Fully protected right-turn phasing
$F_{\text{Shared turns}}$	1.414	Presence of shared turns on approach (eg left-turn/through, right-turn/through, or both)
$F_{\text{Split phasing}}$	0.550	Signal coordination with upstream intersection
$F_{\text{Med island}}$	0.710	Presence of raised median/central island on approach with pedestrian movement

The error structure for this model was found to be Poisson. Figure 11.2 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. As expected, the combined Auckland and Melbourne displayed an improved goodness of fit with a p-value of 0.29.

Figure 11.2 Predicted vs reported grouped means of crashes



The coefficient of total approach volume, q, was lower for the Auckland/Melbourne model than for the model for all cities. A split phasing sequence also showed a higher benefit at the Auckland and Melbourne intersections. The values of the other variables were similar to those found in the model for all cities.

11.1.3 Summary: NA and NB crashes

Table 11.3 summarises the safety impacts of various factors identified in the NA and NB crash type models.

Table 11.3 Summary of NA and NB crash models

Factor	Effect on type NA & NB crashes	
	All sites	Auckland & Melbourne
Higher approaching traffic volume	Increase	Increase
Higher pedestrian volume (bin)	Increase	Increase
More approach lanes	Increase	Increase
Longer cycle time	Increase	Increase
Longer all-red time	Increase	Increase
Split phasing	Decrease	Decrease
Shared turns	Increase	Increase
Raised median/central island	Decrease	Decrease
Cycle facilities	Decrease	Decrease

11.2 Right-turning crashes (NZ type ND and NF)

A single model was developed for right-turning crashes, utilising data from the New Zealand intersections. No data on right-turning crashes involving pedestrians was found in the Melbourne crash database. As a result, the Melbourne intersections were excluded from this model.

Equation 11.3 presents the preferred model form.

$$A_{ND,NF} = B_0 \times q_1^{0.093} \times p^{0.172} \times (\text{Cycle time})^{-0.579} \times (\text{Yellow time})^{0.837} \times F_{\text{Full RT Protection}} \times F_{\text{Residential}} \times F_{\text{Coordinated}} \times F_{\text{Med island}} \quad \text{(Equation 11.3)}$$

Table 11.4 Model variables

Factor	Value	Description
$B_{0\ (\text{Auckland})}$	3.10E-02	Constant for Auckland
$B_{0\ (\text{Wellington})}$	1.03E-01	Constant for Wellington
$B_{0\ (\text{Christchurch})}$	1.09E-01	Constant for Christchurch
$B_{0\ (\text{Hamilton})}$	1.93E-02	Constant for Hamilton
$B_{0\ (\text{Dunedin})}$	2.24E-01	Constant for Dunedin
$F_{\text{Full RT Protection}}$	0.63	Fully protected right-turn phasing
$F_{\text{Residential}}$	0.57	Residential land use
$F_{\text{Coordinated}}$	1.24	Signal coordination with upstream intersection
$F_{\text{Med island}}$	0.99	Presence of raised median/central island on approach with pedestrian movement

Where:

$A_{\text{Loss of control}}$ = number of predicted loss-of-control type C and D injury crashes in 5 years

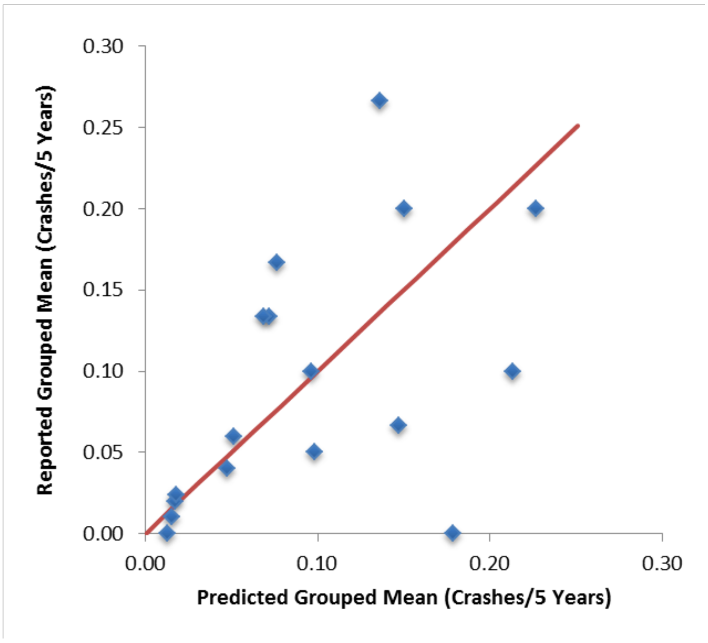
q_1 = daily volume of right-turning vehicles on movement 1 (see approach movement coding)

p = pedestrian volume bin on the side road (on a scale of 1–5, with 1 being low and 5 being high).

The error structure for this model was found to be negative binomial.

Figure 11.3 presents the comparison between the predicted and reported grouped means of crashes for the preferred model. Equation 11.3 has a p-value of 0.056, which indicates that the model had a reasonable goodness of fit.

Figure 11.3 Predicted vs reported grouped means of crashes



The model results suggest that pedestrian volume was a more important factor than motor vehicle volume when it came to right-turning crashes involving pedestrians. Longer cycle times were observed to reduce crashes; however, longer yellow times resulted in an increase in crashes. Fully protected right turns were quite beneficial from a safety perspective, while coordinated signals usually had more ND/NF crashes. The presence of a median for crossing pedestrians was not found to have a significant effect on safety.

12 Model application and toolkit

This section presents examples of practical improvements to existing intersections and how safety may be improved.

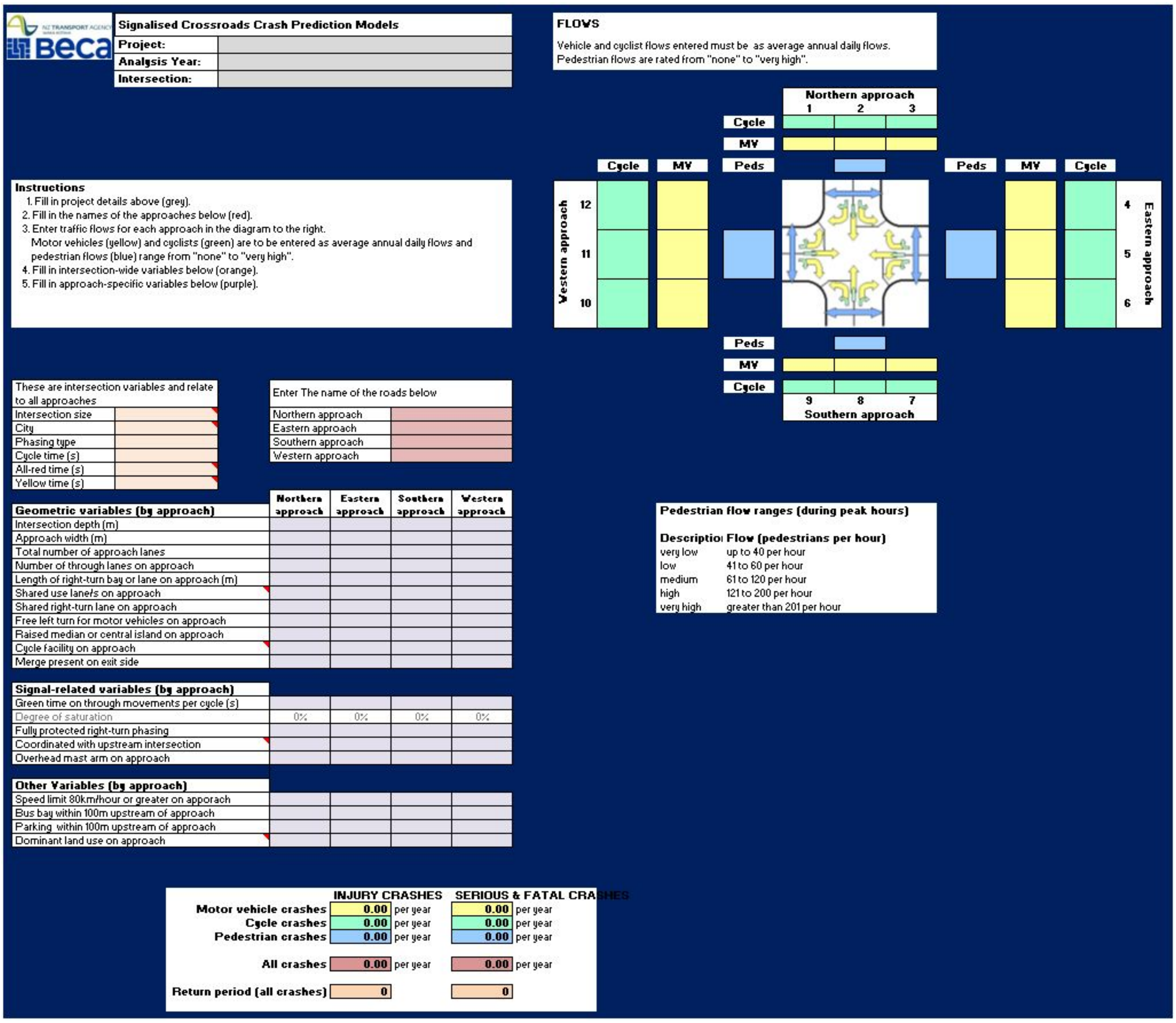
Three intersections of varying size were selected from around New Zealand and safety improvement scenarios identified for each. Next, the crash prediction models developed during this study were applied to calculate the number of injury crashes on the intersections, thereby estimating the safety benefits (through lower crash numbers) expected from the improvements.

These scenarios used the motor vehicle all-cities and whole-day models developed in chapter 10.

The baseline scenario for the intersections used the data from the Master Database (section 4.6).

An Excel toolkit was developed to test the baseline scenario and the improvements. The toolkit could be used to see how changes to an intersection impact on its safety benefits. A screenshot of the toolkit is shown in figure 12.1.

Figure 12.1 The toolkit



The toolkit is available through the NZTA and the research authors.

12.1 Fanshawe Street and Halsey Street intersection

The intersection of Fanshawe Street and Halsey Street is located within Auckland. The intersection has an AADT of 52,000 vehicles entering the intersection. Fanshawe Street is the major road, with five approach lanes, and generates 75% of the intersection traffic. Halsey Street has four approach lanes on the south approach and two on the north approach.

The baseline scenario, where there is no change to the intersection, predicts 1.61 injury crashes per year (one crash per seven months). If the intersection phasing type is changed from split phasing to standard phasing, then the model predicts 1.40 injury crashes per year (one crash per nine months). This is a reduction in injury crashes of 13%.

If the standard phasing remains and the all-red time increases from 1sec to 2sec, the model predicts 1.18 injury crashes per year (one crash per 10 months). This is a further reduction in injury crashes of 16%. Compared with the baseline scenario, implementing both improvements would reduce injury crashes by 27%.

The results are summarised in table 12.1.

Table 12.1 Safety improvement scenarios for intersection A

	Baseline scenario	Improvement 1 – split phasing	Improvement 2[a]– increase all-red time
Predicted crashes (per year)	1.61	1.40	1.18
Percentage reduction	-	13%	27%

a) Improvements 1 and 2 are implemented simultaneously

12.2 Fitzgerald Avenue and Gloucester Street intersection

The intersection of Fitzgerald Avenue and Gloucester Street is located within Christchurch. The intersection has an AADT of 56,000 vehicles entering the intersection. Fitzgerald Avenue is the major road, with five approach lanes, and generates 78% of the intersection traffic. Gloucester Street has two approach lanes on both approaches.

The baseline scenario, where there is no change to the intersection, predicts 1.87 injury crashes per year (one crash per six months). If overhead mast arms are erected on each approach, then the model predicts 1.60 injury crashes per year (one crash per eight months). This is a reduction in injury crashes of 14%.

If the overhead mast arms remain and the all-red time increases from 1sec to 2sec, the model predicts 1.35 injury crashes per year (one crash per nine months). This is a further reduction in injury crashes of 16%. Compared with the baseline scenario, implementing both improvements would reduce injury crashes by 28%.

The results are summarised in table 12.2.

Table 12.2 Safety improvement scenarios for intersection B

	Baseline scenario	Improvement 1 – overhead mast arms	Improvement 2[a] – increase all red time
Predicted crashes (per year)	1.87	1.60	1.35
Percentage reduction	-	14%	28%

a) Improvements 1 and 2 are implemented simultaneously.

12.3 Bay View Road and King Edward Street intersection

The intersection of Bay View Road, King Edward Street and Prince Albert Road is located within Dunedin. The intersection has an AADT of 18,000 vehicles entering the intersection. King Edward Street and Prince Albert Road are the major roads, with two approach lanes, and generate 63% of the intersection traffic. Bay View Road has two approach lanes on both approaches.

The baseline scenario, where there is no change to the intersection, predicts 1.39 injury crashes per year (one crash per nine months). If the traffic signals provide fully protected right-turn phasing on each approach, then the model predicts 1.31 injury crashes per year (one crash per nine months). This is a small reduction in injury crashes, of 6%.

If the fully protected right-turn phase remains and overhead mast arms are erected, the mode predicts 1.21 injury crashes per year (one crash per 13 months). This is a further reduction in injury crashes of 8%. Compared with the baseline scenario, implementing both improvements would reduce injury crashes by 13%.

The results are summarised in table 12.3.

Table 12.3 Safety improvement scenarios for intersection C

	Baseline scenario	Improvement 1 – fully protected right turn	Improvement 2[a] – overhead mast arms
Predicted crashes (per year)	1.39	1.31	1.21
Percentage reduction	-	6%	13%

a) Improvements 1 and 2 are implemented simultaneously.

13 Summary and conclusions

13.1 Relative impacts of intersection treatments

A significant advantage of building crash prediction models for different crash types is that this provides a holistic overview of safety at an intersection. The effects of various intersection treatments on safety for the various crash types shows that some treatments have a positive effect on safety for certain crash types, while negatively affecting other movements.

Table 13.1 provides a summary of results from the models that have been developed as part of this study. The first column lists all the factors that were found to be significant in one or more of the models. The subsequent columns list whether the factor led to an increase (red), decrease (green) or no effect (grey) on the rate of crashes of the respective crash types.

It is acknowledged that some of the results in the table contradict other results, and some do not agree with other research and experiences on the safety or otherwise of various factors. What can be said is that if the majority of models show an increase or decrease in crashes, then this gives us some confidence in the results. To actually confirm the level of effect – eg whether there is a 20% or 40% reduction in crashes – would require a comparison with other studies of this type. If the studies are relatively consistent, then we can have confidence in the results. Going forward, it would be worthwhile to have the research results reviewed by an expert panel and issued as a guidance note to the industry.

Table 13.1 Effect of intersection parameters on crash types

Legend

⇧	Increase in crashes due to the parameter
⇩	Decrease in crashes due to the parameter
(blank)	No identified change in crash rate due to the parameter

Parameter	Motor vehicle crashes										Pedestrian-motor vehicle crashes			Comments
	Right-angle			Right-turn-against		Loss-of-control	Rear-end			Other	Right-angle		Right-turning	
	Overall model	Auckland and Melbourne model	Peak-periods model	Overall model	Peak-periods model		Small intersections model	Medium intersections model	Large intersections model		Overall model	Auckland and Melbourne model		
Higher approaching traffic volume	⇧	⇧	⇧			⇧	⇧	⇧	⇧	⇧	⇧	⇧		Higher traffic volumes (or pedestrian volumes in the case of pedestrian-motor vehicle crashes) led to higher crash rates across all crash types.
Higher right-turning traffic volume				⇧	⇧								⇧	
Higher degree of saturation				⇧	⇧	⇧								
Higher pedestrian volume											⇧	⇧	⇧	
Larger intersection size (number of approach lanes, intersection depth)	⇧	⇧	⇧											Larger intersections (ie those with a larger intersection depth, more approach/through lanes) also had more crashes, except for right-turning crashes involving pedestrians, for which intersection size was not found to be a factor. Intersection size was found to be especially important for rear-end crashes, and three separate models (for small, medium and large intersections) were built as a result.
More approach lanes						⇧		⇧	⇧		⇧	⇧		
More through lanes				⇧	⇧									
Wider approaches										⇧				
Longer cycle time	⇩	⇩	⇩	⇩	⇩	⇩				⇧	⇧	⇧	⇩	Longer cycle times were safer for right-angle, right-turn-against , loss-of-control crashes, and crashes involving pedestrians and right-turning motor vehicles, but less safe for 'other' type crashes and right-angle crashes involving pedestrians. Rear-end crashes were unaffected by the length of the signal cycle. Longer all-red times were safer for right-angle crashes involving motor vehicles, but less safe for right-angle crashes involving pedestrians.Yellow time did not figure in most models, except for right-turning crashes involving pedestrians, where longer yellow times were less safe.
Longer all-red time	⇩	⇩	⇩								⇧	⇧		
Longer yellow time													⇧	
Longer lost time (inter-green and all-red time)							⇩	⇧	⇩					The frequency of rear-end crashes depended on total lost time, and not the total cycle time. Small and large intersections had fewer rear-

Parameter	Motor vehicle crashes										Pedestrian-motor vehicle crashes			Comments
	Right-angle			Right-turn-against		Loss-of-control	Rear-end			Other	Right-angle		Right-turning	
	Overall model	Auckland and Melbourne model	Peak-periods model	Overall model	Peak-periods model		Small intersections model	Medium intersections model	Large intersections model		Overall model	Auckland and Melbourne model		
														end crashes when lost times were higher, as opposed to medium-sized intersections, where longer lost times caused fewer rear-end crashes.
Full right-turn protection				⇩	⇩								⇩	The provision of fully protected right turns for vehicles had a positive effect on right-turn-against crashes and right-turning crashes involving pedestrians.
Split phasing	⇩	⇩	⇩			⇧	⇧	⇧	⇩	⇧	⇩	⇩		Intersections operating in a split phasing sequence had fewer right-angle crashes (both motor vehicles and pedestrians), but more loss-of-control and 'other' crashes. Split phasing reduced rear-end crashes at large intersections, but increased them at small and medium-sized intersections.
Mast arm	⇩	⇩	⇩											The presence of mast arms reduced right-angle crashes involving motor vehicles, but did not affect any of the other crash types.
Coordinated signals	⇧	⇩	⇧							⇩			⇧	Signals lying along a coordinated corridor affected only right-angle MV crashes (increased, except for Auckland and Melbourne), other crashes (decreased) and pedestrian right-turning crashes (increased).
Additional advanced detectors	⇧	⇧								⇩				The presence of an advanced SCATS® detector loop increased right-angle MV crashes, but reduced other crashes.
Shared turns (eg left-turn/through and right-turn/through lanes)	⇧	⇩	⇧							⇧	⇧	⇧		Lanes with shared turns were generally unsafe (higher right-angle – both MV and pedestrians – and other crashes). However, intersections with shared right turns had fewer right-turn-against crashes.
Shared right-turn/through lane				⇩	⇩									
Raised median/central island	⇩	⇩	⇧	⇧	⇧						⇩	⇩		The presence of a raised median reduced right-angle (MV and pedestrian) crashes but increased right-turn-against crashes.
Longer right-turn bay/lane				⇩	⇧		⇩		⇩					Longer right-turn bays/lanes reduced right-turn-against crashes and rear-end crashes at small and large intersections. The other crash types did not show any relationship with this factor.
Free left turn for motor						⇧	⇧	⇧	⇧	⇧				Free left turns increased the risk of loss-of-control, rear-end and

Parameter	Motor vehicle crashes										Pedestrian-motor vehicle crashes			Comments
	Right-angle			Right-turn-against		Loss-of-control	Rear-end			Other	Right-angle		Right-turning	
	Overall model	Auckland and Melbourne model	Peak-periods model	Overall model	Peak-periods model		Small intersections model	Medium intersections model	Large intersections model		Overall model	Auckland and Melbourne model		
vehicles														other crashes.
Presence of merge on intersection exit						⇧				⇩				The presence of exit merges increased loss-of-control crashes but reduced other crashes.
Cycle facilities				⇧			⇩	⇩	⇧		⇩	⇩		
Upstream bus bay within 100m						⇧	⇧	⇩		⇧				Approach bus bays had mixed effects on safety – higher loss-of-control and other crashes, and higher rear-end crashes at small intersections but fewer rear-end crashes at medium-sized intersections.
Upstream parking						⇩				⇩				Approaches with upstream parking had fewer loss-of-control and other crashes.
High speed limit (>=80kph)						⇧		⇧		⇧				Approaches with high-speed limits (80kph or more) had more loss-of-control, rear-end and 'other' crashes.
Commercial land use										⇧				
Residential land use						⇩							⇩	

13.2 Peak-period crashes

Peak-period crashes account for a significant proportion of crashes at signalised intersections (see table 13.2). Of the sites selected, type F crashes were particularly prevalent during AM and PM peaks.

Table 13.2 Percentage of peak-period crashes, by crash type

City	Number of crashes: All-Day	Number of crashes: AM & PM peaks	% of crashes occuring in AM & PM peaks
Auckland	362	81	22%
Christchurch	271	71	26%
Dunedin	74	17	23%
Hamilton	66	15	23%
Melbourne	359	99	28%
Wellington	30	13	43%
Grand Total	1162	296	25%

The following sections present an analysis of the peak-period crash type models that were developed.

13.2.1 Right-angle (HA) crashes

Traffic volume had a less pronounced effect for type HA crashes occurring in peak periods than during the whole day. Factors related to intersection size (such as intersection depth and number of approach lanes) were more significant in the peaks, suggesting that larger intersections had a particularly high proportion of peak-period type HA crashes.

Longer cycle and all-red times, and the presence of split phasing and mast arms, had a significant effect on improving safety for type HA crashes in peak periods.

The presence of a raised median or central island was observed to increase type HA crashes in the peaks, as opposed to the whole day where the presence of these features improved safety for this crash type.

13.2.2 Right-turn-against (LB) crashes

Right-turning traffic volume had a more pronounced effect on type LB crashes in the peaks. However, factors such as longer cycle time, full right-turn protection and the presence of shared right-turn lanes were relatively less significant during the peaks. In contrast, raised medians or central islands were observed to be more unsafe in peak periods.

The effect of longer right-turn bays or lanes was different in the peaks than during the whole day. While the models showed that longer right-turn bays or lanes resulted in a reduction in crashes of this type during the whole day, the effect was the opposite in the peaks, with an increase in crashes. This is possibly a result of sites with higher right-turning traffic volumes being provided with longer right-turn bays.

13.2.3 Rear-end (type F) crashes

Separate models were built according to size of the intersection (small, medium or large) for type F crashes. The models indicate that peak-period traffic volume was a significant factor for the large intersections. The presence of cycle facilities and free left turns for motor vehicles at large intersections reduced type F crashes, as opposed to the increase that was observed for the whole day at these intersections.

Use of a split phasing sequence at small intersections was still shown to be less safe, although to a much lesser extent in the peaks. The model for medium-sized intersections showed similar trends for both the peak and all-day periods.

13.3 City covariates

Table 13.3 lists the city covariate values for the models developed during this study. The values provide an indication of the relative occurrence of crashes of each type in the various locations.

Table 13.3 Model city covariate values

Crash type	B_0 (Auckland)	B_0 (Wellington)	B_0 (Christchurch)	B_0 (Hamilton)	B_0 (Dunedin)	B_0 (Melbourne)	Trend
Right angle	4.27E-05	2.08E-05	8.69E-05	1.13E-04	1.54E-04	4.11E-05	
Right turn against	3.83E+00	4.10E+00	4.41E+00	2.27E+00	4.16E+00	3.95E+00	
Rear end crashes (small intersections)	1.38E+00	6.58E-01	4.34E+00	1.36E+00	7.95E+00	1.25E+00	
Rear end crashes (medium intersections)	9.56E-04	1.41E-03	1.12E-03	9.29E-04	3.06E-03	1.16E-03	
Rear end crashes (large intersections)	3.92E+00		7.74E-01			3.36E+00	
Loss of control	2.65E-02	2.44E-02	9.12E-02	1.31E-02	1.11E-01	3.04E-02	
Other	1.87E-03	1.46E-03	2.32E-03	2.02E-03	2.38E-03	1.55E-03	
Pedestrian – right angle	3.84E-05	1.28E-05	5.30E-05	5.94E-05	8.90E-05	3.39E-05	
Pedestrian – right turning	3.10E-02	1.03E-01	1.09E-01	1.93E-02	2.24E-01		

13.4 Need for future research

13.4.1 Cycle data collection

The cycle data collected as part of this study proved insufficient for developing crash prediction models for the prominent cycle–motor vehicle crash types. There is a need for more and better-quality cycle data from signalised intersections in New Zealand. Future studies should ideally consider a larger sample set for the analysis of cycle–motor vehicle crashes.

Data from 102 signalised intersections is already available as part of research conducted for Austroads (2000). There is scope for building upon the data to include additional sites as well as intersection phasing information for the existing intersections. This will enable a more comprehensive dataset to be built, which can be drawn upon for future studies.

13.4.2 Before/after studies of intersection treatments

The effect of certain intersection treatments was not immediately obvious from the crash prediction models that have been developed. Part of the reason for this is the numerous correlations that are found as a result of having a large dataset (cross-sectional models also have limitations that can make it difficult to fully understand the effects of some road features). Before-and-after studies are one way of assessing the impact of these factors. These would require careful monitoring of the site(s) for a certain period, both before and after intersection upgrades have been implemented at the relevant site(s).

13.4.3 Crash severity reduction

Since this research was undertaken, the NZ Transport Agency has moved to a safer system approach, where there is a particular focus on serious and fatal crashes. While it is not appropriate, given the scarcity of crashes, to build models for only serious and fatal crashes, further research is required to understand what road features and traffic operation are more likely to cause more serious crashes. Such research will be required to enable future development of the NZTA's High Risk Intersection Guide (HRIG).

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Appendix A: Selected intersections

Site ID	Location	Site name
11301	Auckland	Lake/Bayswater/Williamson
11401	Auckland	Wairau/Glenfield
11603	Auckland	East Coast/Hastings
11604	Auckland	East Coast/Sunrise
11610	Auckland	Oteha Valley/East Coast
11613	Auckland	East Coast/Rosedale
11615	Auckland	Rosedale/Bush
11905	Auckland	Oteha Valley Extn/Sh17
11923	Auckland	Oteha Valley/SH17
12001	Auckland	Albert/Customs/Fanshawe
12003	Auckland	Ponsonby/Karngahape
12008	Auckland	Karangahape/Upper Queen
12023	Auckland	Union/Wellington
12028	Auckland	Customs/Commerce
12029	Auckland	Queen/Customs
12030	Auckland	Remuera/Orakei/Ascot
12040	Auckland	Halsey/Fanshawe
12041	Auckland	Albert/Victoria
12051	Auckland	Ponsonby/Richmond/Picton
12054	Auckland	Quay/Commerce
12062	Auckland	Albert/Wyndham
12066	Auckland	Parnell Rise/Beach/The Strand/Stanley
12071	Auckland	Quay/Tamaki/The Strand
12094	Auckland	Vincent/Hopetoun/Pitt
12101	Auckland	Mayoral/Greys
12102	Auckland	Mayoral/Vincent
12105	Auckland	Customs/Gore
12109	Auckland	Rosebank/Ash
12110	Auckland	Great North/Ash
12123	Auckland	Great North/Alford
12124	Auckland	Great North/Blockhouse Bay
12138	Auckland	Dominion/Walters/Valley
12154	Auckland	Bond/New North/Sandringham
12156	Auckland	Neilson/Onehunga Mall
12157	Auckland	Selwyn/Church
12159	Auckland	Onehunga/Mount Smart
12161	Auckland	Selwyn/Neilson
12164	Auckland	Church/Captain Springs
12172	Auckland	Church/Hugo Johnston

Site ID	Location	Site name
12210	Auckland	Mangere/Golf
12257	Auckland	Great South/South Eastern Hwy
12276	Auckland	Mangere/Chelsea
12314	Auckland	Princes/Selwyn
12904	Auckland	Stanley/Grafton
13004	Auckland	Great North/Veronica
13005	Auckland	Great North/Memorial
13007	Auckland	Great North/Titirangi
14101	Auckland	Massey/Buckland
14102	Auckland	Mckenzie/Coronation/Walmsley
14109	Auckland	Massey/Henwood
14111	Auckland	Massey/Savill
14111	Auckland	Mangere/Savill
14202	Auckland	Pakuranga/Ti Rakau
14207	Auckland	Bucklands Beach/Pakuranga
14219	Auckland	Ti Rakau/Botany/Te Inirangi
14221	Auckland	Ti Rakau/Dannemora/Chapel
14223	Auckland	Ti Rakau/Burswood
14224	Auckland	Chapel/Kilkenny
14225	Auckland	Ti Rakau/Harris
14227	Auckland	Chapel/Armoy
14229	Auckland	Ti Rakau/Trugood
14235	Auckland	Ti Rakau/Te Koha
14301	Auckland	Great South/Bairds
14302	Auckland	East Tamaki/Bairds
14307	Auckland	East Tamaki/Birmingham
14308	Auckland	Harris/Smales/Allens
14309	Auckland	East Tamaki/Hills
14319	Auckland	East Tamaki/Newbury
14403	Auckland	Great South/Browns/Orams
14502	Auckland	Great South/Shirley
14503	Auckland	Great South/East Tamaki
14505	Auckland	Great South/Tui
14506	Auckland	East Tamaki/Holroyd
14509	Auckland	Great South/Te Irirangi/Cavendish
14517	Auckland	Great South/Kolmar Rd
14602	Auckland	Te Irirangi/Diorella
14603	Auckland	Te Irirangi/Hollyford
14604	Auckland	Te Irirangi/Dawson
14605	Auckland	Te Irirangi/Ormiston
14605	Auckland	Te Irirangi/Ormiston Te Irirangi N
14606	Auckland	Te Irirangi/Accent

Site ID	Location	Site name
14607	Auckland	Te Irirangi/Smales
14608	Auckland	Te Irirangi/Te Koha
14609	Auckland	Te Irirangi/Haven
14610	Auckland	Te Irirangi/Bishop Dunn
12075/1	Auckland	Upper Queen/Canada
12075/2	Auckland	Upper Queen/Ian Mckinnon
12113/1	Auckland	Jervois/Wallace Intersection 1
12113/2	Auckland	Jervois/Wallace Intersection 2
14232-1	Auckland	Ti Rakau/Greenmount
30016	Christchurch	Colombo/Tuam
30019	Christchurch	Manchester/Tuam
30057	Christchurch	Gloucester/Manchester
30059	Christchurch	Hereford/Manchester
30060	Christchurch	Armagh/Manchester
30061	Christchurch	Bealey/Carlton/Harper/Park
30066	Christchurch	Manchester/Worcester
30067	Christchurch	Cashel/Manchester
30069	Christchurch	Falsgrave/Fitzgerald/Moorhouse
30070	Christchurch	Ferry/Fitzgerald
30072	Christchurch	Cashel/Fitzgerald
30073	Christchurch	Fitzgerald/Hereford
30074	Christchurch	Fitzgerald/Worcester
30075	Christchurch	Fitzgerald/Gloucester
30076	Christchurch	Avonside/Fitzgerald/Kilmore
30077	Christchurch	Bealey/Colombo
30078	Christchurch	Bealey/Sherborne
30079	Christchurch	Bealey/Manchester
30088	Christchurch	Bealey/Fitzgerald/London/Whitmore
30089	Christchurch	Armagh/Colombo
30090	Christchurch	Colombo/Gloucester
30095	Christchurch	Armagh/Fitzgerald
30100	Christchurch	Colombo/Moorhouse
30117	Christchurch	Aldwins/Buckleys/Linwood
30119	Christchurch	Hereford/Linwood
30121	Christchurch	Avonside/Stanmore
30126	Christchurch	Gloucester/Stanmore
30129	Christchurch	North Avon/Stanmore
30130	Christchurch	Stanmore/Worcester
30134	Christchurch	Hargood/Keighleys/Linwood
30200	Christchurch	Ferry/Moorhouse/Wilsons
30201	Christchurch	Aldwins/Ensors/Ferry
30202	Christchurch	Ferry/Hargood/Radley

Site ID	Location	Site name
30203	Christchurch	Ferry/Palinurus/Rutherford
30211	Christchurch	Cranford/Edgeware/Sherborne
30212	Christchurch	Berwick/Cranford
30213	Christchurch	Cranford/Westminster
30214	Christchurch	Cranford/Innes
30215	Christchurch	Innes/Rutland
30253	Christchurch	Heaton/Innes/Papanui
30301	Christchurch	Glandovey/Heaton/Rossall/Strowan
30305	Christchurch	Greers/Wairakei
30306	Christchurch	Grahams/Wairakei
30310	Christchurch	Greers/Harewood
30348	Christchurch	Fendalton/Glandovey
30352	Christchurch	Clyde/Fendalton/Memorial
30353	Christchurch	Ilam/Memorial
30354	Christchurch	Greers/Memorial
30355	Christchurch	Grahams/Memorial
30403	Christchurch	Division/Riccarton
30404	Christchurch	Matipo/Riccarton
30407	Christchurch	Ilam/Middleton/Riccarton
30414	Christchurch	Masham/Russley/Yaldhurst
50033	Dunedin	Stuart/Moray Pl
50034	Dunedin	Stuart/Smith St
50040	Dunedin	Bank/North
50043	Dunedin	King Edward/Hillside
50044	Dunedin	King Edward/Macandrew
50047	Dunedin	Coversham Valley Road/Barnes Drive
50049	Dunedin	Bay View/King Edward
50054	Dunedin	Stuart/London
50056	Dunedin	George/Duke
50058	Dunedin	George/Howe/Warrender
50063	Dunedin	Hillside/Burns
20001	Hamilton	Lincoln/Massey
20002	Hamilton	Kent/Hall
20003	Hamilton	Lake/Hall
20004	Hamilton	Norton/Hall
20009	Hamilton	Collingwood/Anglesea
20010	Hamilton	Ward/Anglesea
20016	Hamilton	Willoughby/Anglesea
20022	Hamilton	Ohaupo/Collins
20025	Hamilton	Bryce/Victoria
20031	Hamilton	Peachgrove/Te Aroha
20032	Hamilton	Tristram/Ward

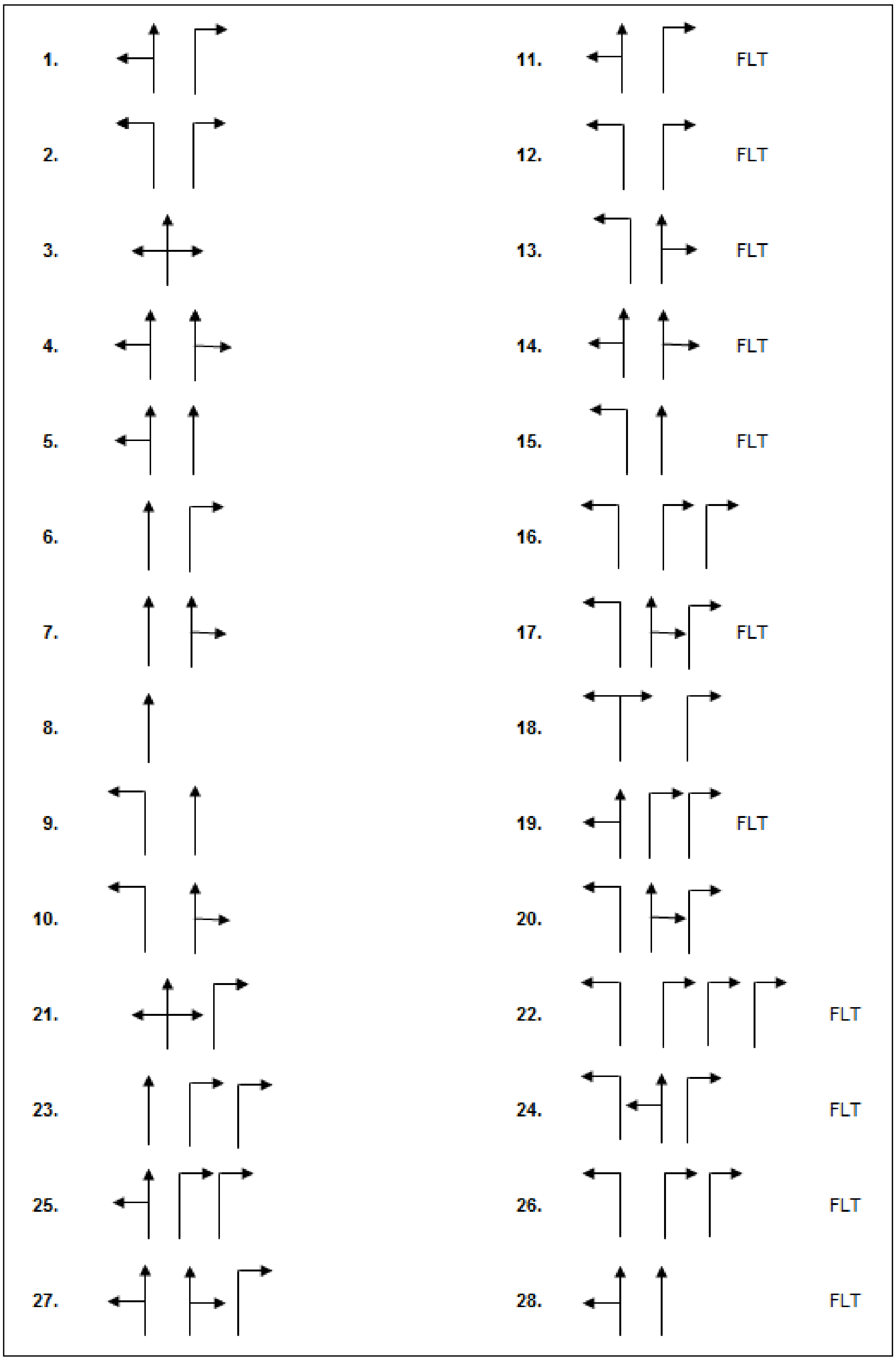
Site ID	Location	Site name
20033	Hamilton	London/Norton/Tristram
20036	Hamilton	Galloway/Naylor
20040	Hamilton	Bryce/Tristram
20042	Hamilton	Normandy/Cobham
20044	Hamilton	Te Rapa/Vardon
20046	Hamilton	Te Rapa/Pukete
60174	Melbourne	Burwood/Scoresby
60252	Melbourne	South Gippsland/Greens/Round Tower
60278	Melbourne	Nepean/Old Mornington/Humphries
60281	Melbourne	Springvale/Wells
60309	Melbourne	Wells/Narelle
60340	Melbourne	Blackburn/King
60369	Melbourne	Boronia/Stud
60417	Melbourne	Springvale/Canterbury
60456	Melbourne	Ferntree Gully/Stud
60495	Melbourne	Hutton/Greens/Perry
60590	Melbourne	Mitcham/Park
60597	Melbourne	Princess/Army
60631	Melbourne	Dorset/Canterbury
60670	Melbourne	Cheltenham/Chandler
60692	Melbourne	George/Victoria
60752	Melbourne	Frankston - Dandenong/Hall/Lathams
60784	Melbourne	Wellington/Stud
60851	Melbourne	Heatherdale/Canterbury
60961	Melbourne	South Gippsland/Abbotts
62332	Melbourne	Nepean/White
62507	Melbourne	Princes/Geelong
62719	Melbourne	Hoffmans/Buckley
62882	Melbourne	Millers/Marigold
62902	Melbourne	Lonsdale/Exhibition
62909	Melbourne	Lonsdale/King
62956	Melbourne	Westall/Centre
63050	Melbourne	Bell/Cumberland
63060	Melbourne	High/Bell
63064	Melbourne	Albert/Bell
63110	Melbourne	Brunswick/Fleming
63117	Melbourne	Normanby/Leinster
63212	Melbourne	Clayton/Boundary/Heatherton/Kingston
63214	Melbourne	High/Spencer/Wood
63221	Melbourne	High/Cramer
63223	Melbourne	High/Regent
63241	Melbourne	Victoria/Dundas

Site ID	Location	Site name
63260	Melbourne	Albert/Tyler
63261	Melbourne	Albert/Wood
63263	Melbourne	Albert/Gower
63264	Melbourne	Albert/Raglan
63265	Melbourne	Albert/Dundas
63431	Melbourne	Docklands/Hyde
63501	Melbourne	Blackshaws/Millers
63502	Melbourne	Millers/Mason
63592	Melbourne	Moreland/Pascoe Vale
63791	Melbourne	Brunswick/Stranger
64196	Melbourne	Buckley/Cooper
64230	Melbourne	Docklands/Williamstown
64240	Melbourne	Barkly/Gordon
64254	Melbourne	Docklands/Wembbley
64255	Melbourne	Docklands/Stephen
64396	Melbourne	Lygon/Grattan
64447	Melbourne	Albert/Clarendon
64448	Melbourne	Albert/Powlett
64647	Melbourne	Boundary/White/Malcolm
64664	Melbourne	Victoria/Darebin
64732	Melbourne	Salmon/Lorimer
64949	Melbourne	Ashley/South
46508	Wellington	Western Hutt/Grounsell Cres
46515	Wellington	River Rd/Fergusson Drive
46516	Wellington	Main Road/Te Moana Street
46517	Wellington	Mana Esp/Pascoe Avenue
46518	Wellington	Mana Esp/Mana View/Station Road
46519	Wellington	SH1/Acheron Rd
46811	Wellington	Waterloo Road/Cornwall Street
46812	Wellington	King Crescent/Cornwall Street
46814	Wellington	Whites Line East/Leighton Ave/Cambridge Tce
46815	Wellington	Whites Line East/Waiwhetu Road/Bell Rd

Appendix B: Lane layout classification

Approaches with a free-left-turn lane were given a separate code (marked as FLT below). Approaches that had the same number and layout of left- and/or right-turn lanes, but had a varying number of through lanes, were also categorised under the same layout type.

Figure B.1 Signalised intersection lane layouts

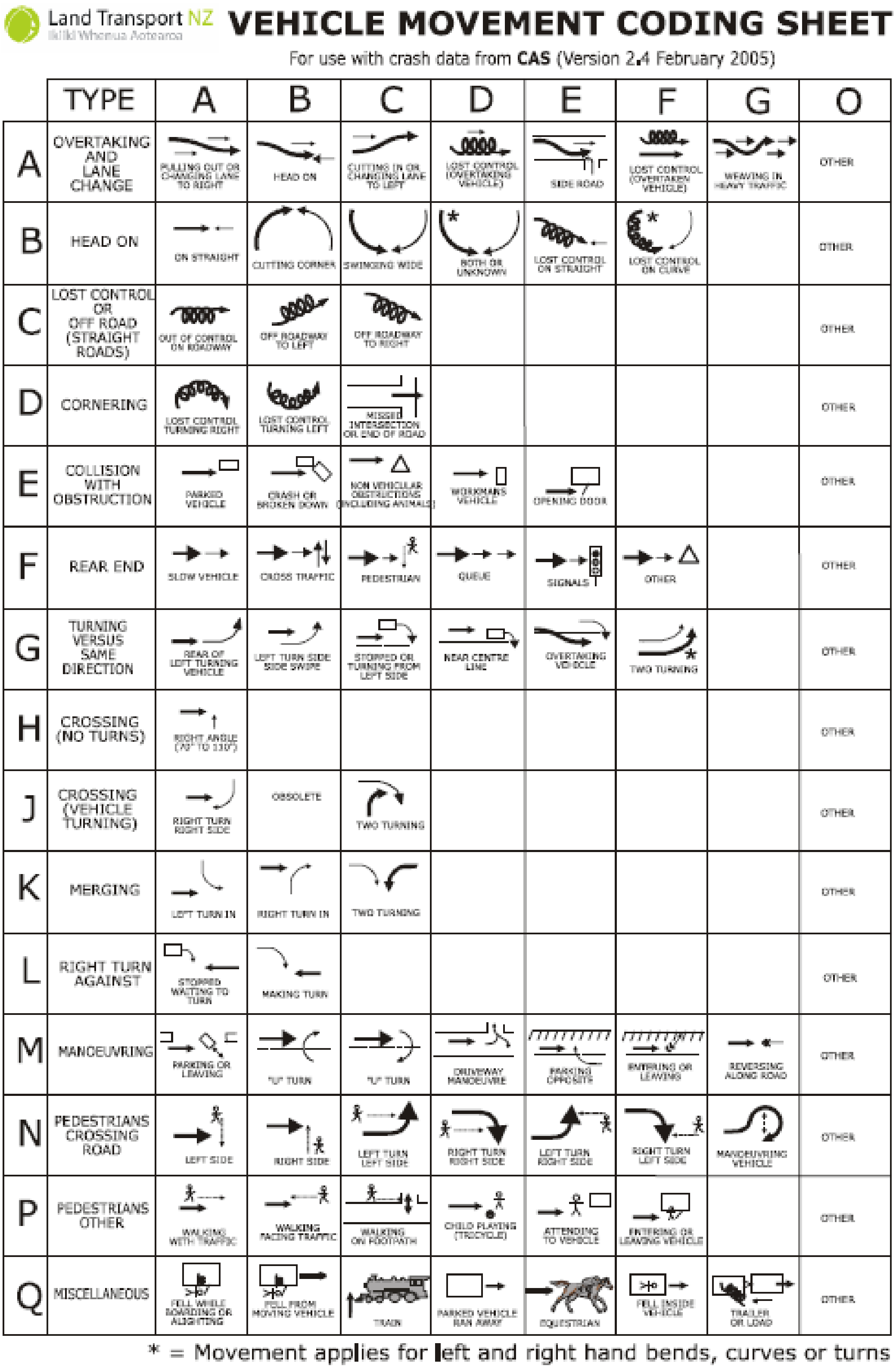


Appendix C: Variable correlation matrix

Correlations	MVCrashes_total	MVCrashes_HA	MVCrashes_LB	MVCrashes_JA	MVCrashes_RearEnd	MVCrashes_LostControl	MVCrashes_Other	CycCrashes_LB	CycCrashes_HA	PedCrashes_NANB	PedCrashes_NDNF	3arm	Highspeed	Coord_allday	Dist_upstrm_int	Phasin_std	Phasin_split	Phasin_cmb	RT_present	RT_fprot	RT_pprot	RT_filter	Ped_present	LT_phase	Barnes_dance	Num_app_lanes	Num_thr_lanes	App_width	Med_island	Int_depth	FLT	Shared_LT	Shared_RT	Shared_turns	Num_exit_lanes	Dist_merge	(1+Len_Rtbay)	(1+Offset_Rtbay)	FLT_ped	Cycle_fac	Cyc_app	Cyc_storage	Dist_parking	App_bus	Exit_bus	LU_Res	LU_Comm	LU_Ind	LU_Rural	Pole_low	Mast_arm	Num_signals	Lights_left	Lights_right	Lights_mastarm	Adv_det	Green	Yellow	Red	App_tot_time	Num_ped_IDM	Cyctime_day	%green	Deg_sat	q_app
MVCrashes_total	1.0
MVCrashes_HA	0.4	1.0
MVCrashes_LB	0.7	0.1	1.0
MVCrashes_JA	0.2	0.0	0.1	1.0
MVCrashes_RearEnd	0.6	0.1	0.2	0.0	1.0
MVCrashes_LostControl	0.3	0.1	0.1	0.1	0.1	1.0
MVCrashes_Other	0.4	0.0	0.1	0.0	0.1	0.0	1.0
CycCrashes_LB	0.0	0.0	0.1	0.0	0.0	0.0	0.0	1.0
CycCrashes_HA	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	1.0
PedCrashes_NANB	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0
PedCrashes_NDNF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	1.0
3arm	-0.1	-0.2	-0.1	0.1	0.0	0.0	-0.1	0.0	0.0	-0.1	-0.1	1.0
Highspeed	0.2	0.0	0.1	0.1	0.3	0.1	0.1	-0.1	0.0	-0.1	-0.1	0.0	1.0
Coord_allday	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.1	-0.1	0.0	1.0
Dist_upstrm_int	0.0	0.0	0.1	0.0	0.1	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.1	-0.2	1.0
Phasin_std	-0.1	0.0	0.0	0.0	-0.2	0.0	-0.1	0.1	0.0	0.0	0.1	0.0	-0.3	0.1	-0.1	1.0
Phasin_split	-0.1	-0.1	-0.1	0.0	0.0	0.1	0.0	0.0	0.0	-0.1	-0.1	0.2	0.0	-0.1	0.0	-0.4	1.0
Phasin_cmb	0.2	0.0	0.0	0.0	0.2	0.0	0.0	-0.1	0.0	0.0	0.0	-0.1	0.3	-0.1	0.1	-0.7	-0.4	1.0
RT_present	0.0	0.1	-0.1	-0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.4	0.1	0.0	0.0	-0.3	0.1	0.2	1.0
RT_fprot	0.0	-0.1	-0.1	0.0	0.2	0.1	0.1	-0.1	0.0	0.0	-0.1	0.1	0.3	-0.1	0.1	-0.9	0.5	0.5	0.3	1.0
RT_pprot	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	-0.1	-0.1	0.0	0.0	-0.2	-0.1	0.3	0.1	-0.2	1.0
RT_filter	-0.1	0.1	0.0	-0.1	-0.2	-0.1	-0.1	0.1	0.1	0.0	0.1	-0.3	-0.2	0.2	-0.1	0.8	-0.3	-0.6	0.2	-0.7	-0.2	1.0
Ped_present	0.0	0.1	0.0	-0.2	0.0	0.0	0.0	-0.1	0.0	0.1	0.0	-0.4	0.0	0.1	0.0	-0.1	0.0	0.0	0.5	0.1	0.0	0.2	1.0
LT_phase	0.1	0.1	0.1	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	-0.4	0.0	0.0	0.0	0.1	0.1	-0.2	0.0	-0.1	0.0	0.1	0.0	1.0
Barnes_dance	-0.1	-0.1	-0.1	0.0	-0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.0	-0.1	-0.1	-0.1	0.1	0.0	-0.1	0.0	-0.1	0.0	0.1	0.1	0.0	1.0
Num_app_lanes	0.3	0.1	0.1	0.1	0.4	0.1	0.1	0.0	0.0	0.1	0.0	-0.2	0.3	0.1	0.0	-0.3	-0.3	0.5	0.1	0.3	0.0	-0.2	0.1	-0.1	0.0	1.0
Num_thr_lanes	0.3	0.1	0.1	0.1	0.3	0.1	0.1	0.0	0.0	0.1	-0.1	-0.2	0.3	0.1	0.0	-0.1	-0.4	0.4	-0.1	0.1	-0.1	-0.1	-0.1	-0.1	-0.1	0.7	1.0
App_width	0.3	0.1	0.1	0.1	0.4	0.1	0.1	0.0	0.0	0.0	-0.1	-0.2	0.4	0.1	0.0	-0.2	-0.3	0.4	0.1	0.2	0.0	-0.2	0.1	-0.1	0.0	0.9	0.7	1.0
Med_island	0.2	0.0	0.1	0.1	0.2	0.1	0.1	-0.1	0.0	0.0	0.0	0.0	0.4	0.0	0.1	-0.2	-0.1	0.3	0.0	0.2	-0.1	-0.2	-0.1	-0.1	-0.1	0.5	0.4	0.5	1.0
Int_depth	0.2	0.1	0.1	0.0	0.3	0.0	0.1	0.0	0.0	0.0	-0.1	-0.2	0.4	-0.1	0.0	-0.3	0.0	0.3	0.1	0.4	-0.1	-0.3	0.1	0.1	-0.1	0.3	0.3	0.3	0.3	1.0
FLT	0.2	0.0	0.1	0.1	0.2	0.1	0.1	0.0	-0.1	-0.1	-0.1	-0.1	0.4	0.0	0.1	-0.3	0.1	0.2	0.0	0.4	-0.1	-0.4	-0.1	0.1	-0.1	0.2	0.1	0.2	0.3	0.3	1.0
Shared_LT	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	-0.2	0.0	-0.1	0.0	0.0	-0.1	0.0	0.1	-0.1	0.1	0.1	0.1	0.2	-0.1	-0.1	0.2	-0.1	-0.2	0.1	-0.3	1.0
Shared_RT	-0.1	0.0	-0.1	-0.1	-0.2	0.0	0.0	0.0	0.0	0.0	0.0	-0.2	-0.1	0.0	-0.1	0.3	0.1	-0.4	0.2	-0.2	-0.1	0.4	0.1	0.1	0.1	-0.4	-0.1	-0.4	-0.3	-0.1	-0.2	0.0	1.0
Shared_turns	-0.1	0.0	0.0	-0.1	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	-0.3	-0.1	0.0	0.0	0.2	0.0	-0.2	0.2	-0.2	0.0	0.3	0.1	0.2	0.0	-0.3	0.0	-0.3	-0.3	0.0	-0.3	0.7	0.5	1.0
Num_exit_lanes	0.3	0.1	0.1	0.1	0.3	0.1	0.1	-0.1	-0.1	0.1	-0.1	0.0	0.3	0.1	0.0	-0.2	-0.2	0.4	-0.1	0.2	0.0	-0.3	-0.1	-0.1	0.0	0.7	0.7	0.7	0.4	0.3	0.3	0.0	-0.3	-0.2	1.0
Dist_merge	-0.1	-0.1	0.0	0.0	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	-0.1	0.0	-0.1	0.1	0.0	-0.1	0.0	-0.1	0.0	0.1	0.0	0.0	0.1	-0.2	-0.2	-0.2	-0.1	-0.2	-0.1	-0.1	0.1	0.0	-0.3	1.0
(1+Len_Rtbay)	0.0	0.1	-0.1	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.0	-0.2	0.1	0.0	-0.1	-0.2	0.2	0.0	0.4	0.3	-0.1	0.0	0.2	0.0	0.0	0.0	-0.1	0.0	0.1	0.1	0.1	-0.1	0.4	0.1	0.0	0.1	1.0
(1+Offset_Rtbay)	0.0	0.1	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	-0.4	0.1	0.0	-0.1	0.1	0.0	-0.1	0.2	-0.1	-0.1	0.2	0.2	0.1	0.0	0.0	0.1	0.0	0.1	0.1	0.0	0.1	0.4	0.3	0.0	0.0	0.3	1.0
FLT_ped	0.2	0.0	0.0	0.0	0.3	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.0	0.1	-0.1	0.0	0.1	0.0	0.2	-0.1	-0.2	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.5	-0.1	-0.1	-0.1	0.2	-0.1	0.1	-0.1	1.0
Cycle_fac	0.0	0.0	0.1	0.0	-0.1	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	-0.1	0.0	0.1	0.0	-0.1	0.1	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.1	0.0	0.1	0.1	0.0	0.0	-0.1	-0.2	-0.1	-0.1	0.0	-0.1	-0.1	0.0	1.0
Cyc_app	0.0	0.0	0.1	0.0	-0.1	0.0	0.0	0.1	0.0	-0.1	0.0	0.0	-0.2	0.0	0.1	0.2	-0.1	-0.1	0.0	-0.2	0.0	0.2	0.0	0.0	-0.1	0.0	-0.1	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.1	-0.2	0.0	-0.1	-0.1	-0.1	0.6	1.0
Cyc_storage	0.0	0.0	0.1	0.0	-0.1	0.0	0.0	0.1	0.1	0.0	0.0	0.0	-0.1	0.0	0.1	0.2	-0.1	-0.1	0.0	-0.2	0.0	0.2	0.0	0.0	-0.1	0.0	-0.1	0.0	0.0	-0.1	-0.1	-0.1	-0.1	-0.1	-0.2	0.0	-0.1	-0.1	-0.1	0.5	0.8	1.0
Dist_parking	0.1	-0.1	0.1	0.0	0.2	0.1	0.1	0.0	-0.1	0.0	-0.1	0.1	0.3	-0.1	0.1	-0.3	0.0	0.3	-0.1	0.3	0.0	-0.4	-0.1	0.0	0.0	0.3	0.3	0.3	0.3	0.3	0.3	0.0	-0.2	-0.2	0.4	-0.1	0.0	-0.1	0.2	0.0	-0.2	-0.2	1.0
App_bus	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.0	-0.1	0.0	-0.1	0.0	0.0	0.0	-0.1	0.0	0.1	0.0	0.0	0.0	0.3	0.0	-0.1	0.0	-0.1	-0.1	-0.1	-0.1	0.0	-0.1	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.0	-0.1	1.0
Exit_bus	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.0	-0.1	0.0	0.0	0.1	-0.1	0.0	0.0	-0.1	0.0	0.1	0.0	0.0	0.2	0.0	-0.1	0.0	-0.2	-0.2	-0.1	-0.1	0.0	0.0	-0.1	0.0	-0.1	-0.1	0.0	0.0	0.1	0.1	-0.1	0.5	1.0
LU_Res	0.0	0.0	0.1	0.0	0.0	0.0	-0.1	0.0	0.0	-0.1	-0.1	0.1	0.0	-0.1	0.2	0.0	0.0	0.0	0.0	-0.1	0.0	0.1	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.1	-0.1	0.1	-0.1	0.0	-0.1	-0.1	-0.1	-0.1	0.0	0.2	0.2	0.1	0.0	-0.1	0.0	1.0
LU_Comm	-0.1	0.0	-0.1	0.0	-0.1	0.0	0.1	0.0	0.0	0.2	0.1	-0.1	-0.2	0.1	-0.2	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0	-0.1	0.0	-0.1	0.1	0.0	0.0	0.1	0.1	0.1	0.0	-0.1	-0.1	-0.1	0.0	0.1	0.1	-0.9	1.0
LU_Ind	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	-0.1	-0.1	0.1	0.2	-0.1	0.0	-0.1	0.0	0.1	0.0	0.1	0.1	-0.1	0.0	0.0	0.0	0.1	0.0	0.1	0.1	0.1	0.1	0.0	-0.1	0.0	0.1	0.0	0.1	0.0	0.0	-0.1	-0.1	-0.1	0.1	-0.1	-0.1	-0.2	-0.3	1.0
LU_Rural	0.1	0.0	0.1	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.0	-0.1	-0.1	0.0	1.0
Pole_low	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	-0.1	0.0	0.0	0.0	0.1	0.0	0.0	0.2	-0.1	0.0	-0.1	-0.1	-0.1	-0.1	0.0	0.1	0.1	-0.1	0.1	0.0	0.0	0.0	0.0	-0.1	-0.1	0.0	0.1	0.1	-0.1	0.1	0.0	0.1	1.0
Mast_arm	0.2	0.0	0.1	0.1	0.2	0.1	0.1	0.0	0.0	0.0	-0.1	0.0	0.3	0.0	0.0	-0.3	-0.2	0.4	-0.1	0.3	0.0	-0.4	-0.1	-0.1	0.0	0.4	0.4	0.4	0.2	0.3	0.3	0.0	-0.3	-0.2	0.4	-0.2	-0.1	-0.1	0.1	0.0	-0.1	-0.1	0.4	0.0	0.0	0.0	-0.1	0.1	0.0	-0.1	1.0
Num_signals	0.2	0.0	0.1	0.1	0.3	0.1	0.1	-0.1	0.0	0.0	-0.1	-0.1	0.3	0.0	0.1	-0.3	-0.2	0.5	-0.1	0.3	0.0	-0.4	0.0	-0.1	0.0	0.6	0.5	0.6	0.5	0.3	0.3	0.0	-0.4	-0.2	0.5	-0.2	0.0	0.0	0.1	0.1	0.0	0.0	0.4	0.0	0.0	0.0	0.0	0.1	0.1	-0.1	0.8	1.0
Lights_left	-0.1	0.0	-0.1	0.0	-0.1	0.0	0.0	-0.1	0.0	0.0	-0.1	0.1	-0.2	-0.1	0.0	-0.2	0.1	0.1	-0.1	0.1	0.2	-0.2	-0.1	0.0	0.1	0.0	-0.1	0.0	-0.1	0.1	-0.2	0.0	0.0	0.0	-0.1	0.0	-0.1	-0.1	-0.1	0.0	-0.1	-0.1	0.0	0.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0
Lights_right	0.3	0.0	0.1	0.0	0.3	0.1	0.0	0.0	0.0	0.0	0.0	-0.1	0.4	0.0	0.1	-0.2	-0.1	0.3	0.1	0.2	0.0	-0.1	0.0	0.0	-0.1	0.5	0.4	0.5	0.6	0.3	0.2	0.0	-0.2	-0.1	0.4	-0.2	0.0	0.0	0.2	0.0	0.1	0.1	0.2	-0.1	-0.1	0.1	-0.2	0.1	0.1	-0.1	0.1	0.4	-0.2	1.0
Lights_mastarm	0.2	0.0	0.0	0.1	0.2	0.1	0.1	0.0	0.0	0.0	-0.1	0.0	0.2	0.0	0.0	-0.4	-0.2	0.6	0.0	0.4	0.0	-0.5	0.0	-0.1	0.0	0.4	0.4	0.4	0.2	0.3	0.3	0.0	-0.3	-0.2	0.4	-0.2	0.0	-0.1	0.1	0.0	-0.2	-0.1	0.4	0.0	0.0	0.0	0.0	0.1	0.0	-0.1	0.9	0.7	0.1	0.0	1.0
Adv_det	0.0	0.0	0.0	0.0	0.0	0.0	-0.1	0.0	0.0	-0.1	0.0	0.2	0.0	0.0	0.1	-0.1	0.0	0.1	-0.1	0.0	0.2	-0.1	-0.1	-0.1	0.0	-0.1	0.0	0.0	0.1	0.0	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	-0.2	0.0	0.0	-0.1	-0.1	0.1	0.0	0.0	0.0	0.0	-0.1	0.2	0.0	0.1	0.0	0.0	-0.1	0.1	1.0
Green	0.0	0.0	0.1	0.1	0.0	0.0	-0.1	0.1	0.0	0.1	0.0	0.1	-0.1	0.3	-0.1	0.2	-0.3	0.0	-0.2	-0.2	0.0	0.1	-0.2	-0.1	0.0	0.3	0.4	0.2	0.1	-0.1	-0.1	0.0	-0.2	-0.1	0.3	0.1	-0.2	0.0	-0.1	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.1	-0.1	0.0	0.0	0.2	0.2	-0.1	0.0	0.1	0.0	1.0
Yellow	0.1	0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.1	0.0	0.1	-0.2	0.0	0.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.1	-0.1	0.0	-0.1	0.0	0.0	0.1	0.1	0.0	0.1	0.1	0.1	0.0	0.1	0.0	-0.1	0.1	0.0	0.1	0.0	0.0	0.1	0.0	0.0	0.0	0.1	-0.1	1.0
Red	0.1	0.0	0.0	0.0	0.2	0.0	0.0	0.1	0.0	0.0	-0.1	0.0	0.3	0.0	0.0	-0.1	-0.2	0.2	0.0	0.1	0.0	-0.1	-0.1	0.0	0.0	0.2	0.3	0.2	0.2	0.3	0.1	0.1	-0.1	0.0	0.2	-0.1	0.0	0.0	0.1	0.0	0.0	-0.1	0.2	0.0	-0.1	0.0	0.0	0.0	0.0	0.0	0.2	0.2	0.0	0.2	0.2	0.1	0.2	-0.3	1.0
App_tot_time	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.1	0.0	0.1	0.0	0.1	-0.1	0.3	-0.1	0.2	-0.3	0.0	-0.2	-0.2	0.0	0.1	-0.2	-0.1	0.0	0.3	0.4	0.2	0.1	-0.1	-0.1	0.0	-0.2	-0.1	0.3	0.1	-0.2	0.0	-0.1	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.1	-0.1	0.0	0.0	0.2	0.2	-0.1	0.0	0.1	0.1	1.0	-0.1	0.2	1.0
Num_ped_IDM	-0.1	0.0	-0.1	-0.1	-0.1	0.0	0.0	0.0	0.0	0.3	0.1	-0.2	-0.2	0.2	-0.2	0.2	-0.1	-0.1	0.0	-0.2	0.0	0.2	0.3	0.0	0.1	0.0	0.0	-0.1	-0.2	-0.2	-0.2	-0.1	0.2	0.1	0.0	0.1	0.1	0.1	-0.1	-0.1	-0.1	-0.1	-0.2	0.1	0.1	-0.3	0.4	-0.2	-0.1	0.2	-0.1	-0.1	0.0	-0.1	-0.1	-0.1	0.1	0.0	-0.1	0.1	1.0
Cyctime_day	0.1	0.0	0.0	0.1	0.1	0.0	0.1	0.0	-0.1	0.1	-0.1	0.0	0.1	0.0	-0.1	-0.4	0.2	0.3	0.0	0.5	-0.1	-0.5	0.0	0.0	0.1	0.3	0.2	0.2	0.2	0.4	0.3	-0.1	-0.2	-0.1	0.3	0.0	0.1	0.0	0.1	-0.1	-0.2	-0.3	0.3	0.1	0.0	-0.1	0.1	0.0	-0.1	0.0	0.3	0.3	0.1	0.0	0.4	0.0	0.2	0.0	0.2	0.2	0.0	1.0
%green	0.0	0.0	0.1	0.1	-0.1	0.0	-0.1	0.1	0.0	0.1	0.1	0.1	-0.2	0.2	-0.1	0.5	-0.4	-0.2	-0.2	-0.5	0.0	0.4	-0.1	-0.1	-0.1	0.1	0.2	0.1	0.0	-0.3	-0.3	0.0	0.0	0.0	0.1	0.1	-0.2	0.0	-0.1	0.0	0.2	0.2	-0.1	0.0	0.1	0.0	0.0	-0.1	0.0	0.0	0.0	0.1	-0.2	0.0	-0.1	0.0	0.8	-0.1	0.1	0.8	0.1	-0.3	1.0
Deg_sat	0.1	-0.1	0.0	0.1	0.1	0.1	0.1	0.0	0.0	-0.1	-0.1	0.1	0.1	-0.1	0.0	-0.4	0.2	0.2	0.0	0.4	0.0	-0.4	-0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.4	-0.1	-0.1	-0.1	0.1	0.0	0.1	-0.1	0.3	0.0	-0.2	-0.1	0.2	0.0	-0.1	0.0	0.0	0.1	0.0	-0.1	0.2	0.1	0.0	0.0	0.2	0.1	-0.2	0.1	0.0	-0.2	-0.2	0.4	-0.4	1.0
q_app	0.3	0.0	0.1	0.2	0.3	0.1	0.1	0.0	0.0	0.0	-0.1	0.1	0.2	0.1	0.0	-0.3	-0.2	0.5	-0.1	0.3	0.0	-0.4	-0.2	-0.1	-0.1	0.6	0.6	0.6	0.4	0.2	0.2	-0.1	-0.3	-0.2	0.6	-0.1	0.0	-0.1	0.1	0.0	-0.1	0.0	0.4	0.0	0.0	0.0	0.0	0.1	0.0	-0.1	0.5	0.5	0.0	0.3	0.5	0.1	0.4	0.0	0.2	0.4	-0.1	0.3	0.2	0.4	1.0

Appendix D: New Zealand crash collision diagrams

Figure D.1 NZTA crash coding (NZTA 2004)



Appendix E: Crash prediction modelling methodology

The process begins by listing and then grouping the key crash types. The critical variables expected to influence each crash type are then identified (the key variables identified have been covered in earlier sections) and collected with a view to determining trends and treatment types used.

Once a functional model form has been selected (in this case the power model), generalised linear models are then developed for each crash type using either a negative binomial or Poisson distribution error structure. Generalised linear models were first introduced to road crash studies by Maycock and Hall (1984), and extensively developed by others (eg Hauer et al 1989). These modelling techniques were further developed in the New Zealand context for motor vehicle-only crashes by Turner (1995).

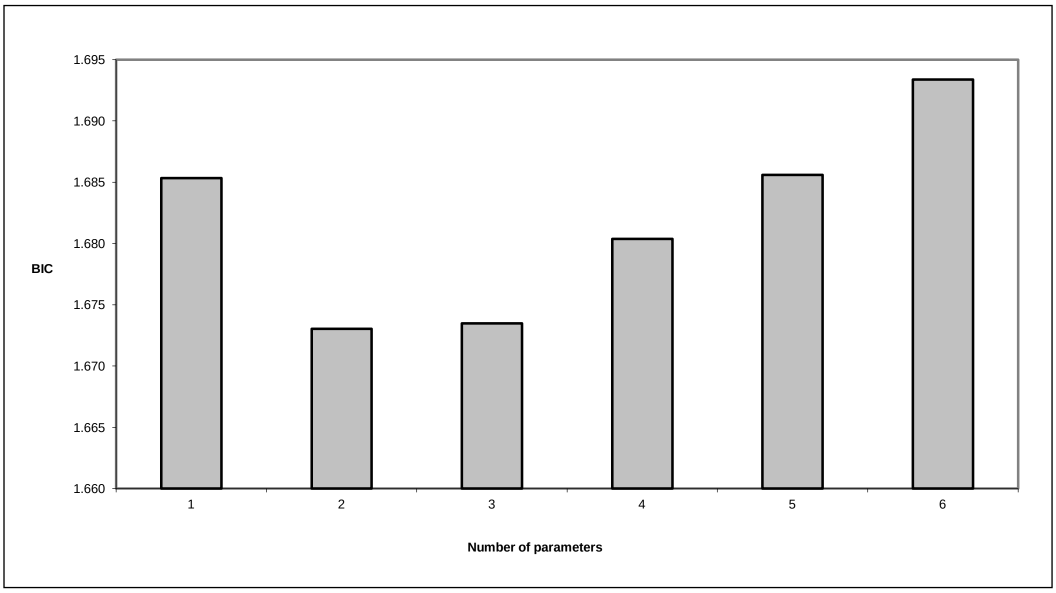
Software has been developed in Minitab in order to fit such models (ie to estimate the model coefficients); however, this can also be readily done in many commercial packages, such as GENSTAT, LIMDEP or SAS.

Given the large number of possible variables for inclusion in the models, a criterion to decide when the addition of a new variable is worthwhile is needed; this balances the inevitable increase in the maximum likelihood (*L)* of the data against the addition of a new variable (where *p* is the number of variables included in the model and *n* is the total number of observations in the sample set). We chose to use the popular Bayesian Information Criterion (BIC). Stop adding variables when the BIC reaches its lowest point. The BIC is given by:

$$BIC = (-2\ln(L) + p\ln(n))/n \qquad \text{(Equation E.1)}$$

The model with the lowest BIC is typically the preferred model form. Addition of a new variable to a model always provides an improved fit, though this may be slight and therefore not reduce the BIC. Figure E.1 illustrates where the BIC indicates that the parsimonious number of parameters is two. However, if an analyst considers that a three-parameter model includes an important variable not contained in the two-parameter model, then they could justifiably select the model with three parameters, depending on the outcome of goodness-of-fit testing (see the next section).

Figure E.1 Bayesian Information Criterion (BIC)



Modelling every possible combination of variables to determine which has the lowest BIC would be time-consuming and inefficient. Instead, the process used involves all non-flow variables being modelled with the main motor-vehicle and cyclists-flow variables. The variables in the resulting models that maximise the log-likelihood (and therefore minimise the BIC) are then added together into a new model with more variables, and the BIC is tested. This is done for a number of combinations of variables (but not all combinations), as often the variables can be correlated, meaning that the 'best' two variables may not result in a better model.

E.1 Goodness of fit

While the BIC provides us with a model, the model may still not fit the data well. The usual methods for testing goodness of fit of generalised linear models involve the scaled deviance G^2 (twice the logarithm of the ratio of the likelihood of the data under the larger model, compared with that under the smaller model) or Pearson's X^2 (the sum of squares of the standardised observations). These did not work in our situation because of the 'low mean value' problem; our models were being fitted to data with very low mean averages. This difficulty was first pointed out by Maycock and Hall (1984).

In Wood (2002) a 'grouping' method was developed, which overcomes the low mean value problem. The central idea is that sites are clustered, and then aggregate data from the clusters is used to ensure that a grouped scaled deviance follows a chi-square distribution if the model fits well. Evidence of goodness of fit is provided by a p-value. If this value is less than 0.05, say, there is evidence at the 5% level that the model does not fit well. Software has been written in the form of Minitab macros in order to run this procedure.

The goodness of fit is often calculated for a number of the better models as indicated by BIC. This is because although the best model as indicated by the BIC may have the crash rate following the modelled negative binomial distribution more closely at each combination of variables, there may be some combination where the model fits poorly (ie the true crash rate is very different from our prediction). The goodness of fit would indicate that this is poorly fitting model. As the goodness of fit is the best overall arbiter of the worth of the model, a model with a poorer BIC (but better goodness of fit) may be selected as the preferred model.

E.2 Model interpretation

Once models have been developed, in some simple cases the relationship between crashes and predictor variables can be interpreted. Caution should always be exercised when interpreting relationships, as two or more variables can be highly correlated. However, the modelling process described in the previous sections usually means that variables in the preferred models are not highly correlated, because the method acknowledges that adding a variable correlated to those already in an existing model does not improve the fit of the model compared with the addition of important non-correlated variables. Likewise, functional forms that deviate from a power function are also difficult to interpret. In these situations it is always best to plot the relationship.

In models with a power-function form, where the variables are not correlated, an assessment of the relationship can be carried out. For a typical model with a power-function form and two continuous variables (such as flows or speeds), the equation takes the following form:

$$A = b_0 x_1^{b_1} x_2^{b_2} \qquad \text{(Equation E.2)}$$

Where:

A is the annual mean number of crashes

x_1 , x_2 are continuous flow or non-flow variables

b_0, b_1 and b_2 are model parameters.

In this model form, the parameter b_0 acts as a constant multiplicative value. If the number of reported injury crashes is not dependent on the values of the two predictor variables (x_1 and x_2), then the model parameters b_1 and b_2 are zero. In this situation, the value of b_0 is equal to the mean number of crashes. The value of the parameters b_1 and b_2 indicates the relationship that a particular predictor variable has (over its flow range) with crash occurrence. There are five types of relationship for this model form, as presented in figure E.2 and discussed in table E.1.

Figure E.2 Relationship between crashes and predictor variable x for different model exponents (b_1)

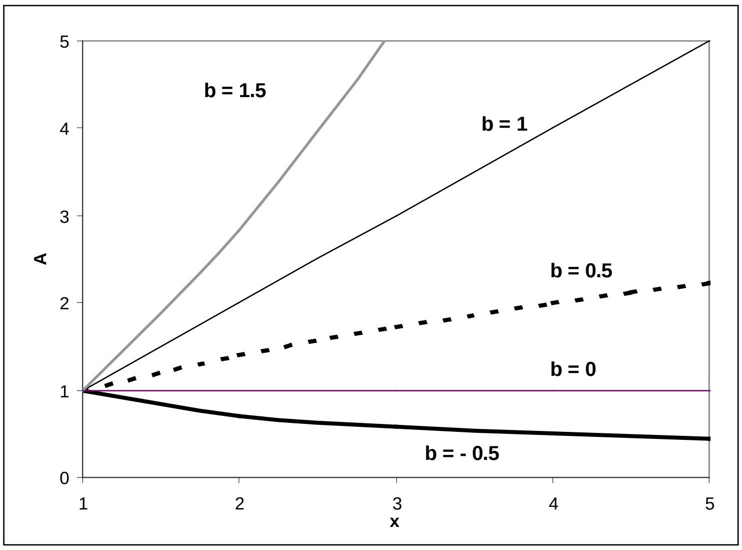


Table E.1 Relationship between predictor variable and crash rate

Value of exponent	Relationship with crash rate
$b_i > 1$	For increasing values of the variable, the number of crashes will increase, at an increasing rate.
$b_i = 1$	For increasing values of the variable, the number of crashes will increase, at a constant (or linear) rate.
$0 < b_i < 1$	For increasing values of the variable, the number of crashes will increase, at a decreasing rate.
$b_i = 0$	There will be no change in the number of crashes with increasing values of the variable.
$b_i < 0$	For increasing values of the variable, the number of crashes will decrease.

Generally, models of this form have exponents between $b_i = 0$ and $b_i = 1$, with most flow variables having an exponent close to 0.5, ie the square root of flow. In some situations, however, parameters have a value outside this range. This is particularly true of cycle volumes, where the parameter is often well below 0.5, normally as a result of the 'safety in numbers' effect that has been observed in a number of studies for cycle–vehicle crashes.

In the case of models including a covariate (here, discrete variables with a small number of alternatives) a multiplier for different values of the variable is produced, and it is easy to interpret the relationship. This factor indicates how much higher (or lower) the number of crashes is if the feature is present. A factor of 1 indicates no effect on crash occurrence.