CRASH PREDICTION MODELLING AT INTERSECTIONS IN NEW ZEALAND 1990 TO 2009

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ABSTRACT

A large number of crash prediction models have been developed in New Zealand, for different road elements and for different speed limits. These models provide insight into crash causing mechanisms, which can in turn assist engineers in diagnosing safety problems. In conjunction with other road safety research (e.g. results of `before and after' studies) they can also be used to predict the change in crashes that might result from an engineering improvement, whether good or bad.

The crash modeling methods used in New Zealand are based on best practice overseas, from the UK, Canada and the USA, with some local enhancements. The research to date has produced a number of interesting and thought-provoking outcomes including the `safety-in-numbers' effect for cyclists and pedestrians and that reducing visibility can lead to safety gains at roundabouts. This paper profiles the models that have been developed for low and high speed traffic signals, roundabouts and priority intersections in New Zealand.

In addition to presenting the crash models and the modeling methods, the paper will show how the models are used to compare various forms of control at an intersection. It will highlight the importance of using the models within the prescribed flow ranges. The models are less accurate when used to extrapolate to traffic volumes that are not typical for the intersection type, for example, for low volume traffic signals and high volume priority intersections.

INTRODUCTION

Since the late 1980s both New Zealand and Australia have introduced a number of ‘engineering’ processes and measures to reduce crash occurrence and trauma. This has included the widespread adoption of road safety strategies and targets (at national/federal, state, regional and local levels), crash reduction studies, road safety auditing, emphasis on low cost safety improvements, and more recently, safety management systems. In New Zealand this has been a major contributor to the drop in the fatality rates per 100,000 population from 23.8 in 1987 (or 795 fatalities) to 7.7 in 2008 (or 331 fatalities).

As high crash locations are upgraded and crashes become more dispersed and more varied, it becomes increasingly difficult to diagnosis the safety problems at particular locations. While analysis of crash trends and crash occurrence over routes and areas is useful, it does not provide safety specialists with a relationship between ‘engineering’ factors and crashes. Since the early 1990s New Zealand researchers have been developing crash prediction models, following the lead of researchers such as Hauer (1989) in Canada and Maycock and Hall (1984) in the UK. Crash prediction models are regression models that relate crashes to a number of traffic operation characteristics and road features. Most of the early work focused on the relationships between crashes and traffic volume. Over more recent years the research has extended to look at the non-flow variables, including visibility, speed and road layout.

This paper profiles some of the research undertaken in New Zealand on crash models at urban and rural intersections. This includes the work by Jackett (1992 & 1993), Wood (1992 and 1993), Turner et al. (2000) and Harper and Dunn (2005). These methods, using multiple predictors, enabled the typically non-linear relationship between a number of ‘engineering’ factors and crashes to be quantified. As with most crash modeling research, care must be applied when using the research results due to correlations between variables. For details on the modeling methods used by Turner and Wood, refer to Turner (1995), Wood (2002 and 2005) and Wood and Turner (2008).

Crash prediction models are often used to compare various forms of intersection control. The models can be used to assess the total change in crashes and also the change in crash types. What the models do not predict is the change in crash severity. In New Zealand the change in crash severity for the different crash types and in turn intersection types can be taken into account using crash costs. So by multiplying the predicted number of injuries by the crash cost to calculate the social cost it is possible to compare intersection types. When comparing intersection types, particularly over a long timeframe, it is important to consider the flow range over which the models apply. For example, there are few signalised intersections that have low traffic volumes, and the models do not perform well at low volumes.

TRAFFIC SIGNALS MODELS

A number of studies have focused on crash prediction models for traffic signals. In addition to flow-only models for all vehicle types in a slow speed urban environment, models have been developed for various crash types, including those involving pedestrians and cyclists and for high speed signals. Models with non-flow variables, including visibility, signal phasing and intersection layout, including one-way streets, have also been developed. The more important studies are outlined in this section.

Low Speed Traffic Signals (50kph and 60kph on all approaches)

Turner (2000) developed flow-only crash prediction models for signalised cross-roads and T-junctions in New Zealand, for all major crash types. The models were based on a sample set of 109 cross-roads (or 436 approaches) and 30 T-junctions. Figure 3 shows the conflicting movements that were used in the models for each approach.

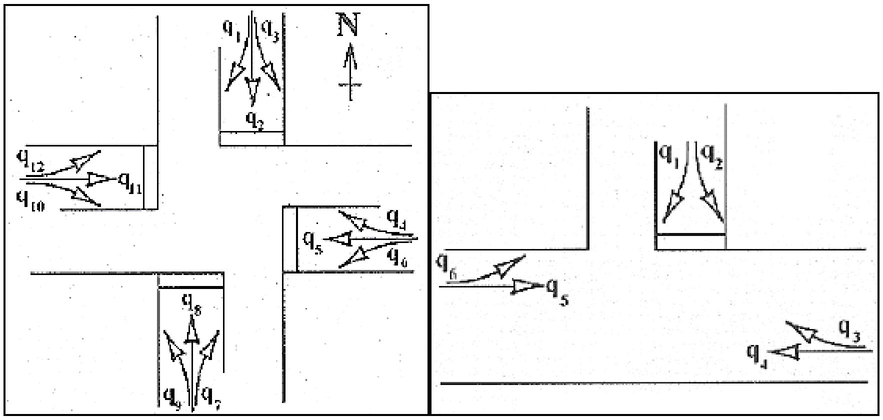

Figure 1: Conflicting and approach flow types (Cross-roads and T-junctions)

The four major all-vehicle crash types at cross-roads and T-junctions, and the corresponding models are shown in Table 4 and 5 respectively.

Table 1: Signalised cross-road crash prediction models

Crash Type	NZ Crash Codes	Equation (crashes per approach)
Crossing (No Turns)	HA	$A = 1.57\times10^{-4} \times q_2^{0.36} \times q_{11}^{0.38}$
Right Turn Against	LB	$A = 9.57\times10^{-5} \times q_2^{0.49} \times q_7^{0.42}$
Rear-end	FA to FE	$A = 1.66\times10^{-6} \times Q_e^{1.07}$
Loss-of –control	C & D	$A = 2.96\times10^{-6} \times Q_e^{0.95}$
Others	Rest of codes	$A = 1.26\times10^{-} \times Q_e^{0.46}$

Table 2: Signalised T-junction crash prediction models

Crash Type	NZ Crash Codes	Equation (crashes per approach)
Right Turn Against	LB	$A = 1.08\times10^{-1} \times q_5^{-0.38} \times q_3^{0.56}$
Rear-end	FA to FD	$A = 7.66\times10^{-8} \times Q_e^{1.45}$
Crossing (Vehicle Turning)	JA	$A =2.67\times10^{-2}\times q_5^{-0.30}\times q_1^{0.49}$
Loss-of –control	C & D	$A = 1.91\times10^{-3} \times Q_e^{0.17}$
Others	All other codes	$A = 1.69\times10^{-2} \times Q_e^{0.15}$

The large negative parameters in the ‘LB’ and ‘JA’ models indicate that intersections with higher flows have fewer crashes. It is speculated that at high traffic flows the installation of right turn bays and exclusive right turn phases reduces crash occurrence. This issue was investigated for right-turn-against crashes in the study by Turner and Roozenburg (2004).

Turner and Roozenburg (2004) considered a number of non-flow variables for right turn crashes, including visibility, number of opposing lanes and signal phasing type. The flow-only model form was modified to allow non-flow variables to be added. The form of the extended crash prediction model is as follows:

$$A_T = b_o \text{ x } q_2^{\ b1} \text{ x } q_7^{\ b2} \text{ x } c^{\ b3} \text{ x } \varphi$$

where:

- $\mathbf{b_o}$ is a model constant
- $\mathbf{q_2}$ is the daily through traffic flow opposing right turning traffic
- $\mathbf{b_1}$ is a model parameter, applied as the power of $\mathbf{q_2}$
- $\mathbf{q_7}$ is the daily right turning traffic flow
- $\mathbf{b_2}$ is a model parameter, applied as the power of $\mathbf{q_7}$
- **c** is a continuous non-flow or non-conflicting flow variable – such as visibility (V) or intersection depth (I), both measured in metres
- $\mathbf{b_3}$ is a model parameter, applied as the power of **C**
- **φ** is a multiplicative model parameter for a discrete non-flow variable, which can take on one of two values.

In addition to the three important features of visibility, number of opposing lanes and signal phasing type, other variables, such as opposing right turning flow and intersection depth (distance between opposing limit, or hold, lines) were also investigated, but were found to be not significant. Visibility and intersection depth are continuous variables and are added as 'c' variables. Number of opposing lanes and signal phasing are discrete variables and are added as a multiplicative parameter **φ**.

The first variable to be added to the model was visibility V. The model form is:

$$A_T = 1.29{\times}10^{-5} \times q_2^{\ 0.729.} \times q_7^{\ 439} \times V^{\ -0.111}$$

The negative exponent of visibility indicates that the number of crashes decreases as the visibility increases. Although this model does predict a change in the number of crashes with varying visibility, an initial data analysis indicated that there was a stronger relationship between the number of crashes and the difference between the observed and recommended visibility ($V - V_R$, V_R is specified in the Austroads Guides to Traffic Engineering). The resulting model was as follows.

$$A_T = 5.97{\times}10^{-5} \times q_2^{\ 0.664} \times q_7^{\ 0.446} \times (V - V_R + 100)^{-0.329}$$

The second non-flow variable to be included in the crash prediction model was the number of opposing through lanes (φ_L). The resulting model is as follows:

$$A_T = 1.47{\times}10^{-4} \times q_2^{\ 0.435} \times q_7^{\ 0.389} \times \varphi_L$$

where $\varphi_L = 0.714$ for a single opposing through lane and $\varphi_L = 1.401$ for multiple opposing through lanes. The factors for φ_L indicate that where the opposing through and right turning traffic volumes are held constant, then the predicted crash rate for an intersection with two or more opposing through lanes would be nearly twice that for the same flows as a single opposing lane.

Another non-flow variable that was investigated was that of the signal phasing with the model depending on whether the right turn was a filter turn or a fully or partially protected turn (green arrow) (φ_P). The resulting model is:

$$A_T = 6.68\times10^{-6} \times q_2^{\,0.735} \times q_7^{\,0.457} \times \varphi_P$$

where $\varphi_P = 1.048$ for a fully filtered turn and $\varphi_P = 0.954$ for fully or partially protected turns. The factors for φ_P indicate that the predicted number of crashes is higher, but only slightly so for approaches with a fully filtered turn, than for approaches where there is at least a partially protected signal phase. The model indicates that a site with a fully filtered turn on average would have a crash rate 10% higher than one with a partially or fully protected turn.

Turner et al. (2005) developed crash prediction models for crashes involving pedestrians and cyclists. Pedestrian and cycle crashes are fairly common at most low speed urban traffic signals. Models were developed for the two major cycle versus motor-vehicle crash types at signalised cross-roads. The first, a 'same direction' model, predicts crashes on a single approach between cyclists either colliding with a stationary vehicle or moving motor-vehicle, traveling in the same direction.

Cycle and Pedestrian Models at Low Speed Traffic Signalised Intersections

The second model is for right-turn-against crashes where a cyclist is travelling straight through the intersection and collides with a motor vehicle turning right. These two models, along with the proportion of cycle crashes by type at signalised cross-roads are shown in Table 3. Cycle movements are coded in a similar manner to motor-vehicle movements. Entering flows, for example C_e, are the sum of all cycle entering flows, for example $c_1 + c_2 + c_3$. Figure 2 shows the movements graphically.

Table 3: Signalised cross-road cycle crash prediction equations

Crash Type	**Equation (crashes per approach)**	**Proportion of Cycle Crashes**
Same Direction	$A = 7.49\times10^{-4} \times Q_e^{0.29} \times C_e^{0.09}$	35%
Right Turn Against – Motor vehicle turning	$A = 4.41\times10^{-4} \times q_7^{0.34} \times c_2^{0.20}$	21%

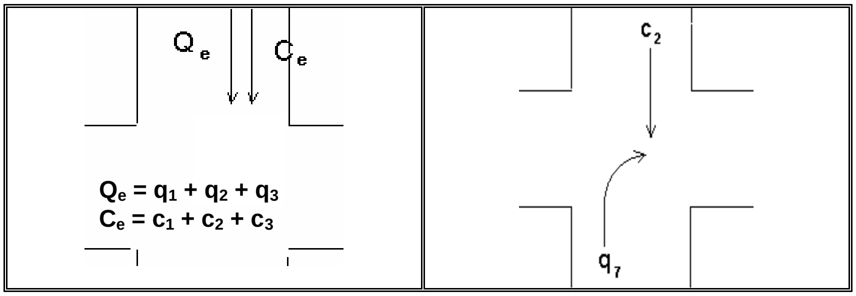

Figure 2: Cycle model variables

The small value of the exponent of cycle flows b_2 in Table 3 indicates a ‘safety in numbers’ effect where the crash rate per cyclist decreases substantially as the number of cyclists increase. For example, the increase in ‘right turn against’ crash rate when the cycle volume doubles from 200 to 400 cyclists per day, and the right turn volume is 1200 vehicles per day, is only 15% (0.014 compared to 0.016 crashes per year for this movement). There is a reduction in the individual risk per cyclist of 43% due to the increase in cyclists alone.

Models were developed for the two major pedestrian crash types at signalized cross-roads. The majority of crashes involving pedestrians, not just those at signalised cross-roads, occur where vehicles are traveling straight along the road and the pedestrian is crossing. These crashes represent 50% of pedestrian crashes at signalized cross-roads. The second major type of pedestrian crash at signalized cross-roads is where right turning vehicles collide with pedestrians crossing the side road. Table 4 present these two models. Figure 3 shows the pedestrian movements used graphically.

Table 4: Signalised cross-road pedestrian crash prediction equations

Crash Type	Equation (crashes per approach)	Proportion of Ped. Crashes
Crossing – vehicle intersecting	$A = 7.28\times10^{-6} \times Q^{0.63} \times P^{0.40}$	50%
Crossing – vehicle turning right	$A = 5.43\times10^{-5} \times q_4^{0.43} \times p_1^{0.51}$	36%

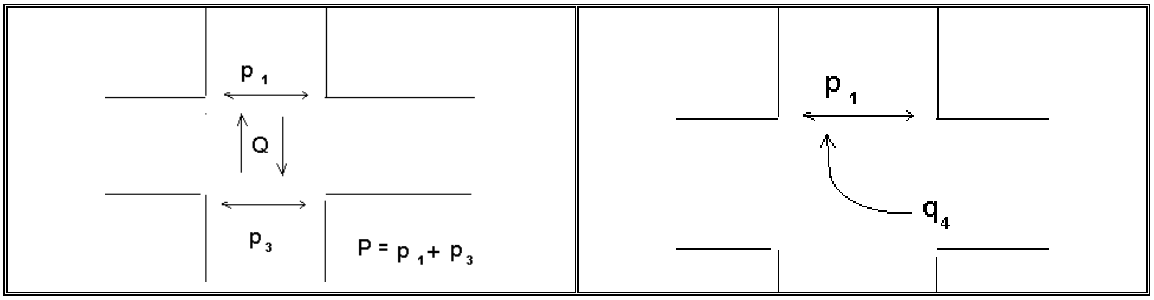

Figure 3: Pedestrian model variables

Unlike the cycle models, the ‘safety in numbers effect’ is not as pronounced, with exponents of flow being similar to those observed for motor vehicle flows, but there is still an effect.

High Speed Traffic Signals (80 kph and above on some or all approaches)

Crash prediction models for high speed signalized cross-roads and T-junctions were developed by Turner et al. (2006b) from high speed limit sites in New Zealand and from the State of Victoria, in Australia (Melbourne). The variable ϕ_{VIC}, a multiplication factor, was included in the models to show the differences between New Zealand and Victoria. The models for high speed cross-roads and T-junctions are shown in Tables 5 and 6. Q_{major} and Q_{minor} are the major (highest volume) and minor roads that intersect respectively.

Table 5: High-speed signalised crossroad crash prediction models

Crash Type	Equation (crashes per approach)	ϕ_{VIC}	Error Structure	Significant Model
Crossing	$A = 6.82 \times 10^{-5} \times q_2^{0.31} \times q_{11}^{0.35} \times \phi_{VIC}$	3.95	Poisson	Yes
Right-turn-against	$A = 2.17 \times 10^{-2} \times q_2^{0.20} \times \phi_{VIC}$	1.43	NB (K=0.9)*	Yes
Rear-end	$A = 4.16 \times 10^{-7} \times Q_e^{1.18} \times \phi_{VIC}$	9.91	NB (K=1.7)*	Yes
Loss of Control	$A = 5.75 \times 10^{-5} \times Q_e^{0.70} \times \phi_{VIC}$	1.15	NB (K=5.7)*	Yes
Others	$A = 1.04 \times 10^{-2} \times Q_e^{0.14} \times \phi_{VIC}$	4.44	NB (K=1.7)*	Yes

*K is the shape parameter for the negative binomial (NB) distribution.

Table 6: High-speed signalised T-junction crash prediction models

Crash Type	Equation (crashes per approach)	ϕ_{VIC}	Error Structure	Significant Model
Right-turn-against	$A = 8.29 \times 10^{-2} \times q_3^{0.26} \times q_5^{-0.15} \times \phi_{VIC}$	2.85	NB (K=2.3)*	Yes
Rear-end (Major Road)	$A = 2.28 \times 10^{-2} \times Q_{Major}^{0.29} \times \phi_{VIC}$	0.89	Poisson	Yes
Crossing (Vehicle turning)	$A = 3.18 \times 10^{-2} \times q_1^{0.12} \times \phi_{VIC}$	1.67	Poisson	Yes
Loss-of-control (Major Road)	$A = 5.77 \times 10^{-3} \times Q_{Major}^{0.32} \times \phi_{VIC}$	1.06	Poisson	Yes
Other (Major Road)	$A = 1.82 \times 10^{-3} \times Q_{Major}^{0.37} \times \phi_{VIC}$	2.81	Poisson	No
Other (Minor Road)	$A = 1.40 \times 10^{-3} \times Q_{Minor}^{0.41} \times \phi_{VIC}$	5.04	NB (K=8.0)*	Yes

The ‘Victorian’ factor/variable indicates that for most crash types the number of crashes at similar high-speed traffic signals in Victoria is higher than in New Zealand. It is unclear why the crash risk in Victoria is higher than New Zealand. It could be due to differences in reporting rates (the models provided are for reported injury crashes) or due to different road layout standards, although there is not a lot of difference in roading standards between the countries, or due to different driver behavior.

ROUNDABOUT MODELS

Low Speed Roundabouts (50kph or 60kph on all approaches)

Flow-only crash prediction models for 4-arm roundabout were first developed in New Zealand by Turner (2000). The models for all vehicles types are presented in Table 7. The circulating flow (Q_c) is the traffic to which the entering flow (Q_e) at each roundabout approach must give-way.

Table 7 Roundabout Crash Prediction Equations

Crash Type	NZ Crash Codes	Equation (crashes per approach)
Entering vs Circulating	HA, LB, JA, MB, KA &KB	$A = 4.46E\text{-}04 * Q_e^{0.42} * Q_c^{0.45}$
Rear-end	FA to FD	$A = 2.88E\text{-}06 * Q_e^{1.19}$
Loss-of–control	C & D	$A = 1.51E\text{-}03 * Q_e^{0.55}$
Others		$A = 1.14E\text{-}02 * Q_e^{0.26}$

Cycle versus motor-vehicle models were developed for roundabouts by Turner et al. (2006a). These models are shown in Table 8. The model is based solely on the volume of motor vehicles and cycles using each facility. Again the ‘safety-in-numbers’ effect is evident by the lower parameter for the cycle volume.

Table 8: Crash prediction models for Cyclists Crashes (Flow-only)

Crash Type	Equation (crashes per approach)	Error Structure	Significant Model
Roundabouts			
Entering versus circulating (HA, LB, KB & KA)	$A = 2.40 \times 10^{-5} \times Q_e^{0.79} \times C_c^{0.32}$	NB (K=0.8)*	Yes

*K is the shape parameter for the negative binomial (NB) distribution.

where

A	Annual number of crashes for an approach or mid-block section
Q_e	Motor vehicle flow entering the intersection for an approach
C_c	Circulating cycle flow at an approach

Harper and Dunn (2005) details research on the development of crash prediction models for roundabouts, including geometric variables. The models were developed using a dataset of 95 urban roundabouts throughout New Zealand. A number of the roundabouts used in this study were common to the study undertaken by Turner (2000). Harper and Dunn (2005) developed models for individual crash types and ‘product of link’ crashes using similar crash types to those used by Turner (2000). They found that in most cases the inclusion of geometric variables improved the predictive accuracy of the models.

Scaled aerials were used to measure a number of geometric variables. The measurements were taken in respect to each approach. Harper and Dunn (2005) noted that sight distance could not be accurately calculated from aerial photos and therefore excluded this from their analysis. Also, deflection was excluded from the analysis as no apparent standard had been established for defining the deflection path. The geometric characteristics used in the study are illustrated in Figure 4.

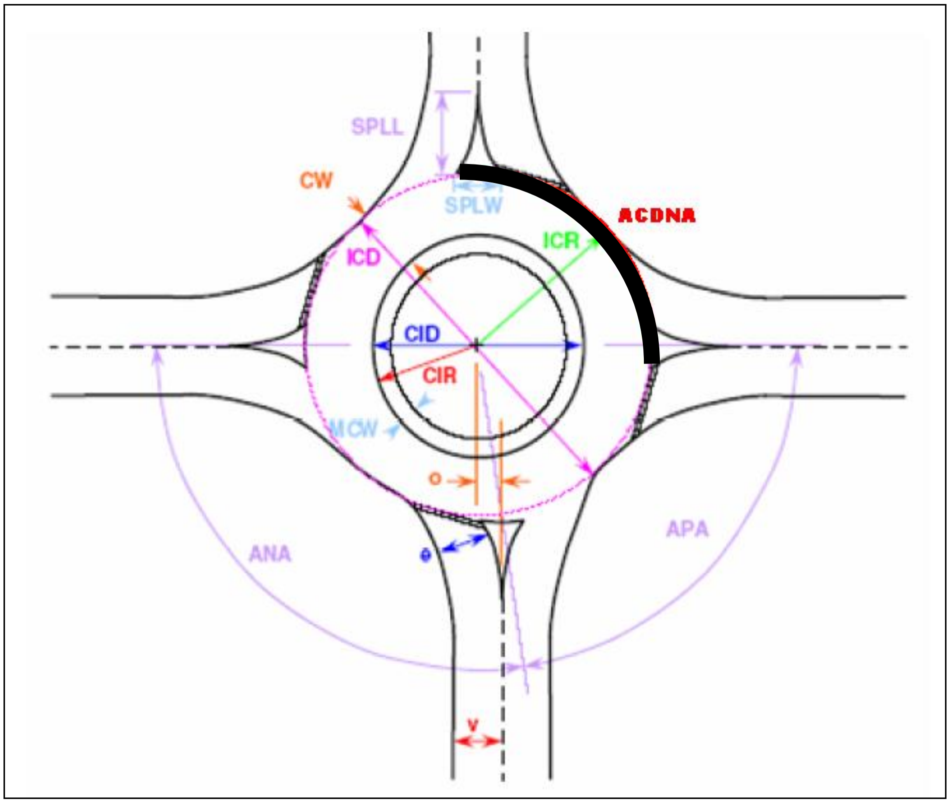

Figure 4 Basic Geometric Measurement Definition Plan (Harper and Dunn, 2005)

The general model form used for the (conflicting) flow models is specified as follows.

$$A = b_0 \times Q_e^{b_1} \times Q_c^{b_2} \times e^{\sum G_i b_i}$$

where the variables are:

Q_e	Entering flow on the approach;
Q_c	Circulating flow perpendicular to the entering flow;
G_i	Geometric variables; and
b_i	Model parameters.

It was found that the ‘entering versus circulating’, rear end, and pedestrian flow-only crash prediction models had similar relationships to flow as those developed in Turner (2000). It was reported that models for ‘loss of control’ and ‘rear end crashes’ could not be enhanced by the addition of any of the 28 geometric variables tested. Harper and Dunn (2005) stated that this is not surprising, as the traffic volume variables make many of the geometric variables redundant for the purposes of crash prediction, as a number of the variables were correlated with flow.

The model for the total number of crashes included only one non-flow variable

$$A_{Total} = 5.31 \times 10^{-4} \times Q_e^{0.47} \times Q_c^{0.29} \times e^{ACWL \times 0.057}$$

Where *ACWL* is the 'adjacent circulating width left'. This is the circulating width between the current approach and the next approach in a clockwise direction. Harper and Dunn argue that a narrow circulating width constricts all vehicles entering and circulating around the roundabout, and therefore has a significant influence on speed and crash frequency. The parameter of this variable indicates that as ACWL increases so does the number of crashes.

Two geometric variables were significant in models for 'entering versus circulating' crashes as follows.

$$A_{EvsC} = 2.93 \times 10^{-5} \times Q_e^{0.59} \times Q_c^{0.73} \times e^{(ACDNA \times -0.057)+(EL \times -0.52)}$$

where,

ACDNA	Alternative chord distance to next approach. The distance between the tip of the splitter island of the current approach and that of the previous approach in a clockwise direction, based on the average inscribed circle radius; and
EL	Number of Entry Lanes: The number of entry lanes in the current approach.

Harper and Dunn state that the 'entering versus circulating' model is possibly the most logical, with the number of entry lanes and distance to next approach having strong significance. Their model indicates that the number of crashes of this type decrease with increasing numbers of entry lanes and greater ACDNA.

Harper and Dunn also developed models for pedestrian crashes. The model includes all crossing locations classified as kerb–cut-downs only, zebra crossings and signalized crossings and a number of geometric variables. It is unclear from the paper how many approaches are used with each facility type. The model indicates that as the distance of the crossing from the intersection increases so does the number of crashes. This may be due to a reduction in inter-visibility between drivers exiting the roundabout and pedestrians crossing at the crossing point. The form of the model is

$$A_{Ped} = 4.10 \times 10^{-4} \times Q_c^{0.63} \times e^{(PDG \times 0.058)}$$

Where *PDG* is the pedestrian crossing distance to give way line, or the distance from the give way line of the current approach to the closest point of the pedestrian crossing.

Turner et al. (2006c) developed roundabout models including a number of non-flow variables. The key variables that were found to be significant included:

V_{10}	Visibility 10 meters back from the limit/hold line
S_c	Circulating speed
S_{LL}	Speed at limit/hold lines
φ_{MEL}	Multiple entry lanes

The crash prediction models developed are shown in Table 9.

Table 9: Urban roundabout crash prediction models

Crash Type	Equation (crashes per approach)	Error Structure	GOF**
Entering-vs-Circulating (Motor-vehicle only)	$A = 6.12 \times 10^{-8} \times Q_e^{0.47} \times Q_c^{0.26} \times S_C^{2.13}$	NB (k=1.3)*	0.26
Rear-end (Motor-vehicle only)	$A = 9.63 \times 10^{-2} \times Q_e^{-0.38} \times e^{2.42 Q_e}$	NB (k=0.7)*	0.25
Loss-of-control (Motor-vehicle only)	$A = 6.36 \times 10^{-6} \times Q_a^{0.59} \times V_{10}^{0.68}$	NB (k=3.9)*	0.25
Other (Motor-vehicle only)	$A = 1.34 \times 10^{-5} \times Q_a^{0.71} \times \phi_{MEL}$ $\phi_{MEL} = 2.66$	Poisson	0.17
Pedestrian	$A = 3.45 \times 10^{-4} \times P^{0.60} \times e^{0.67 Q_a}$	NB (k=1.0)*	0.17
Entering-vs-Circulating (Cyclist circulating)	$A = 3.88 \times 10^{-5} \times Q_e^{0.43} \times C_c^{0.38} \times S_{LL}^{0.49}$	NB (k=1.2)*	0.61
Other (Cyclist)	$A = 2.07 \times 10^{-7} \times Q_a^{1.04} \times C_a^{0.23}$	Poisson	0.50
All Crashes	$A = 6.11 \times 10^{-4} \times Q_a^{0.58} \times \phi_{MEL}$ $\phi_{MEL} = 1.66$	NB (k=2.2)*	0.28

*K is the shape parameter for the negative binomial (NB) distribution.
**GOF (Goodness Of Fit statistic) indicates the fit of the model to the data. A value of less than 0.05 indicates a poor fit whereas a higher value (above 0.05) indicates a satisfactory fit.

Table 9 shows the importance of speed and visibility in the various crash prediction models and for the various road users. Given that approach visibility tends to have an influence on approach speed these two variables are inter-related. Arndt (1994 and 1998) has previously shown that there is a relationship between speed, geometric factors and crashes at roundabouts in Queensland. This research also shows a relationship between approach visibility and crashes. Generally lower approach visibility is safer, although there is likely to be a lower limit beyond which safety may be compromised. There are several unanswered questions on the relationships between variables at roundabouts and further research is required.

High Speed Roundabouts (80 kph plus on several approaches)

Using the smaller sample set of 17 high-speed roundabouts (with speed limits on at least two approaches greater than 70 km/hr) the influence of high speed limits was investigated by Turner et al. (2006b). As this data consisted only of the approach volume and number of crashes, no non-flow variables could be examined for this dataset other than speed limit. Using the link flow data a covariate analysis of the effect of higher speed limits on crashes was carried out, as follows.

$$A = 3.21 \times 10^{-4} \times Q_a^{0.66} \times \phi_{HS}$$

where:

A	Annual number of all crashes occurring on each approach
Q_a	Approach flow (sum of entering and exiting motor-vehicle flows); and
Φ_{HS}	Factor to multiply the crash prediction by if there is a (high) speed limit on the approach greater than 70 km/h. This factor is: $\varphi_{HS} = 1.35$

The covariate for the higher speed sites indicates that at speed limits of 80 km/hr or greater there are 35% more reported injury crashes than at a roundabout with an urban speed limit, for a given traffic volume.

PRIORITY CONTROL INTERSECTION MODELS

Low Speed Priority Intersections (50kph or 60kph on all approaches)

Models for low speed (urban) priority intersections were developed by Turner (2000). These crash prediction models are presented in Table 10. At priority cross-roads the straight through flows are differentiated into those with priority (q_p) and those which have to give-way ($q_{g/w}$), or stop. For the crossing (no turns) crashes, both the q_2 and q_{11} flows (refer to Figure 1 for flow descriptions) are used as predictors, but their order in the equation depends on their priority.

Table 10 Priority Cross-road Crash Prediction Equations

Crash Type	NZ Crash type	Equation (crashes per approach)
Crossing (No Turns)	HA	$A = 1.95E^{-3} * q_{g/w}^{0.38} * q_p^{0.37}$
Right Turn Against	LB	$A = 3.75E^{-3} * q_2^{0.05} * q_7^{0.53}$
Crossing (Vehicle Turning)	JA	$A = 5.40E^{-7} * q_2^{1.13} * q_4^{0.44}$
Loss-of –control	C & D	$A = 5.22E^{-3} * Q_e^{0.30}$
Others		$A = 1.87E^{-3} * Q_e^{0.57}$

The crash rates at priority T-junctions are predicted by crash type and approach using the equations in Table 11.

Table 11 Priority T-junction Crash Prediction Equations

Crash Type	NZ Crash Codes	Equation (crashes per approach)
Right Turn Against	LB	$A = 3.33E^{-6} * q_5^{0.48} * q_3^{0.42}$
Rear-end	FA to FD	$A = 1.45E^{-6} * Q_e^{1.18}$
Crossing (Vehicles Turning)	JA	$A = 3.60E^{-5} * q_5^{0.93} * q_1^{0.22}$
Loss-of –control	C & D	$A = 8.22E^{-3} * Q_e^{0.30}$
Others		$A = 2.49E^{-3} * Q_e^{0.51}$

The crash rates at uncontrolled T-junctions (that is T-junctions that have no give-way, stop or signal controls) are predicted by crash type and approach using the equations in Table 12.

Table 12 Uncontrolled T-junction Crash Prediction Equations

Crash Type	NZ Crash Codes	Equation (crashes per approach)
Right Turn Against	LB	$A = 1.49E^{-3} * q_5^{0.31} * q_3^{0.42}$
Rear-end	FA to FD	$A = 8.69E^{-8} * Q_e^{1.5}$
Crossing (Vehicles Turning)	JA	$A = 3.62E^{-4} * q_5^{0.22} * q_1^{0.81}$
Loss-of –control	C & D	$A = 2.51E^{-3} * Q_e^{0.31}$
Others		$A = 6.27E^{-3} b_0 * Q_e^{0.41}$

The parameters for each of these junction types for the major crash types tend to be close to the square root, except rear-end crashes where the exponent is greater than one, indicating that the risk per vehicle increases as traffic volumes increase.

High Speed Priority Intersections (80kph or plus on some or all approaches)

The work on low speed priority intersection was extended by Turner (2006b) to high speed (rural) intersections and non-flow variables, including approach speed and visibility. The typical mean-annual numbers of reported injury crashes for rural T-junctions can be calculated using turning movement counts and the crash prediction models in Table 13.

Table 13: Rural priority T-junction crash prediction models

Crash Type	Equation (crashes per approach)	Error Structure	GOF#
Crossing – Vehicle Turning (Major Road approach to left of Minor Road)	$A = 5.29 \times 10^{-6} \times q_1^{1.33} \times q_5^{0.15} \times (V_{RD} + V_{LD})^{0.33}$	NB (K=8.3)	0.45
Right Turning and Following Vehicle (Major Road)	$A = 5.29 \times 10^{-27} \times q_3^{0.46} \times q_4^{0.67} \times S_L^{11.0}$	NB (K=1.4)	0.48
Other (Major Road approach to right of Minor Road)	$A = 1.59 \times 10^{-5} \times (q_5 + q_6)^{0.91}$	NB (K=1.0)	0.20
Other (Major Road approach to left of Minor Road)	$A = 2.99 \times 10^{-4} \times (q_3 + q_4)^{0.51}$	NB (K=3.0)	0.12

#GOF (Goodness Of Fit statistic) indicates the fit of the model to the data. A value of less than 0.05 indicates a poor fit whereas a higher value (above 0.05) indicates a satisfactory fit.

Table 13 shows that when the sum of the visibility deficiency (visibility is compared with the visibility in the Austroads Guides to Traffic Engineering) to the left and right of the minor road ($V_{RD} + V_{LD}$) increases, the number of crossing-vehicle turning crashes also increases. The model for crashes involving vehicles turning right from the major road and vehicles travelling in the same direction is strongly influenced by the approach speed S_L. The exponent for this variable is positive, indicating that crashes increase with increased speed.

The typical mean-annual numbers of reported injury crashes for rural cross-road intersections can be calculated using the crash prediction models in Table 14.

Table 14: Rural priority crossroad crash prediction models

Crash Type	Equation (crashes per approach)	Error Structure	GOF#
Crossing (Minor Road vehicle hit from left)	$A_{RMXP1} = 1.20 \times 10^{-4} \times q_2^{0.60} \times q_5^{0.40}$	NB (K=0.9)*	0.22
Crossing (Minor Road vehicle hit from right)	$A = 2.05 \times 10^{-4} \times q_2^{0.40} \times q_{11}^{0.44}$	NB (K=2.0)*	0.07

Crash Type	Equation (crashes per approach)	Error Structure	GOF#
Right Turning and Following Vehicle (Major Road)	$A = 1.08^{-6} \times q_4^{0.36} \times q_5^{1.08} \times \phi_{RTB}$ $\phi_{RTB} = 0.22$	NB (K=2.6)*	0.28
Other (Major Road)	$A = 1.14 \times 10^{-4} \times (q_4 + q_5 + q_6)^{0.76}$	NB (K=1.1)*	0.23
Other (Minor Road)	$A = 3.44 \times 10^{-3} \times (q_1 + q_2 + q_3)^{0.27}$	NB (K=0.2)*	0.01

#GOF (Goodness Of Fit statistic) indicates the fit of the model to the data. A value of less than 0.05 indicates a poor fit whereas a higher value (above 0.05) indicates a satisfactory fit.

Table 14 indicates that in high speed environments the right-turn-bay will reduce the number of right-turning and following vehicle crashes by 78% (since ϕ_{RTB} is 0.22).

Arndt (2004) and Arndt and Troutbeck (2005) developed crash prediction models for Queensland, Australia, for urban and rural priority cross-road and T-junctions intersections, taking into account a large number of non-flow variables. This study includes a sample set of 63 cross-roads and 143 T-junctions, with main road operating speeds varied from 40kph to 110kph. A large number of non-flow variables were considered, including the presence of a right turn bay, with even larger reductions in crashes to that observed by Turner.

SAFETY COMPARISON OF INTERSECTION FORMS OF CONTROL

There are three key forms of at-grade control used at intersections; priority, roundabout and traffic signals. For analysis purposes intersections can also be broken down into urban (speed limit of 50 or 60kph on all approaches) and rural/high speed (speed limit on major road is 80 or 100kph). Flow-only models for total crashes have been used to demonstrate the differences in predicted injury crashes and crash cost at each intersection, for urban intersections.

Crash occurrence is influenced by a number of factors, both flow and non-flow. Traffic volumes for the various movements around an intersection, and the number of crashes, can vary considerably from intersection to intersection. The volume of cyclists and pedestrians can also have a major influence on crash occurrence, particularly at roundabouts. Non-flow variables, such as visibility, approach speed and road layout factors can have a major impact on crash rates at all intersection types. So the examples given in this section, along with the preferred form of control for the given traffic volumes, is based on an intersection that has a fairly good layout, normal volume of pedestrians and cyclists and a typical distribution of flows across the various turning movements.

Figure 5, 6 and 7 show the predictions for total crash models for the three forms of control at cross-roads, with minor volumes (lowest volume road) of 2,000vpd, 5,000vpd and 9,000vpd respectively in urban areas (these models are presented in Table A6.2 of the NZ Transport Agency Economic Evaluation Manual). Table 15 shows the flow range over which the crash prediction models can be applied (reference section in EEM). Figure 5 and 7 do not show the traffic signal and priority model predictions respectively as the minor volume is outside the flow range for these models. The crash predictions are also only shown for the Annual Average Daily Traffic (ADDT) which falls within the major traffic volume bands for each model. In the graphs TS stands for traffic signals, while SRnd and MRnd stand for single lane and multi-lane roundabouts respectively.

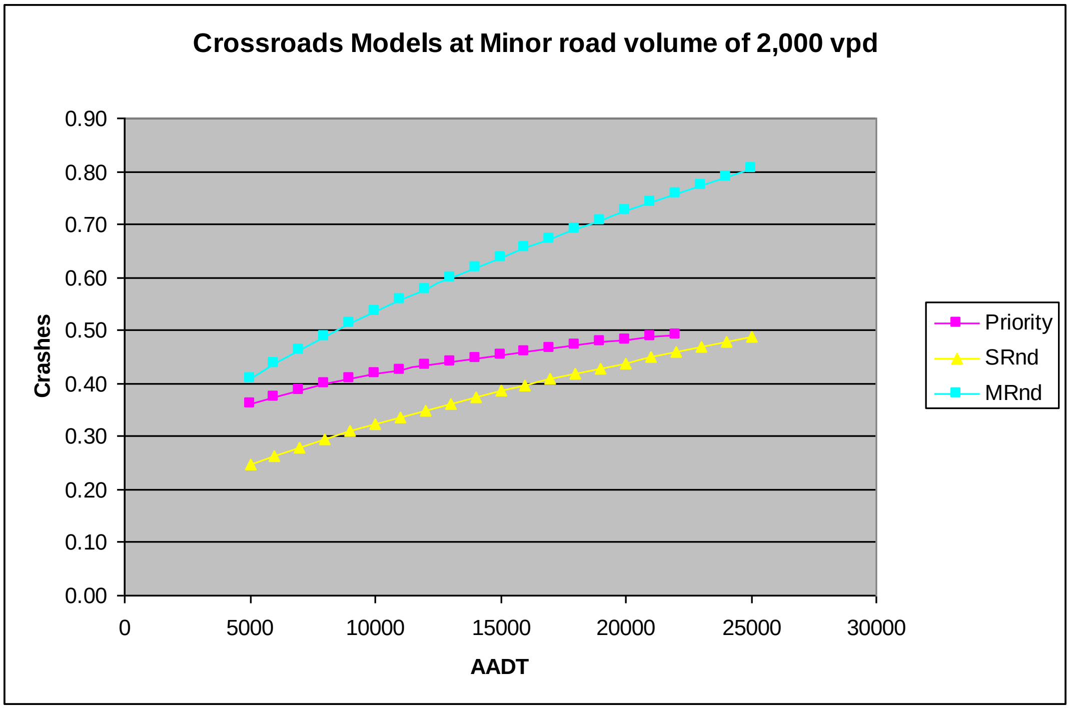

Figure 5 - Crash Predictions where the minor volume is 2,000vpd

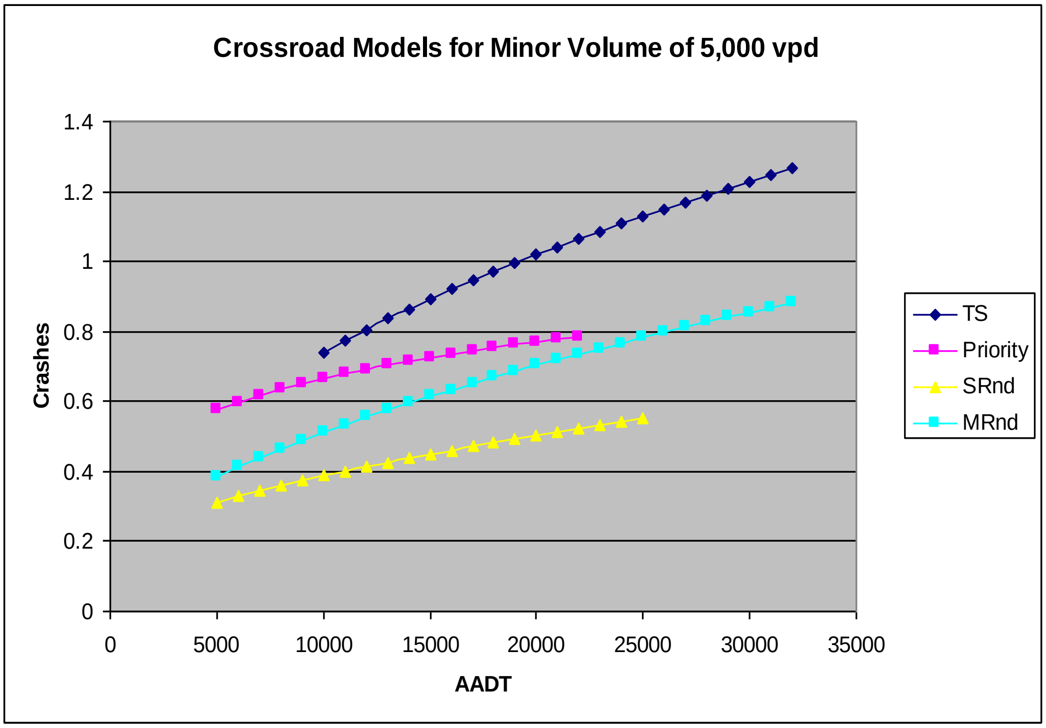

Figure 6 - Crash Predictions where the minor volume is 5,000vpd

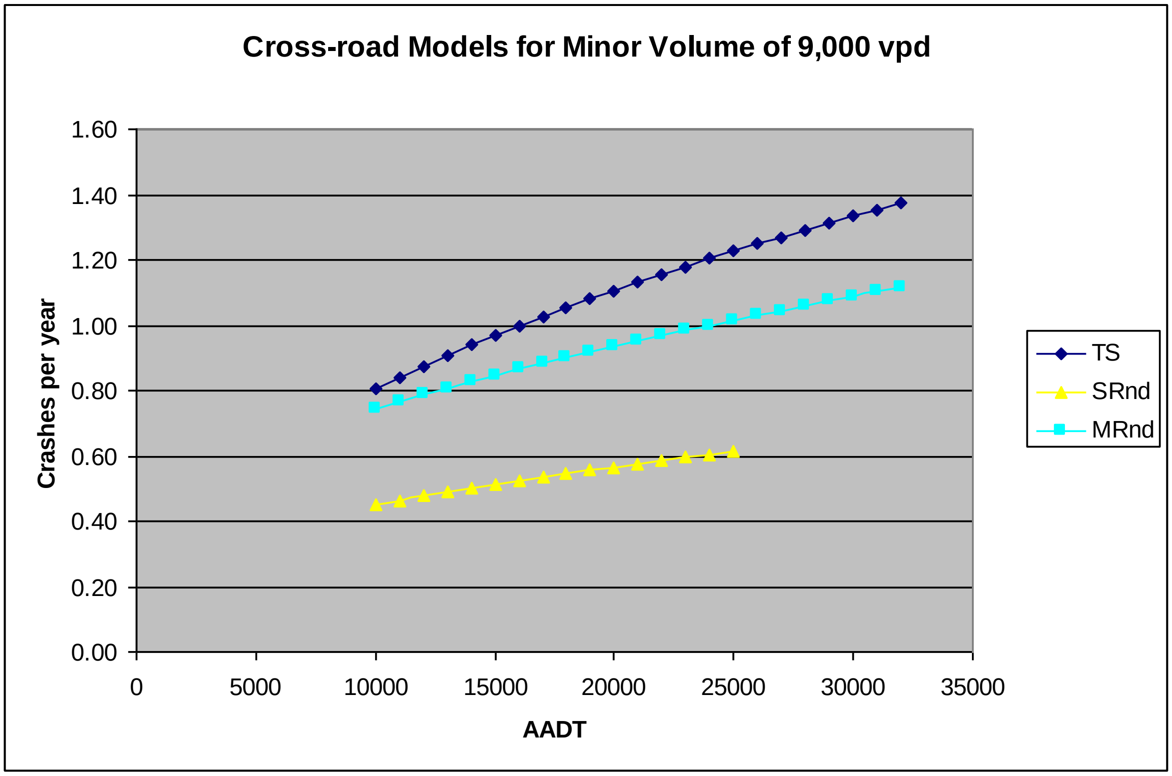

Figure 7 - Crash Predictions where the minor volume is 9,000vpd

Table 15: Traffic Volume Ranges for each Cross-road Crash Model

Intersection Type	Q major		Q minor	
	Minimum	Maximum	Minimum	Maximum
Traffic Signals	10000	32000	5000	16000
Roundabout #	200	25000	200	25000
Priority	5000	22000	1500	7000

the same range is given for both single and multiple circulating lanes

All three figures indicate that roundabouts typically lead to lower crash rates than traffic signals and priority intersections. Even at higher volumes (25,000 vpd plus on the major road and 9,000 vpd on the minor road) multi-lane roundabouts have lower crash rates than traffic signals. However, research by Campbell (2009) indicates that where cycle volumes are median to high, as might occur in Christchurch, that crash rates at roundabouts increase to the levels where they are less safe than traffic signals. This is particularly true of multi-lane roundabouts. Roundabouts are also not ideal in areas with high pedestrian volumes.

The model parameters indicate that for cross-road signals and to a greater extent T-junction signals, the volume of traffic has much less impact on the safety of the intersection, compared with other forms of control. Hence traffic signals have a high crash rate per vehicle at low volume and a much lower crash rate per vehicle as volumes increase.

It is unclear from the current models how priority intersections operate at higher traffic volumes. As major road volumes increase, the availability of gaps for side-road traffic tends to reduce, except where there are gaps created by upstream traffic signals. The current models do not adequately reflect the deterioration in safety at these high volumes as very few of the intersections in the sample set used for the research operate at the higher traffic flows. A separate study is currently underway focused on the safety of high volume priority junctions to assess how these perform as the number of gaps in the main road reduces.

Table 16 shows the crash costs that are applied per predicted injury crash (reference Table A6.22 of Appendix A6, NZ Transport Agency Economic Evaluation Manual). The unit crash costs takes into account the average severity of crashes for each intersection type, the average level of under-reporting of injury crashes and the average number of non-injury crashes per reported injury crash. This table shows that the average unit crash cost per reported injury crash is the same for priority and signalized intersections and slightly lower for roundabouts. This reflects a lower severity of crashes at roundabouts, where the right angle crashes are less prevalent. The affect of the lower unit crash rate reinforces the view that roundabouts are typically safer than priority intersections and traffic signals.

Table 16: Unit Crash Costs at Urban Crossroads

Intersection Type	Unit Crash Cost (per reported injury crash)
Traffic Signals	$170,000
Roundabout	$140,000
Priority	$170,000

DISCUSSION/CONCLUSIONS/FINDINGS

This paper outlines a number of research projects that have focused on developing crash prediction models for various intersections in New Zealand. Some of the key findings include;

1. The crash models for pedestrian and cycle crashes indicates that there is a safety-in-numbers effect, so that the crash rate per cyclist and pedestrian reduces when volumes of these modes increase
2. Loss-of-control crashes at roundabouts can be reduced by reducing the visibility on each approach of the roundabout
3. The crash rates at high speed roundabouts (80kph and above) is 35% higher than that of an equivalent roundabout in a lower speed environment
4. Crash rates at rural priority junctions can be improved by reducing approach speeds on the main road and improving visibility
5. A reduction on average of at least 78% in rear-end shunts into right turning vehicles can be achieved by installing right turn bays at rural intersections
6. The individual crash risk for drivers typically reduces as traffic volumes increase. This is not the case for rear-end crashes where the risk per driver is typically increasing, as illustrated in the exponents being greater than one

7. Roundabouts, particularly single lane roundabouts, are typically safer than traffic signals and priority junctions, except when cycle (and pedestrian) volumes are high.

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