Relationships between road safety and traffic performance in urban areas

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1 Introduction

Urban roads have a crash profile which is characterised by a dominance of multiple (more than one) vehicle crashes due to vehicle-to-vehicle conflicts. These conflict types arise through the high degree of competition between vehicles for use of the same road space at specific locations such as intersections as well as temporal based conflicts such as congestion and interrupted traffic flow as traffic conditions change throughout the day.

With road safety treatments in urban areas, there is a tendency to identify a problematic site and implement improvements in isolation with little consideration for up and downstream conditions. Often the improvements involve physical enhancements such as improving guidance, traffic control or road geometry which are above and beyond what are actually needed in terms of design warrants. However, the high crash rates are often a result of large volumes of conflicting traffic streams, rather than these physical deficiencies. As such, the implementation of isolated improvements in this manner may not be appropriate in addressing the greater road safety problem. In addition, improvements carried out at one location, may lead to traffic re-distribution effects and crash migration which would merely move the problem elsewhere.

In this regard, there is a need to further explore opportunities for addressing urban road safety needs through optimisation of the network from a traffic management perspective. By doing so, the exposure to traffic based conflicts can be better balanced across the whole network to achieve more network-wide benefits. Isolated improvements will still be required for distinct crash clusters, but these should also be implemented with consideration of the greater network.

Stereotypical crash rates associated with varying traffic conditions allow relationships between traffic flow and safety performance in urban areas to be identified. These relationships may present opportunities for identifying traffic flow conditions that will allow for optimisation of the network from both a traffic management and road safety viewpoint. It is emphasised that *optimisation* rather than *elimination* is the key to network-wide crash reduction improvements and that in some cases this may require some sites to continue to have significant rates of crashes for the benefit of the greater network.

Often traffic management and road safety objectives are viewed as being in conflict. At one extreme, a system that facilitates efficiency from a mobility viewpoint, is perceived to have increased safety risks due to (perceived) higher speeds, a high frequency of vehicle-to-vehicle conflicts and poorer space/time separation between conflicting traffic. This is often the case when the system is viewed microscopically. For example, a minimum *all-red* time at a signalised intersection will maximise the capacity of the intersection with the safety tradeoff being reduced time-separation between conflicting traffic streams. Similarly, isolated safety improvements such as banning certain movements will lengthen travel times and possibly prolong the duration of traffic congestion during saturated periods.

A more macroscopic view reveals a more complementary relationship between traffic and safety performance. In the above example, the capacity of an intersection may be increased by reducing the all-red period, but improving the overall capacity of the route will require optimised co-ordination of the traffic control devices at each control point such that the bandwidths permitting *through* travel are maximised. This may allow safety improvements to be implemented which were previously (microscopically) viewed as being in conflict with traffic management objectives. It also provides traffic flow benefits which reduce crash conflicts and the time during which a road user is on the road and hence *exposed* to crash occurrence. Similarly, if turn movements are banned to improve safety at one site, then there may actually be increased congestion in the down stream environment and migration of crash potential to other access points. From a network-wide viewpoint, there may be little net benefit.

2 Stereotypical crash rates

Crash rates expressed as crashes per Million-Vehicle-Kilometres-Travelled (MVKT) are regarded as appropriate safety performance indicators for roads as they account for the number of vehicles "exposed" to the occurrence of a crash event and the length of road in which they can be involved in such an event. Normalising crashes by this "exposure" parameter (MVKT) means that long roads with high traffic volumes can be more comparable with shorter roads carrying less traffic.

However, there are still cases where some roads are inherently much safer than other roads. In such cases, it may not be appropriate to compare the two roads' safety performances without acknowledging this fact. For example, dual carriageway roads are typically much safer than undivided roads. Hence it would be more appropriate to compare the safety performance of a dual carriageway to all other dual carriageway roads. Similarly, the safety performance of the undivided road should be compared with other undivided roads.

As such, the RTA has developed *stereotypical crash rates* which are average crash rates for specific types of roads known as *road stereotypes*. Road stereotypes are defined based on one or a combination of common road attributes which are usually physical aspects of the road such as curve alignment, shoulder width, median width and clear zone standard. A road stereotype can be broadly defined (eg. dual carriageway roads versus undivided roads) or made more specific (eg. undivided roads with shoulder widths between 0-1.0m versus similar roads with shoulder widths between 1.0-2.0m).

There are several advantages of using stereotypical crash rates. These include:

- They provide safety performance benchmarks for different road types.
- They allow for more valid comparisons and assessment of road safety performance as roads can be compared to the benchmarked performance levels for the specified road type (stereotype).
- The benefits of varying improvement measures can be evaluated and compared. For example, it is possible to determine the relative benefits and costs of providing a 1m sealed shoulder versus a 2m sealed shoulder. This also allows for optimisation of resources by incrementally improving the road standard to achieve network-wide road safety benefits, rather than concentrating the limited resources on providing the ultimate standard at a few isolated locations. For example, all roads could be upgraded to have a minimum 1.0m sealed shoulder and then later upgraded to the ultimate shoulder width rather than immediately building to the ultimate for a smaller number of roads.
- They provide an indication of the *safety improvement potential* of the road in its current layout. For example, in comparing (i) a four-leg intersection experiencing 35 crashes/MEV (million-vehicles-entering the intersection) with (ii) a T-intersection experiencing 30 crashes/MEV, the inclination may be to give more priority to the four-leg intersection due to its higher crash rate. However, if the stereotypical crash rate

was 30 crashes/MEV for four-leg intersections and 20 crashes/MEV for T-intersections, there would be more improvement potential with the T-intersection.

- They provide a means of forecasting crash reduction. It should be noted that the use of *before and after* crash analyses for safety projects may be more useful if the crash reduction forecasting is required for prominent crash locations. Stereotypical crash rates can be used for roads where crashes are not clustered or to validate the process. They are also better able to account for regression to the mean, crash migration, crash metamorphosis and risk compensation effects.

Most stereotypical crash rates studies conducted in the past have used road stereotypes defined by physical features that are relatively constant over time (eg. road curvature, gradient, lane width, and shoulder width). These physical features are likely to be singular or dominant factors in the rural road crashes and would also be involved in a high volume of common crash types. This has therefore lead to a good correlation between the physical standard of the road and the safety performance for rural roads.

By contrast, urban roads are more complex. Crash events could result from a number of factors including physical road attributes; external factors such as driver distraction, the complexity of the driving task, large volumes of conflicting traffic; and indirect factors such as landuse and its traffic generation impacts. As such, urban road stereotypes defined by physical attributes would result in a high degree of variability in the calculated crash rates as demonstrated by the following examples:

Example 1: Quantifying the safety impacts of certain landuse types.

The type of landuse adjacent to a road segment would have traffic generation implications on that section of road. Therefore, the crash rate for an industrial area would be affected by the heavy vehicle access demands as well as the time of travel (during business hours). The type of landuse adjacent to a road would be used to define the road stereotype. However, the traffic generated by that landuse would not only be restricted to those segments of road, but to up and downstream segments that may contain other landuse types. Therefore, safety implications of certain landuse would extend to portions of the network which are not populated with that landuse type.

Example 2: Quantifying the safety impacts due to the frequency of conflict points.

The number of intersections and driveways (as sources of traffic conflicts) is often perceived as a good example of a physical feature that would have strong correlations with urban safety performance. However, the drawback is that there is no consideration for the actual volumes of conflicting traffic. Higher volumes of conflicting traffic would tend to have poorer crash rates.

The above two examples demonstrate how an assessment of road safety performance of urban roads by physical attributes would lead to a high degree of variability. Therefore, in selecting an attribute for defining road stereotypes, the following characteristics were sought:

- An attribute that can account for the temporal changes in safety condition.
- An attribute that can represent the symptoms of all other safety deficiencies (ie. Exposure to conflicts, traffic generation issues)

Consequently, *traffic flow performance* rather than physical attributes was used to define the urban roads stereotypes. The advantages in this approach are as follows:

- Traffic flow performance varies by hour of the day, day of the week etc. Therefore, this attribute can better account for both crash exposure and crash type exposure.
- Traffic flow is symptomatic of relevant physical attributes which have implications on safety performance. Using the above example, the more conflict points there are along a route, the greater the likelihood of interrupted traffic flow and crash exposure. Furthermore, traffic flow performance provides a measure of the severity of those conflict points as the traffic conditions varies with different time periods.

3 Crash profiles

A 2005 (unpublished) study by the author showed that the crash profile of a road depends on the extent to which it fits an urban or rural character. For the sake of simplicity, these have been termed *urban* and *rural crash profiles*. However, it must be acknowledged that a road will rarely have 100% of its crashes meeting one profile. Most roads experience crashes from both profiles, but will have a dominance of one profile over the other. Table 1 lists the dominant characteristics of each profile.

Table 1 A simplified comparison of crashes that dominate urban and rural roads.

Urban crash profile	Rural crash profile
Crashes characterised by: • Multiple vehicle involvement • Occuring in interrupted traffic flow • Occuring in lower speed environments • High involvement of temporal based attributes such as congestion, vehicle-to-vehicle conflicts either in causation or outcome of crash. • Tendency for lower severity outcomes due to lower speeds. (NB. However, there are more likely to be pedestrians crashes. These tend to be high severity.)	Crashes characterised by: • Single vehicle involvement * • Occuring in un-interrupted traffic flow • Occuring in higher speed environments • High involvement of physical attributes (road geometry, road side hazard) either in causation or outcome of the crash. These tend to be fixed in time and space. • Tendency for higher severity outcome due to higher speeds.

*Includes multiple vehicle crashes that have the same mechanism as single vehicle crashes. Eg. a loss of control event on a low volume road is likely to remain a single vehicle crash. The same event on a higher volume road may result in an impact with another vehicle.

In these respects, the development of stereotypical crash rates for urban areas should consider the following:

- The higher probability for urban road crashes to include more than one vehicle and to occur at conflict points that often change according to time of day, day of week etc.
- The increased contribution of traffic flow conditions to road safety performance.

4 Study hypothesis

4.1 Objective

The objective is to identify relationships between traffic flow quality and road safety performance to provide opportunities for better integration of traffic and safety improvement projects.

4.2 The hypothesised outcome

The author hypothesises that the relationship between crash occurrence and crash exposure (defined by the MVKT parameter), although widely perceived to be a linear relationship, only remains linear for a low range of MVKTs. At one extreme, a road which carries no traffic (ie. MVKT = 0) cannot experience any crashes. As the MVKT increases, the crash potential also increases in a somewhat linear fashion. For low volumes of MVKT, it is assumed that there is also very little interaction and large headways between vehicles on the road. It is also assumed that under these traffic conditions, the arrival rate of vehicles in minor approaches to intersection is so scarce that the number of vehicle-to-vehicle conflicts is kept to a minimum. Therefore, the rise in crash potential is limited to a small variety of crash types (ie. most probably a range of single vehicle crashes). In a broad sense, this is how many rural roads perform with low ranges of traffic volumes (hence MVKTs) and a limited number of crash types. In Figure 1, this range of MVKTs falls within *traffic condition type 1*. Urban roads can also exhibit this behaviour, but this would tend to be during the times of day when traffic volumes are low (ie. night time).

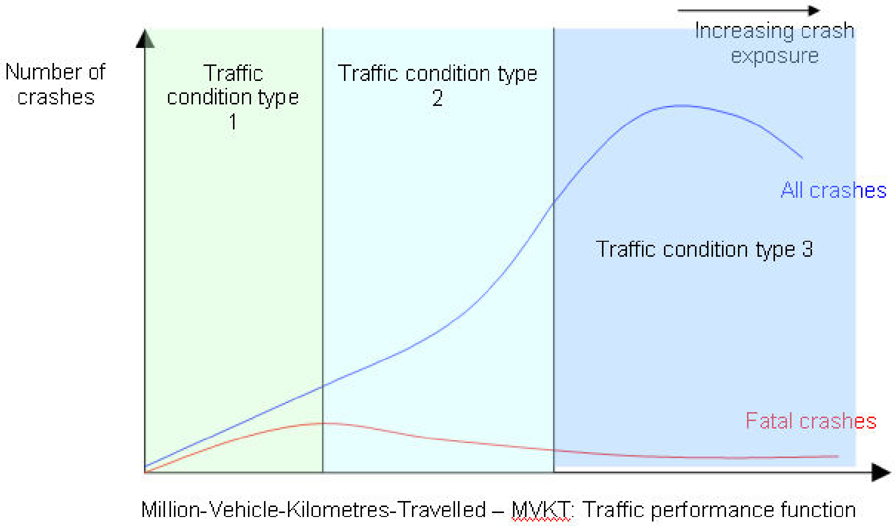


Figure 1 The hypothesised relationship between number of crashes and MVKT

As the MVKT continues to increase, the interaction between vehicles on the road will increase. This would be due to smaller and more variable headways between vehicles as well as the increased arrival rate of vehicles in side roads and hence the greater co-incidence of conflicting traffic streams at these conflict points. Further increases in MVKT would affect traffic flow and increase nose-to-tail conflict (rear-end, side swipe and lane change crashes) due to the stop-start effect, bunching and platooning, persistent lane changing and differential speeds. Abrupt changes in traffic capacity (such as merges and lane terminations) as well as sharp increases in travel demand (such as at intersections, major accesses and access ramps all have the potential of causing flow turbulence, possible congestion and spillback effects into the upstream road section. As such, the crash potential will increase due (i) traffic volume exposure (MVKT) *and* (ii) *crash type exposure* which refers to the exposure to more crash types. As such, the curve appears more exponential than linear. In this case, vehicles can now be involved in a range of both single vehicle crashes and multiple vehicle crashes, as opposed to traffic condition type 1 where single vehicle crashes would be the dominant crash type. The range of MVKT that promote this traffic and safety condition is shown in Figure 1 as *traffic condition type 2*.

Further increases in MVKT would result in increasing congestion, reduced mobility, lower travel speeds and increased delays. This traffic condition may result in further increases in crashes, but due to the loss of mobility and lower travel speeds, the rate of increase tends to slow down as shown by *traffic condition type 3* in Figure 1. The apex of the curve refers to the point where crashes cease to increase due to the level of congestion. The number of crashes would tend to decrease beyond this level of congestion. Whilst this traffic condition leads to a reduction in crash occurrence, it renders the road ineffective in its primary role – to carry traffic. Furthermore, it should be acknowledged that if the traffic condition gets to this level of congestion, then in relieving the congestion, it will naturally have to move to less congested (lower MVKT) levels which are accompanied by higher crash rates.

As this is a hypothesis there are no actual values for the MVKT thresholds bounding each traffic condition.

Figure 1 also shows the hypothesised relationship between fatal crashes and MVKT. By contrast, a disproportionately higher number of fatal crashes may occur in traffic condition type 1 due to the more liberated and less constrained travel speeds potentially raising the severity of any crash event. The counter-argument to this is the consideration of pedestrian crashes, which tend to be high severity crashes even at lower speeds. Pedestrians are also more likely to be on the road during periods when the MVKTs are high, ie. during day-time and weekday peak periods.

In this hypothesis, the increase in MVKT has been assumed to indicate a change in traffic flow quality. This assumption is plausible as road length is fixed so any fluctuation in MVKT would be in traffic volumes. The high MVKT ranges are assumed to correspond to poor traffic flow conditions. For this hypothesis only, it is helpful to regard changes in MVKT as being relative. Traffic volume capacity would vary with each road and hence the threshold MVKT separating each traffic condition type would be different for each road type.

It should also be acknowledged that specific criteria need to be met in order for crashes to be reported. In high MVKT ranges, there may actually be more crashes, but many of these may not be severe enough to be reported.

Figure 2 shows a similar graph where the vertical axis is crash rate per MVKT.

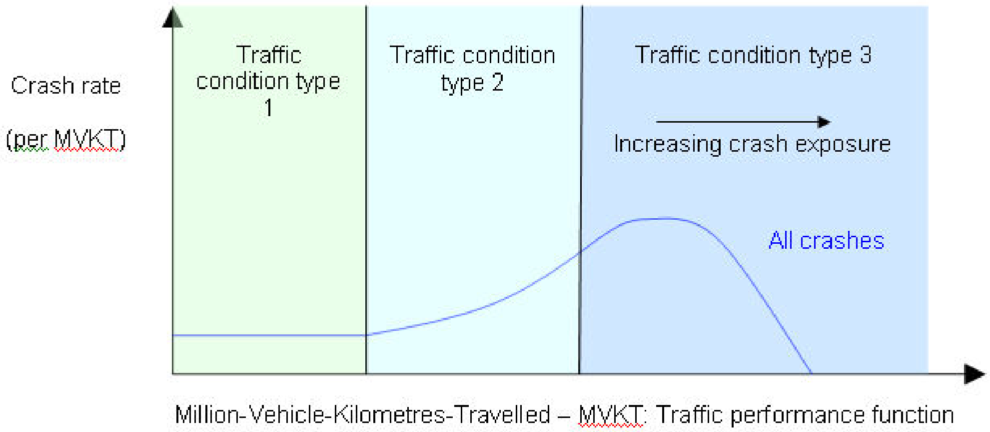


Figure 2 **The hypothesised relationship between crash rate (per MVKT) and MVKT (as a function of traffic flow condition).**

5 Analysis

5.1 Methodology

To identify relationships between crash rates (road safety performance) and traffic flow condition, the data required includes crash, traffic volume and traffic flow performance data. In this case, travel time data has been used to derive a traffic flow performance indicator as described below. Urban road stereotypes have been defined based this performance indicator.

5.2 Traffic flow condition data (derived from travel time)

The RTA routinely collects travel time and hence space mean speed data for the principal traffic routes across the Sydney Metropolitan area. This data is presented for discreet segments along these routes. The segments are usually bounded by key intersections or landmarks such as road overpasses. Travel time data was available for three periods:

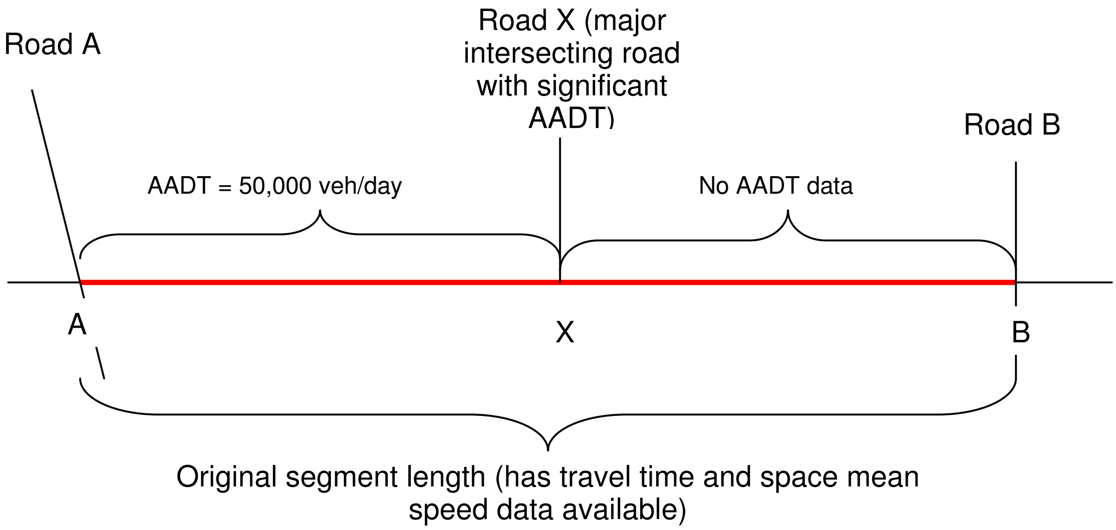
- Weekday AM period (0700-0900h),
- Weekday PM period (1700-1900h),
- Weekday Business Hour period (1000-1500h).

As travel time data was readily available, it was decided that this should be used (rather than other performance indicators like *volume to capacity ratio* or *delay*) to derive the traffic flow condition parameter (P) which defined the urban road stereotypes.

20 principal routes were selected to make up a suitable representative sample of urban roads across the Sydney metropolitan area. In most cases, the segment used to report the travel time data was used as the segment in the analysis. However, there were some cases where segments were further broken up (into smaller segments). In these cases, it was assumed that the space mean speed for the original segment length applied uniformly to the whole segment. This space mean speed (direction specific) was then converted to a direction specific travel time by dividing it into the length of the (new) shorter segment. In reality, the segment may not experience a uniform speed throughout its length due to changes in speed zoning, road capacity, intersection frequency etc. However, the probable error associated with this assumption was regarded as insignificant.

There were several reasons for further segmenting the travel time segments or for omitting them all together. These included:

1. Traffic volume data may not have been available for a significant portion of a (usually a long) segment. See Figure 3.
2. Some segments were removed as they were parallel to and in very close proximity to other roads. As such, it was difficult to differentiate crashes occurring on one road from the other road using the spatial information software (described below). This could be managed if the datasets were later checked. However, this would significantly delay the data collection process.
3. In general, short segments with lengths less than 1km were avoided, although some of them were ultimately included in the analysis.



In this example, travel time and space mean speed data available for the whole length A-B. However, traffic volume data is only available for A-X. Furthermore Road X is a major intersecting road which carries a significant traffic volume (AADT). Therefore, the revised segment length would be A-X and the travel time information would be derived by dividing the length A-X (km) by the space mean speed for A-B (km/h)

Figure 3 A situation where further segmentation of a road segment would be justified.

When road stereotypes are defined by physical attributes that are fixed in time and space, they generally produce one sample road segment for any given length of road. However, when temporal-based attributes, such as travel time (which varies throughout the day and week) are used, the number of sample segments produced depends on the number of traffic flow condition periods. In this study, there were three such periods being AM Peak, PM Peak and business hours on weekdays as described above. Therefore, any given length of road generated three samples for which there was a relationship between traffic flow condition and road safety performance. In future, the RTA will be collecting travel time information for weekends, which will enable inclusion of this period as an added sample.

The 20 principal traffic routes included in the sample yielded 128 discreet road segments, which then boosted the sample to 435 entries when the three traffic flow condition periods were taken into account.

Travel time as a stand alone parameter cannot accurately represent traffic flow quality as some segments are much longer, or have lower speed limits than others. For this reason, a traffic flow condition (unit-less) parameter (P) was developed as shown in the equation below:

$$\text{Traffic flow condition parameter (P)} = \frac{\text{Volume weighted travel time}}{\text{Theoretical Ideal Travel Time}}$$

Volume weighted travel time (h) is the travel time for a segment which is calculated by accounting for the travel time in each direction and the amount of traffic (in each direction) experiencing those travel times. The formula for calculating the volume weighted travel time is given below.

$$\text{Volume weighted travel time} = \frac{\{(\text{Travel time direction 1} * \text{Traffic volume direction 1}) + (\text{Travel time direction 2} * \text{Traffic volume direction 2})\}}{(\text{Traffic volume direction 1} + \text{Traffic volume direction 2})}$$

The *theoretical ideal travel time (h)* is a hypothetical travel time which assumes that a vehicle can travel from the beginning of a segment to the end at the posted speed limit without stopping. Although this is rarely achievable, it does provide an indication of the segment's potential with regard to traffic flow quality. If there are several speed limit changes in a segment, then the following formula is used to determine the theoretical ideal travel time:

$$\text{Theoretical ideal travel time} = (\text{Distance 1} / \text{Speed limit 1}) + (\text{Distance 2} / \text{Speed limit 2}) + \ldots \ldots + (\text{Distance Z} / \text{Speed limit Z})$$

There were usually no more than three speed limit changes in the sample of road segments.

The traffic condition parameter (P) ensures the traffic flow performance for a segment for any given time period can be compared with the performance of other segments or to itself when operating in another time period. Theoretical ideal travel time will always be shorter in duration than the measured travel time. Therefore, the smaller this ratio (ie. the closer to 1.0 this ratio is), the better the traffic condition. Contrastingly, the greater this ratio is, the poorer the traffic condition.

A decision was made to use a volume weighted travel time accounting for individual travel times in both directions rather than the direction specific travel time. Originally, it was perceived that travel time differences in each direction (indicating a non-equal distribution of traffic for each direction) would enable direction specific crash rates to be compared to indicate the impact of traffic flow on safety performance. However, most of the roads in the analysis were single carriageway undivided roads where traffic in one direction could not be considered exclusively and independently of the traffic in the opposing direction. Several examples illustrate this point as detailed below:

Example 1: *Where vehicles are required to perform right-turns across opposing lanes. In this example, the traffic condition in the other direction affects the ability for the motorist to perform this right turn. Therefore any traffic condition in the opposing direction immediately becomes relevant for the primary direction.*

Example 2: *Vehicles in opposing directions on undivided roads are often in conflict with each other. As vehicles in one direction can crash into vehicles in the other direction, the two directions cannot be considered exclusively and independently.*

In this regard, the volume-weighted travel time accounting for both directions was considered the most appropriate way of representing traffic flow quality for the segment.

5.3 Traffic volume data

The traffic volume data used in this analysis was provided through either sample stations (short duration surveys) or permanent traffic counters (continuous survey). Sample stations were only able to provide a daily traffic volume (Annual Average Daily Traffic - AADT) in vehicles per day. The permanent stations were able to provide both an AADT as well as the distribution of traffic per hour-of-day and day-of-week. The latter was useful as traffic data for specific hours of the day were required (ie. *AM peak* required weekday traffic volume data for the period 0700-0900h, *PM peak* required data for the 1700-1900h period and the *business hour* period required data for the 1000-1500h period).

The traffic volumes for each of the time periods was regarded as a daily traffic volume for that specific segment and time period.

In cases where permanent traffic counting stations were not available for a segment, the traffic volumes for each of the time periods were derived using the overall AADT for the segment, and assuming a similar hourly distribution as a nearby permanent traffic counting station.

One possible source of error, particularly with long segments, is that traffic passing over a counter near the end of the time period may still be within the segment when the time period is over. (ie. accounting for traffic that uses the segment outside the time period). Similarly, at the beginning of the time period, there would be vehicles in the segment that would have passed over the traffic counter before the period commenced (ie. not accounting for traffic that is using the segment in the time period). This lag effect was not considered to be significant as (i) there were very few long segments and (ii) the extra vehicles counted at the end of the period and the number of vehicles not counted at the beginning of the period would most likely cancel each other out.

5.4 Crash data

With the segment boundaries identified by (i) the original travel time segments and (ii) (where necessary) revised according to the availability of traffic volume data (Figure 3), crash data was then extracted for each segment. Using spatial information software and geocoded crash data, crashes were manually selected by visually scanning each road segment. It was assumed crashes, particularly near the limits of each segment were geocoded accurately. There was no other way of controlling for geocoding errors.

In extracting crash data for each segment, specific rules were observed to ensure consistency. These included:

- Where segments were bounded by intersections, any crash occurring on the side of the intersection within the segment and at the actual intersection of the two roads were included as crashes for that segment. Any crashes occurring on the side of the intersection outside the segment were not included, even though many of these were coded as *intersection crashes*. This rule was observed to minimise double counting of crashes to adjacent segments. The crashes geocoded to actual intersection of the two roads was, however, included in both segments.
- For practical purposes, crashes coded as *intersection crashes* that occurred on the minor approaches of an intersection were not included in the segment. It was difficult to determine whether these crashes were related to traffic flow deficiencies on the primary road or on the minor road (particularly in the case of crashes that occurred in intersection departures).

Once the crash dataset for each road segment was extracted, the numbers of fatal, injury and towaway crashes for the period 2001-2003 were extracted and recorded for each time period. This allowed crash rates to be determined.

5.5 Crash rate derivation

By this stage, the following information was available:

- Identified segments.
- A measure of the traffic flow performance (P parameter).
- Direction-specific and bi-directional traffic volume data for each of the three time periods.
- Crash data broken down by fatal, injury or towaway severity class for each of the time periods.

Crash rates were then derived using the equation below:

$$\text{Crash rate (per 100MVKT)} = \frac{(\text{No. of crashes}) * (100{,}000{,}000)}{(\text{AADT}) * (\text{no. of days per year}) * (\text{no. of years}) * (\text{length of segment})}$$

The *number of crashes* was the number pertaining to the given time period and segment for the three year period from 2001-2003.

AADT was equivalent to the number of vehicles per day using the segment for the specified time period.

Number of days per year referred to the number of days for which the time period (AM, PM or business hour) was experienced per year. In this case, there were 261 weekdays per year. The effects of public and school holidays were not considered in this study. The *number of years* of crash data was three years.

The *length of the segment* was expressed in kilometres.

5.6 Traffic Control Signal Frequency

A potential risk was that a short segment with a high density of conflict points (such as intersections) would exhibit a much higher crash rate irrespective of the traffic flow condition. As the frequency of conflict points (per kilometre) is rarely consistent across all segments, it was necessary to develop a parameter for measuring this. For practical purposes, it was decided that the number of traffic control signals (intersections and midblocks) per kilometre would be an appropriate indicator of the frequency of (significant) conflict points was across the segments.

As shown in the results, crash rates for the segments have been categorised by the frequency of traffic control signals per kilometre.

6 Results

Figures 4 and 5 show plots of all segments with regard to the absolute number of crashes versus the MVKT parameter. The data has been further categorised by the number of traffic control signals (TCS) per kilometre with 0-1, 1-2 and 2-3 TCS/km plotted in Figure 4 and 3-4, 4-5 and >5 TCS/km plotted in Figure 5.

Trend lines have also been plotted using a 2^{nd} order polynomial curve with correlation factors (R^2) ranging from 0.3 to 0.8.

Although there is a high degree of variability in the plotted results, the curves fit the data range reasonably well. The general trend is that the absolute number of crashes rises initially with increasing MVKT. However, this tends to peak and then decline with further increases in MVKT. These trends tend to support the hypothesised result in Figure 1, particularly with regard to the shape of the curve for traffic condition types 2 and 3.

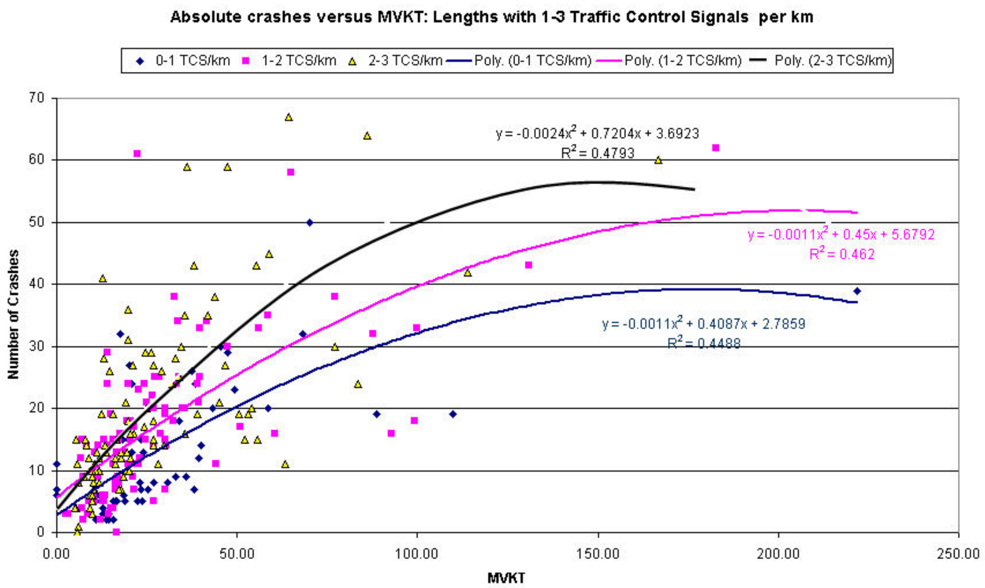


Figure 4 **Absolute crashes versus MVKT for segments with 0-3 traffic control signals per kilometre**

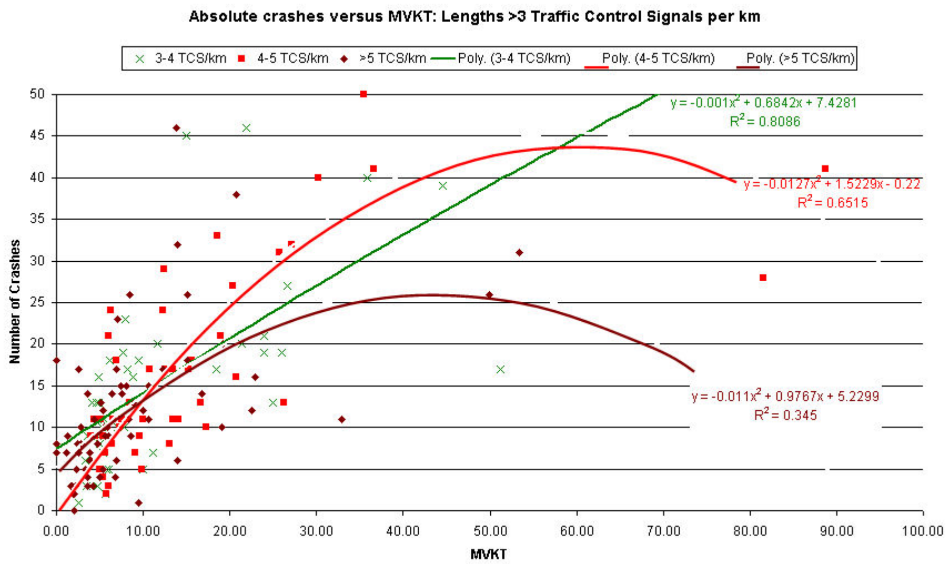


Figure 5 **Absolute crashes versus MVKT for segments with more than 3 traffic control signals per kilometre**

Figures 6 and 7 show the relationships between absolute numbers of casualty crashes and the MVKT parameter. The attempt was also to prove the hypothesised outcome (red curve) in Figure 1. Similarly, 2nd order polynomial curves have been used as trend lines and these have correlation factors (R^2) ranging from 0.2 to 0.8. These trend lines do not strongly support the hypothesised outcome.

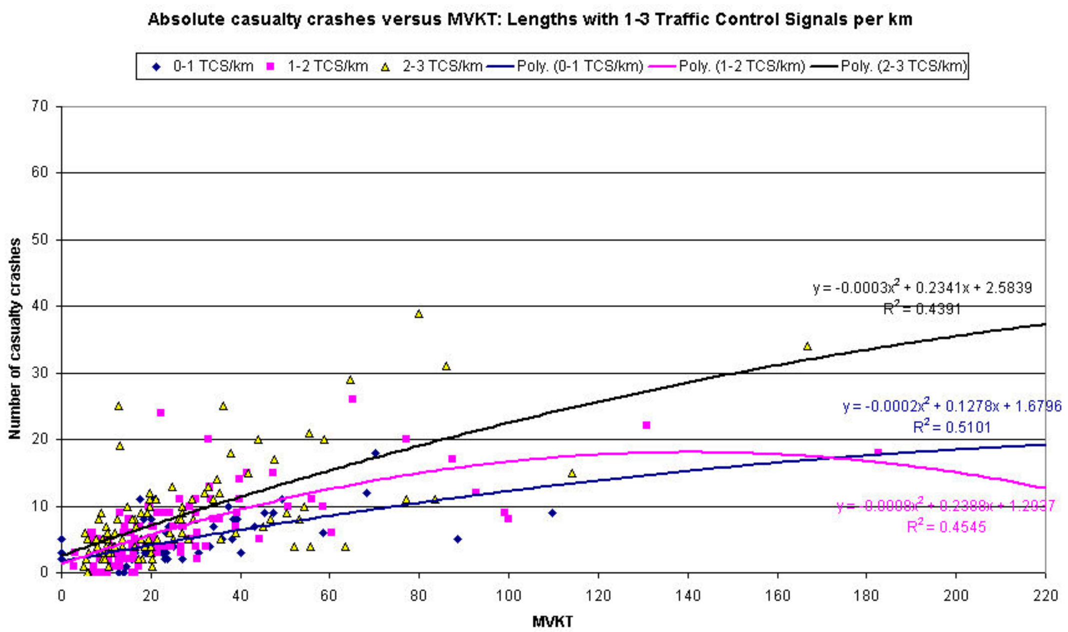


Figure 6 **Absolute casualty crashes versus MVKT for segments with 0-3 TCS per kilometre.**

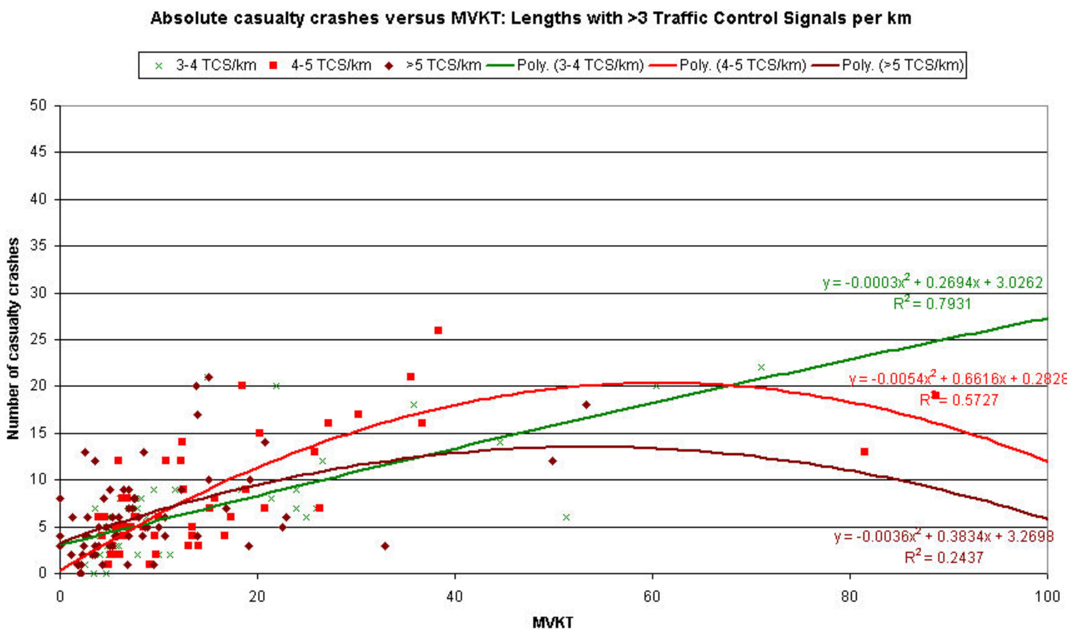


Figure 7 **Absolute casualty crashes versus MVKT for segments with more than 3 traffic control signals per kilometre**

Figure 8 shows the plot of the crash rate (per 100MVKT) versus the traffic flow parameter P (equivalent to volume weighted travel time/theoretical ideal travel time). Similarly, there is much variability in the results. Trend lines for differing frequencies of TCSs have also been plotted using 2nd, 3rd and 4th order polynomial curves. The high degree of variability has resulted in low R^2 correlation factors.

The trend lines support the hypothesised result as presented in Figure 2, particularly for the portions of the curve that related to traffic condition types 2 and 3. This is particularly the case for the 3rd and 4th order polynomials.

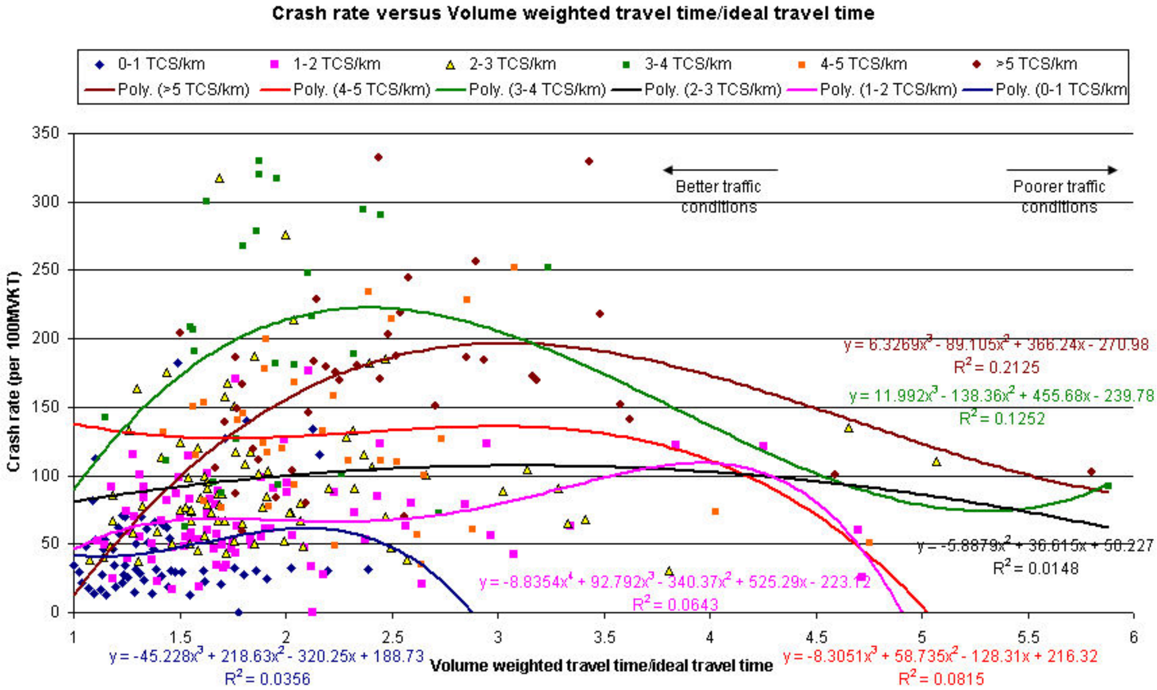


Figure 8 **Crash rate (per 100MVKT) versus the traffic flow parameter P (volume weighted travel time/ideal travel time).**

Figure 9 shows the plot of average crash rates (per 100MVKT) for different TCS frequency. Separate lines have been provided for each of the time periods. As seen, increases in the frequency of traffic control signals (per kilometre) result in an increase in crash rate as well. This result was expected as the frequency of traffic control signals was considered to be an appropriate indicator representing the number of conflict points per kilometre. The interesting result is that most of the curves show a relative trough for segments with 4-5 TCS/km. This could be due to the hypothesis detailed above, whereby the crash rate tends to reduce in poorer traffic flow conditions, which would be expected by such a high frequency of signals (particularly if poorly coordinated).

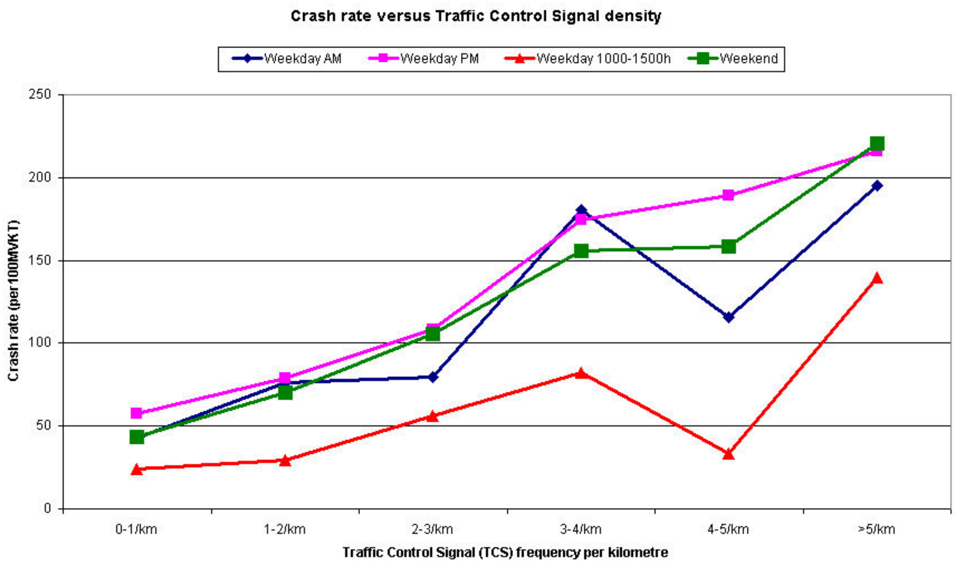


Figure 9 **Average crash rate versus Traffic Control System frequency.**

Figure 9 also shows that the AM and PM weekday periods and the Weekend period exhibited crash rates which were significantly higher than the Weekday business hour period.

7 Conclusions

The study hypothesis was that there were three broad traffic condition types which exhibited differing relationships between traffic flow condition and road safety performance. In low ranges of traffic volumes, a linear relationship is believed to exist between the absolute number of crashes and traffic volume. The crash exposure under these flow conditions is believed to facilitate a low range of crash types (mainly single vehicle crashes).

As the traffic volumes increase and the traffic flow changes to become more interrupted, there is also an increase in traffic conflicts which increase *crash exposure* as well as *crash type exposure*. As such, the rate of increase in crashes becomes more exponential. With further increases in volume, the traffic flow, which is closer to saturation levels, constrains both flow and speed, which in turn results in a reduced rate of increase in crashes. There will be a critical point where the crashes will cease to increase with further increases of traffic volume. This is when the traffic flow becomes forced and the low speeds result in a reduced crash probability.

A sample of 128 road segments with individual travel time, traffic volume and crash data were used to generate a sample of 435 road segments with time-specific traffic flow characteristics. Crash rates were calculated for each segment and for each of the time periods as well as a traffic flow condition parameter (P), equal to the ratio of the volume weighted travel time to the theoretical ideal travel time. The crash rates and the P value for each entry enabled the relationship to be plotted. Results were further differentiated with regard to the frequency of traffic control signals as an indication of the number of conflict points in each segment.

Trend lines were fitted to the plotted results based on 2nd, 3rd, and 4th order polynomial curves. Despite the high degree of variability (which was expected), the trend lines provided sufficient support for the hypothesised outcome. This suggests that there are opportunities for improving the safety performance of urban areas through traffic management strategies. This is particularly the case with the large magnitude of crashes that result from vehicle-to-vehicle conflicts either at conflict points or in locations and time periods where traffic flow quality is poor. Traffic that is constantly moving is more likely to have minimal exposure to traffic conflicts and hence reduced crash potential at conflict points. In addition, improved flows and reduced travel times ensure that vehicles are removed from the traffic system as early as possible, thereby reducing the time exposure to crash causation factors. Smooth traffic flows are also likely to have larger and more consistent headways between following vehicles which further reduces crash potential.

In most cases, it would not be possible to convert heavily congested road segments to ones that have traffic flow characteristics of rural highways. However, all urban roads will have periods of the day when traffic flow is at least closer to the rural situation (ie. with uninterrupted flow, and minimal volumes of conflicting traffic). This implies that if the traffic flow can be managed to exhibit these characteristics, then crash reduction benefits are also likely to be achieved. Indeed, the best approach may be to implement measures to ensure network-wide, albeit smaller scale, traffic and safety improvements and continue to incrementally improve the performance of the road network.

The need to optimise the network from both a traffic management and safety viewpoint may lead to a shift from the more traditional practice of implementing physical improvements to one that exploits the potential of electronic and intelligent systems that are highly automated and real-time. This may be achieved by providing more advice to motorists on real-time traffic performance to enable equitable route selection as well as further optimising traffic flow through better predictability of traffic arrival rates at conflict points.