Speed limit reductions to support lower SCRIM investigatory levels

February 2018

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NZ Transport Agency research report 636
Contracted research organisation – Opus International Consultants

ISBN 978-1-98-851286-0
ISSN 1173-3764 (electronic)

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Cenek, P and R Henderson (2018) Speed limit reductions to support lower SCRIM investigatory levels. *NZ Transport Agency research report 636*. 77pp.

Opus International Consultants was contracted by the NZ Transport Agency in 2014 to carry out this research.

Keywords: crash risk, economic evaluation, operating speed, skid resistance investigatory levels, speed limit setting

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Acknowledgements

The authors gratefully acknowledge Tiffany Lester of Opus Research for her assistance in locating technical references, extracting crash statistics for SH58 and report formatting. The authors would also like to thank the peer-reviewers Dr Doug Wilson (Senior Lecturer, University of Auckland) and David Wanty (Director, Wanty Transportation Consultancy Ltd) for their many helpful comments and in the case of David Wanty, his assistance in reviewing and integrating all the individual spreadsheets developed by the project team into a tool that has an interface that is familiar to the industry. Finally, the support and technical guidance provided by the Transport Agency's project manager, Dr Fergus Tate (Lead Safety Advisor, Roads and Roadsides), have been much appreciated.

Contents

Executive summary

Because of financial and resource constraints, there is sometimes a need to manage the wet friction of state highways to values below those specified by the NZ Transport Agency's T10 specification. This results in an adverse impact on crash risk that can be negated through reducing speed limits with attendant benefits (ie reduced vehicle operating costs and longer seal lives) and dis-benefits (ie longer travel times). Research was therefore undertaken to develop an analysis framework that allows the impact of lowering both the wet friction levels that state highways are managed to and speed limits to be assessed on the basis of crash risk and road user costs.

The analysis framework is based on vehicle speed-related procedures incorporated in the NZ Transport Agency's *Economic evaluation manual*, which include travel times, vehicle operating costs, carbon dioxide emissions and crash severity. New Zealand-specific research related to relationships between the skid resistance level of the road surface and curve crash risk and expected service life of the road surface was also employed in the framework. As a consequence, the calculation of benefits resulting from the combined lowering of maintenance levels of skid resistance and speed limits is pertinent to New Zealand conditions and can be directly compared with other competing asset preservation or safety projects.

A prototype spreadsheet tool was written to perform the analysis framework and is located in a separate appendix D, available at www.nzta.govt.nz/resources/research/reports/636. All inputs required can be readily extracted from the following Road Assessment and Maintenance Management System (RAMM) tables:

- carriageway
- curve context
- high-speed geometry
- sign
- skid resistance

The state highway (SH) 58 section with a 100 km/h speed limit was selected to trial the analysis framework. This section includes 28 high-risk curves, is approximately 10 km long and has an average crash density of 0.72 casualty crashes per kilometre per year. The key findings of this trial application were as follows:

- The analysis was dominated by a significant margin by speed change cycle and travel time considerations. This highlights the need for accurate estimation of free-flow vehicle speeds along rural state highways, especially in the vicinity of curves, if the economic assessment of adopting lower speed and skid resistance limits is to be robust.
- A decrease in operational speed of only 10 km/h from 100 km/h to 90 km/h was found to be sufficient to completely negate the increased curve crash risk that results when the skid resistance of the high crash risk curves is managed to a level of 0.4 equilibrium SCRIM coefficient (ESC) rather than the recommended investigatory level, which on 19 of the 28 curves was ≥ 0.5 ESC.
- The associated user benefit was significant, corresponding to an annual cost saving of about $1.5 million. This indicates that, for SH58, reducing the 100 km/h speed limit is a very effective safety measure if the skid resistance of the high-risk curves cannot be maintained at or above their recommended investigatory level.

- The existing RAMM curve context table needs to be updated as it has been based on 2010 geometry data and curve entry and departure speeds that assume near instantaneous acceleration/deceleration. However, in trialling the proposed analysis framework on SH58, the six-year time difference between the geometry data that forms the basis of the curve context table and that contained in the current road assessment and maintenance management (RAMM) geometry table did not prove to be problematic. This is as expected if no major road alignment has taken place in this intervening period, which can easily be checked by extracting geometry data for both the current and the 2010 survey contained in the RAMM geometry table and comparing curvature profiles for the length of state highway of interest.
- The current (2017) version of RAMM contains both speed and speed limit tables but these have not yet been populated. Having both validated speed and speed limit data would considerably simplify and improve the robustness of both the speed limit setting and annual cost analysis performed by the spreadsheet tool. Therefore, it is recommended that priority is given to populating these two RAMM tables. Ideally the free vehicle speeds in the speed table will be derived from actual speed measurements of both passenger and heavy commercial vehicles rather than from theoretically derived vehicle speeds.

Abstract

This report details a framework for rationally arriving at economically justifiable operating speed reductions to compensate for the inability to achieve recommended levels of skid resistance on high-risk curves. The framework is based on vehicle speed-related procedures incorporated in the Transport Agency's *Economic evaluation manual.* These procedures include travel time, vehicle operating costs, carbon dioxide emissions and crash severity. Relationships between the skid resistance level of the road surface and curve crash risk and expected service life of the road surface derived from previous New Zealand specific research are also employed.

The framework was trialled on a 10 km section of state highway 58 with a 100 km/h speed limit. A decrease in operational speed of only 10 km/h was found to be sufficient to completely negate the increased curve crash risk that results when the skid resistance of the high crash risk curves is managed to a lower level of 0.4 ESC rather than the recommended investigatory level. The overall benefit of reducing the 100 km/h speed limit by 10 km/h amounted to an annual cost saving of about $1.5 million, with the reduction in speed change cycle costs making the largest contribution.

1 Introduction

The Ministry of Transport's 2014 statistical statement of road crashes in New Zealand (MoT 2014), shows that on open roads, loss of control while cornering accounts for the majority of injury crashes by a significant margin (1,280 out of 3,349 corresponding to 38% of open road injury crashes) and is second to head-on (not overtaking) for fatal crashes (49 out of 190 corresponding to 26% of open road fatal crashes).

Interrogation of the NZ Transport Agency's (the Transport Agency) crash analysis system (CAS), New Zealand's primary tool for capturing information on where, when and how road crashes occur, confirmed that loss of control while cornering accounts for a significant percentage of fatal and serious injury crashes on open road (rural) sections of state highway. With reference to table 1.1, loss of control (LoC) while cornering accounted for 20% of fatal crashes and 31.5% of serious injury crashes on rural state highways over the five-year period 2011–2015.

Table 1.1 Rural state highway crash numbers for the period 2011–2015

Year	Fatal		Serious		Minor		Injury	
	LoC, cornering [a]	All	LoC, cornering	All	LoC, cornering	All	LoC, cornering	All
2011	24	112	139	455	548	1,705	711	2,272
2012	25	131	152	470	530	1,643	707	2,244
2013	24	93	141	429	496	1,624	661	2,146
2014	20	114	132	466	489	1,529	641	2,109
2015	23	127	155	467	537	1,766	715	2,360
Average	**23**	**115**	**144**	**457**	**520**	**1,653**	**687**	**2,226**

[a] CAS movement codes DA, DB and DC as per MoT (2014)

The *Transport Agency research report 477* (Cenek et al 2012a) documents the relationship between road curvature, the degree to which a curve is out of context, road surface friction as measured by the sideway-force coefficient routine investigation machine (SCRIM) and injury crashes. This relationship and curve crash history have been used to identify the most appropriate SCRIM skid resistance investigatory level (IL) for the safety management of curves with a horizontal radius of curvature of 400 m or less located on rural state highways. The procedure adopted for assigning ILs to moderate to tight curves is detailed in Cenek et al (2011).

Unfortunately, the polishing action of cornering traffic can cause the friction provided by the road surface to fall below the IL well before the end of the surface's design life if the aggregate used has insufficient polishing resistance for the traffic volume or cornering forces at the site. When surfaces reach an undesirably low skid resistance, the only fix has traditionally been to apply a reseal with an aggregate that has more polishing resistance. In extreme cases, this may involve the use of artificial aggregates such as calcined bauxite or melter aggregate or to transport in natural aggregates with proven resistance to polishing such as Waiotahi or Reids Pitt, causing the cost of resealing to increase substantially.

Because of maintenance budget constraints and limited availability of aggregates that are highly polishing resistant, it is not possible to ensure all curves are appropriately treated to provide SCRIM skid resistance levels that remain above the specified IL throughout the design life of the seal. As a consequence, treatment of curves with lower than desirable levels of friction has to be prioritised. Prioritisation means some curves may not be treated at all or treated with aggregates that have inadequate polishing resistance and so will have a heightened crash risk. Typically, these will be curves on lower volume, lower standard highways, with a history of few or no loss of control crashes.

One way to mitigate the increased risk associated with lower levels of road surface friction is to reduce the demand for friction by lowering the operating speed of the highway. While this approach could be applied temporarily on a curve-by-curve basis, a route-based approach that would result in speed limits on sections of lower volume highway being permanently lowered, may be more appropriate.

The question is how would such decisions be made to ensure optimal outcomes?

There are two components to this problem. First, what is the relationship between changes in speed limit and changes in operating speeds, and second, what is the impact of changes in operating speed on crash risk, vehicle operating costs (VOC) and travel times. This report focuses on the second component.

The report has been structured as follows. Chapter 2 summarises the key findings of a comprehensive literature review undertaken to identify wet friction, crash risk, VOC and travel time models that have vehicle speed as an input parameter and are appropriate for New Zealand conditions. Chapter 3 presents an analysis framework for investigating the economic consequences of either lowering wet friction levels and/or lowering operating speeds over a route. Chapter 4 covers the application of the proposed analysis framework to a trial rural state highway and the resulting key findings. Chapter 5 gives details of the spreadsheet tool (available as appendix D at www.nzta.govt.nz/resources/research/reports/636) that has been created for investigating the impact of different wet friction levels and speed limits on expected crash risk and for calculating the associated whole-of-life costs. Conclusions and recommendations are presented in chapter 6.

2 Related literature

Literature was consulted to:

- Identify and recommend the most appropriate models to incorporate in the analysis framework. The models included:
  - wet friction models
  - crash risk models
  - VOC models
  - travel time models.
- Determine what actual travel speeds result from a posted rural speed limit.

Specific topics covered by the literature review included:

- prediction of skid resistance degradation over time
- prediction of speed on two-lane rural roads
- prediction of speed changes when entering and exiting a curve
- effect of speed changes on VOCs
- curve crash risk
- expected lives of road surfaces used on the New Zealand state highway network
- examples where speed limits have been reduced on two-lane rural highways for possible validation of the analysis framework.

As well as the NZ Transport Agency's (2013) T10 skid resistance policy, the following are two key references:

- Cenek et al (2012a).
- appendices A4 and A5 of the *Economic evaluation manual* (EEM) (NZ Transport Agency 2016a).

The following Austroads and/or Monash University publications were also considered relevant to the review:

- de Roos and Jurewicz (2010) (of particular relevance in this Austroads reference are table 1 (page iv) and the first nine pages of section 3 (ie section 3 up to 3.1.3, or pages 8 – 16)
- Fildes et al (2005)
- Cameron (2009)
- Jurewicz et al (2014).

Essentially, these references are literature summaries in part. Accordingly, attempting to further summarise these references would likely omit key findings for the sake of brevity. Therefore, further summary is not always attempted here. Instead, readers are encouraged to read the complete references.

Finally, the power-point presentation by Kinnear (nd) of the UK's Transport Research Laboratory (TRL) is recommended as useful background reading because a fair proportion of it is loosely relevant to some of the topics of interest.

In this summary of the related literature, often numerical data appears to be 'missing' (ie not included). This is not the case, and any relevant data in the reviewed literature is reported if it was available. (If it is absent from this review, then it can be taken that the source document also omits the desired numerical data.)

2.1 Skid resistance

2.1.1 Prediction of its degradation

Crowley and Parker (2001) comment that 'most highways are resurfaced to remedy poor friction performance well in advance of any structural failure'. Given this, estimating skid resistance performance with age is important in pavement life analysis.

This skid resistance performance of a surface can be plotted as a function of axle-passes (eg Roe and Hartshorne 1988). To assess the skid resistance degradation over time, the skid resistance of a pavement can then be plotted against cumulative vehicle passes (ie age x annual daily traffic (ADT), eg Cenek et al (2012b). According to Cenek et al (2012b, p22), the variation of SCRIM coefficient with ADT is approximately '0.004 per 100,000 vehicle passes'.

No other significant literature on the above topic could be found. (However, although not relevant for other than stone mastic asphalt (SMA) surfaces, Woodward et al (nd) investigate why the early and midlife skid resistance of SMA surfaces can be lower than when the surface is more mature.)

2.1.2 General non-degradation literature

2.1.2.1 Models

Claeys et al (2001) report on a mathematical model of a tyre on a wet pavement. The model appears detailed and takes into account, via the Navier-Stokes fluid flow equations, such phenomena as the tyre tread sinkage time (ie the time required for the tyre tread to sink through the water film and make contact with the pavement). Their tyre/pavement model appears to be suited to applications where comprehensive information from a single tyre at a selected speed and on a selected pavement profile is of interest. It is not thought suited for the crash risk modelling required for setting speed limits.

Ghandour et al (2010) also report on a model for tyre friction, but for application in an electronic driver assistance systems (EDAS). In their control strategy for the EDAS, a new method for estimating the maximum tyre/road friction coefficient is developed that takes into account the instantaneous tyre:road friction (eg due to the road surface being dry, wet, snow-covered or ice-covered). However, their method does not appear applicable to the crash risk modelling required for setting speed limits as it appears intended for incorporation in an EDAS control strategy.

In the paper by Do and Delanne (2004), validation of a model predicting the speed dependency of tyre:road friction is presented. Their model is based on the shape of the friction–speed curve and relies on coupling two models. However, their model seems to require inputs to have a level of detail (eg road surface macro- and micro-texture, rubber relaxation time, wheel slip, tyre tread depth and water layer thickness) which would make application to the crash risk modelling required for setting speed limits inconvenient or impossible.

The thesis by Kosgolla (2012) considers analytical and numerical tools for estimating wet pavement skid resistance. However, these tools appear to require too much computational effort and require too many detailed inputs to be suited to the crash risk modelling required for setting speed limits.

2.1.2.2 Observations

Hall et al (2009) describe their development of a *Guide for pavement friction*. Both micro-texture and macro-texture are considered along with the effect of water reducing a pavement's 'dry' skid resistance. The authors also present results of what appears to be a commendably comprehensive literature review. In this review they reference the paper of Viner et al. who state: 'for single carriageways in the UK shows that crash risk approximately halves as pavement friction doubles over normal ranges'.

Kulakowski (1991) presents a model for skid resistance that is a function of speed. According to Kulakowski, there are several inconsistencies associated with the percent normalised gradient. These inconsistencies are presented, and an alternative parameter called a 'speed constant', which is free of these inconsistencies, is proposed.

Wilson et al (2013) in their NZ Transport Agency research report on the effect of road roughness and test speed on GripTester measurements, state: 'It has been well proven that as the skid resistance of a road surfacing decreases, the number of loss-of-control crashes in wet conditions increases, causing road death and injuries'.

Long et al (2014) in their FHWA report on investigations into the relationship between vehicle crashes and pavement skid resistance conclude: 'the crash risk increases when the skid resistance decreases'.

2.2 Prediction of speed on two-lane rural roads

Praticò and Giunta (2012) present an excellent review of speed prediction models for rural roads before going on to present their own models and correlations with measured speed data. They conclude that 69% of the variance between modelled and measured speed data can be explained by including in the speed model: horizontal curvature, the longitudinal slope, and the length of an element (10 m for New Zealand Road Assessment and Maintenance Management System ((RAMM)) geometry data). It is interesting that crossfall is not among these three variables.

In an earlier paper, Praticò and Giunta (2010) comment that although several models are available in literature for the prediction of operating speeds, they are usually road-specific and transportability is an unsolved issue.

Similarly, Hashim (2011) reviews literature before presenting his own analysis of speed data collected in Egypt. He concludes that a headway of 5s or more results in free flow of traffic. For free flow, the 85th percentile speed is a function of road geometry only (ie vehicle-vehicle interactions do not need to be considered).

Cafiso and Cerni (2011) gathered trial speed data by installing GPS receivers on sample vehicles. This data gathering method, rather than spot speed measurements, is reported to be useful for studying the actual speed profile with regard to acceleration/deceleration behaviour and maximum speed on tangents and curves. From analysing this collected speed data, they developed a free-flow speed model. Variables in the model are horizontal curvature and the vertical grade as weighted values of the preceding and following geometrical features of the alignment with respect to the vehicle position. Therefore, to predict the operating speed, the model takes into account both the preceding and *following* geometrical alignment of the road, as this information is used by drivers to establish their driving task, which is based on memory expectation, vehicle dynamics and visual perception.

Fitzpatrick et al (2000) in their FHWA paper on speed prediction for two-lane rural highways emphasise the importance of design consistency for highway geometry features to assist drivers to have 'no surprises'. Drivers make fewer errors in the vicinity of geometric features that conform to their expectations. A technique to evaluate the consistency of a road design is to evaluate changes in operating speeds. The paper reports on efforts undertaken to predict operating speed for different conditions such as on horizontal, vertical and combined curves; on tangent sections using alignment indices; and on grades prior to or after a horizontal curve. Their model considers both horizontal and vertical curvature and the acceleration or deceleration behaviour as a vehicle moves from one feature to another.

The thesis by Bennett (1994) comments that the undulating terrain which predominates over much of rural New Zealand means traffic operations on two-lane highways are strongly influenced by road geometry. He goes on to develop a free traffic flow speed prediction model that is a function of road geometry.

In another paper originating from New Zealand, Wanty et al (1995) discuss Application of Road Geometry Data Acquisition System data to the New Zealand state highway network. They present a free-flow speed prediction model that is a function of horizontal curvature and cross slope. The report of Cenek et al (2012a) employs the same equation.

2.3 Prediction of speed changes when entering and exiting a curve

Xiao and Tian (2009) collected and analysed experimental data on two lane secondary highways in China. Experimental speed data was collected using radar guns at three locations: (1) the straight immediately prior to a curve, (2) the mid-point of the curve, and (3) the straight immediately following the curve. They found in the analysis of this experimental data that: (1) vehicle speeds prior to-, during- and post-curve follow the pattern of 'slow entry -> maintain stability –> acceleration', (2) the smaller the curve radius, the greater the speed difference prior to the curve and within the curve, (3) vehicle speed before and after the centre of curve follows an approximately quadratic plot shape, (4) the operating speed of vehicles increases with curve radius, and (5) the range of motor cars speed increases is greater than that of medium-sized trucks.

Takahashi and Akamatsu (nd) investigated 69 curves obtained from a Japanese driving database and developed a curve entrance velocity prediction model that is a function of: (1) the mean curvature radius, and (2) the *velocity tendency* of the curve (*velocity tendency* is a characteristic developed by the authors that describes the virtual velocity if the curve were not there).

Koorey et al (2002) reference the report of Bennett (1994) who suggested that generally vehicles decelerate in the first half of a curve, and accelerate in the second half of a curve. In their own experimental work on curves in New Zealand using optical sensors they found that, generally: (1) most vehicles made little change to their speeds through the curve, suggesting that any speed reduction occurred largely before the curve, (2), it was not until the middle curve section that speeds approached the minimum, and (3) there was no marked increase in speed on the final curve portion, suggesting that drivers were generally waiting to leave the curve before accelerating again.

Mark and Marek (2013) discuss considerations when designing horizontal alignment for roadways. These include: (1) compound curves should be used with caution, (2) for compound curves in rural conditions, the radius of the flatter curve should not be more than 50% greater than the radius of the sharper curve, (3) alignment consistency should be sought (eg sharp curves should not follow tangents or a series of flat curves), (4) reverse curves on high-speed facilities should include an intervening tangent section of sufficient length to provide adequate superelevation transition between the curves, and (5) broken-back curves (ie two curves in the same direction connected with a short tangent) should normally not be used as this type of curve is unexpected by drivers and is not pleasing in appearance.

2.4 Vehicle operating costs and air pollution

2.4.1 Models

Chatti and Zaabar (2012) report on research that had the objective of recommending models for estimating the effects of pavement conditions on VOC. The recommended models reflect 2012 vehicle technologies in the US. It was found that the most important cost components affected by roughness are fuel consumption followed by repair and maintenance, then tyre wear.

The VOC and emissions model for the World Road Association (PIARC) as reported by Bennett and Greenwood (2004) appears commendably comprehensive and considers: forces opposing motion; free speeds; effects of volume on traffic flow; fuel consumption; tyre consumption; maintenance and repair costs; utilisation and service life; capital costs; engine oil consumption; travel time, crew and overhead costs; safety; non-motorised transport; work zone effects on traffic and user costs; heavy vehicle trailers; vehicle emissions; noise emissions and energy balance considerations.

A VOC model is reported in section A5 of NZ Transport Agency (2016). Separate VOCs are given for six classes of vehicles ranging from passenger cars to buses. Coefficients are tabulated for cost calculations for speeds up to 120 km/h. Like the PIARC VOC model, the list of VOC components considered by the Transport Agency VOC model seems very comprehensive. Road categories considered are: urban arterial, urban other, rural strategic and rural other.

Tan et al (2012) note that at the time of their paper preparation, Australia did not have well-developed VOC versus roughness relationships. After highlighting this, their paper seeks to address this by: (1) consolidating local and international research, and (2) suggesting options for future Australasian research.

2.4.2 Observations

Olde et al (2005) in a study on freeways in the Netherlands found a 5% average improvement in air quality resulted from a speed limit reduction to 80 km/h from 100 km/h and 120 km/h.

Fildes et al (2005) report on a project aiming to develop a speed setting methodology for Australasia. They mention that [vehicle] speed has ‘associations with operational aspects of transport such as mobility, travel time, fuel use, vehicle operating costs, and emissions’. Further economic considerations are discussed more comprehensively in section 1.5.3 of this reference.

Cameron (2009) in an economic evaluation of a 10 km/h posted speed reduction on 100 km/h and 110 km/h rural Tasmanian roads concluded: ‘The envisaged reduction in the 110 km/h speed limit to 100 km/h on Category 1 (national highway) roads in Tasmania would be economically justified on both the divided and undivided sections under consideration’. Cameron’s economic analysis appears to be commendably detailed and considers the economic impact of the lowering of these speed limits on VOCs. These VOCs were calculated according to the published Austroads model.

Degraeuwe et al (2012) in their computer modelling with the software package VERSIT of the Liege-to-Antwerp motorway in Belgium conclude: (1) a general speed limit of 90 km/h would result in a small decrease in air pollutant emissions, and (2) measures that reduce heavy congestion, without attracting more traffic, can decrease air pollutant emissions, but increase noise emissions.

ECCI (nd) in a study on Scotland roads observes that reducing speeds to 40 mph (from an unspecified speed) is likely to have a positive impact on vehicle emissions and air pollutants, but reducing speeds beyond 40 mph is likely to have a dis-benefit.

The European Environment Agency (nd(a) and nd(b)) in a study of speed limits on European motorways remark that significant fuel savings could be achieved by encouraging drivers to maintain a consistent speed and restrict their speed. In addition, the European Environment Agency (nd(a)) comments that based on simulation results, cutting motorway speed limits from 120 km/h to 110 km/h could deliver fuel savings for passenger cars of 12–18%, assuming smooth driving and 100% compliance with speed limits. However, relaxing these assumptions to a more realistic setting implies a saving of just 2–3%.

European Transport Safety Council (2005) comments that fuel consumption is 30% higher at speeds above 120 km/hour than at speeds at 90 km/hour.

Hickman (2010) states that research in Germany has shown the greater the speed of vehicles in built-up (ie urban) areas, the higher the incidence of acceleration, deceleration and braking, all of which increase air pollution.

WSDOT Library (nd) gives the details of a number of items of literature that address the issue of the variation in emission levels with road speed changes.

2.5 Crashes

2.5.1 Rural roads and roads of unspecified designation

Olde et al (2005) in a study on freeways in the Netherlands found, on average, the total number of crashes decreased by 35% and the total number of injury crashes decreased by 47% with a speed limit reduction to 80 km/h from 100 km/h and 120 km/h. In future years, these researchers expected the speed limit crash reduction benefits to be less because of increasing intensity and congestion.

Arona et al (2014) in a study of a speed limit reduction of 20 km/h on 110 km/h urban and 130 km/h interurban [ie rural?] motorways in the north of France conclude: (1) the reduction results predict a decrease [of unspecified magnitude] in the crash count, and (2) crash reductions for interurban motorways are greater than for their lower-speed urban counterparts.

Helfenstein (1990) in an investigation of when speed limit reductions in Zurich showed an effect, concludes that from inspecting crash data, there were significant effects three months *prior* to the actual speed limit reduction introduction date. According to Helfenstein, a possible explanation of this result may be that the media had informed the public before the speed limit reduction was actually introduced.

Nilsson (1990) considers the effects of a speed limit reduction from 110 km/h to 90 km/h on motorways and main roads (including rural roads) during two months of the summer of 1989. He reports that the traffic safety situation on rural roads improved in relation to the corresponding period in 1988.

Fildes et al (2005) report on a project aiming to develop a speed setting methodology for Australasia to achieve a 10% reduction in the cost of casualty crashes. The results of a literature review into the association between speed and crash risk conclude: ‘The consistent finding was that there existed a strong association between higher speed and more serious crash outcomes, with there also being evidence for an association between higher speed and greater casualty crash involvement’. Any further attempts to summarise the findings of this reference would likely omit key findings for the sake of brevity, so a further summary is not attempted here. Instead, readers are encouraged to read the executive summary of this report.

In a more recent Austroads publication, Jurewicz et al (2014) outline development of model Australian guidelines for setting speed limits at high-risk locations. The model guidelines aim to provide consistent speed limits on roads and intersections in response to high severe crash risk, while minimising the frequency of speed zone changes. Inherent in this paper is the principle that reduced speed increases safety.

Coesol and Rietveld (1998) in their paper report that, in extreme cases, reductions in fatal crashes can be as high as 30% resulting from a speed limit reduction (the magnitude is not specified).

Hillman (1997) in a study of speed limits on UK roads conclude UK speed limits should not exceed 55 mph on motorways, 50 mph on dual-carriageway trunk roads and 40 mph on single-carriageway main roads. If these speed limits were adopted, the corresponding crash rates on such roads would be reduced to 55%, 55% and 80% of their present levels. This study possibly assumes full compliance of vehicle speeds with posted limits. Therefore, these crash rate reductions may be optimistic.

de Pauw et al (2014) in their study of the effect of the 2001 20 km/h speed limit reductions on 90 km/h highways in Flanders (a region in Belgium) state there was a 5% decrease in crash rates after the speed limit restriction. A greater effect was identified in the case of crashes involving serious injuries and fatalities (the decrease for these was 33%). Separate analyses of crashes at intersections and at road sections showed speed limit reductions had the most benefit in reducing crash rates at road sections.

Matírnez et al (2013) state in the abstract to their report that a low-cost 'solution' can reduce traffic speed and crashes. It would be interesting to read detail on this low-cost 'solution'. However, this has not been possible as the full paper does not appear to be available on the internet.

Dutta and Noyce (nd) reviewed literature for a project considering *raising* speed limits on Wisconsin's freeways. They estimate that raising the speed limits from 65 mph to 70 mph may result in an initial increase in speed-related traffic fatalities.

ECCI (nd) in a study on Scotland roads observes that small changes in mean speeds can be expected to result in much larger changes in crash outcomes. In other words, lowering vehicle speeds reduces severe crashes (ie those crashes resulting in severe injuries/deaths) disproportionately in relation to small changes in mean speeds. Keeping traffic flowing at similar speeds is also safer: roads with a small speed differential between the fastest and slowest vehicles are safer than roads with high-speed differential.

The European Environment Agency (nd(a)) in a study of speed limits on European motorways remarks that 'The safety gains from slower driving are also indisputable'.

The European Transport Safety Council (2005) comments that on rural roads, a 1 km/h speed reduction only results in 2% fewer injury crashes.

Ferguson (2005) in her review of papers investigating the effects of speed changes, concludes the overwhelming majority of evidence suggests reductions in speed limits reduce vehicle speeds and crashes; increases in speed limits increase speed, as well as crashes. Although not explicit, Ferguson's review seems to be aimed at those with an interest in speed limits on US interstate highways.

FHWA, T-FHRC (1992) conclude in a before-and-after study on US highways: 'Lowering speed limits below the 50th percentile does not reduce crashes .... Conversely, raising the posted speed limits did not increase speeds or crashes'.

Friedman et al (2009) in their examination of the effects of raising speed limits in the US found a 3.2% increase in road fatalities attributable to the raised speed limits on all road types in the US. The highest increases were on rural interstates (9.1%) and urban interstates (4.0%).

Haney and Weber (1974) in their literature review conclude 'In general, the research literature shows that lowered speed limits reduce ...the number of serious- and fatal-injury crashes'.

Kweon and Kockelman (2005) in their estimates of the safety effects of speed limit changes on high-speed interstate highways and state highways in Washington State found fatal and non-fatal crash rates move differently when traffic levels rise, with non-fatal rates remaining unchanged and fatal rates falling.

Langlotz (1999) in his analysis of NHTSA data to assess the effect of the 1995 raising of posted speed limits in many states of the US concludes: ‘the best available recent data show no effect on traffic safety when interstate speed limits are raised’.

Dealing with the same 1995 speed limit increases in the US, but considering rural interstate highways, Patterson et al (2002) conclude from their statistical analysis that: within a year of the repeal of the national maximum speed limit in the US, the changes in fatalities were less than those one would predict if the speeds had changed by the amount of the speed-limit change.

The RAC Foundation (2013) states: ‘Even small changes to the speed travelled by the driving population lead to large and measurable changes in risk. A 5% increase in mean speeds typically leads to an increase in injury crashes of 10% and an increase in fatal crashes of 20%’.

RoSPA (2017) states: (1) ‘inappropriate speed contributes to around 6% of all injury collisions, 15% of crashes resulting in a serious injury and 26% of collisions which result in a death’, and (2) ‘a 1 mph reduction in average speed would reduce accident frequency by about 3% on the higher speed urban roads and rural single carriageway main roads’.

Cameron (2009) in a report considering a 10 km/h posted speed reduction on 100 km/h and 110 km/h rural Tasmanian roads estimates there would be a 25% reduction in fatal crashes, 15% reduction in serious injury crashes, and nearly 12% reduction in minor injury crashes on roads with the speed limit reductions.

2.5.2 Urban

The following references are thought to have little relevance to the topic of this review, which focuses on the open speed (ie rural) environment but are included as they may be useful to some readers.

Archer et al (2008) in their paper on *urban* speed limits in Australia state: ‘The relationship between vehicle speed, crash risk and crash outcome severity is well established in traffic safety literature. Research shows that reduced speed is likely to bring about a reduction in average travel speed and have a positive impact on both the number of crashes and crash outcome severity. Other secondary benefits are also derived including: reduced fuel and VOC, and significant reductions in vehicle emissions and noise.’

Woolley et al (2000) seek to present evidence quantifying the impacts of lower *urban* speed limits in Australia in terms of: (1) measured speeds and volumes, (2) community attitudes, (3) environmental impacts, (4) travel times, and (5) road safety outcomes.

The European Transport Safety Council (2005) comments that a speed decrease on busy urban roads where there is a lot of slow traffic and large speed differences results in a 6% crash reduction per km/h reduction.

Woolley (2005) in his analysis of crash rates on urban residential streets in Australia after a 10 km/h posted speed limit reduction from 60 km/h to 50 km/h comments after the -10 km/h speed limit change: ‘reductions in the order of 20% in casualty crashes have been observed’.

2.6 Expected lives of road surfaces used on the New Zealand state highway network

Average pavement lives used on New Zealand state highways calculated from RAMM data should be viewed with caution as the two-coat chipseal population is not yet 'mature' (Patrick et al 2014). Accordingly, to avoid this complication, RAMM 'default lives' are used in table 2.1 below in preference to calculated averages. These RAMM 'default lives' are the life values selected by expert practitioners (Patrick et al 2014).

Table 2.1 RAMM 'default lives' in years (NZ Transport Agency 2009, table C3)

Surfacing type	Use 1	Use 2	Use 3	Use 4	Use 5	Use 6	Use 7
	(<100 vpd)	(100–500 vpd)	(500–2,000 vpd)	(2,000–4,000 vpd)	(4,000–10,000 vpd)	(10,000–20,000 vpd)	(>20,000 vpd)
Texturising seals							
Grade 6	6	5	4	3	2	1	Open 1
Grade 5	8	7	6	5	4	3	2
Void fill seals							
Grade 6	6	5	4	3	2	1	1
Grade 5	8	7	6	5	4	3	2
Locking coat seals							
Grade 6	6	5	4	3	2	1	1
Grade 5	8	7	6	5	4	3	2
First coat seals							
Grade 4	3	2	1	1	1	1	1
Grade 3	4	3	2	1	1	1	1
Grade 4/6	6	4	3	2	2	1	1
Grade 3/5	8	6	5	4	3	2	1
Grade 2/4	10	8	6	5	4	3	2
Second coat seals							
Grade 6	6	5	4	3	2	1	1
Grade 5	8	7	6	5	4	3	2
Grade 4	12	10	8	7	6	5	4
Grade 3	14	12	10	9	8	7	6
Grade 2	16	14	12	11	10	9	8
Grade 4/6	14	12	10	9	8	6	4
Grade 3/5	16	14	12	11	10	8	6
Grade 2/4	18	16	14	13	12	10	9
Reseals							
Grade 6	6	5	4	3	2	1	1
Grade 5	8	7	6	5	4	3	2
Grade 4	12	10	8	7	6	5	4
Grade 3	14	12	10	9	8	7	6

Surfacing type	Use 1 (<100 vpd)	Use 2 (100–500 vpd)	Use 3 (500–2,000 vpd)	Use 4 (2,000–4,000 vpd)	Use 5 (4,000–10,000 vpd)	Use 6 (10,000–20,000 vpd)	Use 7 (>20,000 vpd)
Grade 2	16	14	12	11	10	9	8
Grade 4/6	14	12	10	9	8	6	4
Grade 3/5	16	14	12	11	10	8	6
Grade 2/4	18	16	14	13	12	10	9
Slurry seal	8	7	6	5	4	3	2
Open grade emulsion	12	11	10	9	8	7	6
Premium skid	10	9	8	7	6	5	4
Struct. AC	20	20	19	19	18	17	16
Concrete	60	60	50	50	40	40	40
BOLIDT polyurethane	18	16	14	12	11	10	8
Bicouche/sandwich	14	12	10	9	8	6	4
Prime and seal	7	6	5	4	3	2	1

Patrick et al (2013) give information in table 2.2 that serves to supplement table 2.1.

Table 2.2 RAMM 'default lives' in years (excerpt from Patrick et al 2013)

Surfacing type	Use 1 (<100 vpd)	Use 2 (100 – 500 vpd)	Use 3 (500 – 2,000 vpd)	Use 4 (2,000 – 4,000 vpd)	Use 5 (4000 – 10,000 vpd)	Use 6 (10,000 – 20,000 vpd)	Use 7 (>20,000 vpd)
Reseals							
Thin AC	16	15	14	13	12	11	10
OGPA	12	11	10	9	8	7	6
SMA	15	14	12	11	10	8	7

The Seal Life Advisory Group (SLAG) is presently carrying out work to validate and update the estimates of default lives given in tables 2.1 and 2.2 and this is discussed further in section 3.7.4.

2.7 Travel time and traffic flow

2.7.1 Models

Canada's Victoria Transport Policy Institute (VTPI 2013) presents a comprehensive discussion of travel time modelling. The report discusses some aspects of the travel time models developed worldwide (eg in North America, Australia, New Zealand, Europe and the UK) and is useful for those seeking a good overview of travel time modelling and models.

The thesis by Tapani (2005) develops a micro-simulation model for traffic flow on rural highways. The first part of his thesis reviews other traffic flow simulation models.

Bergh and Carlsson (1995) report on the National Swedish Road Administration guidelines on geometric design of rural highways. Their study emphasises the impact of passing lanes on traffic flows. They also address crash levels.

2.7.2 Observations

Olde et al (2005) in a study on freeways in the Netherlands found that an 80 km/h speed limit reduced from 100 km/h or 120 km/h led to an unspecified reduction in the variation in journey time, with the largest decreases being for free flow.

Fildes et al (2005) report on a project aiming to develop a speed setting methodology for Australasia. Considerations for setting speed limits include ‘increased travel times arising from lowered speed limits and the [associated] economic implications’.

Dutschke and Woolley (2009) conclude through model predictions that on undivided rural roads, the increase in travel time is less than that predicted by considering only the allowed speed limit. The model also shows that a driver who desires to maintain a constant travel speed must overtake more often when the speed limit is higher than when it is lower.

Coesol and Rietveld (1998) report that, while speed limit lowering often results in reductions in crashes, on the other hand, travelling time increases.

ECCI (nd) in a study on roads in Scotland observes that changes to journey times are normally the biggest contributor to economic impacts of speed limit changes, but are frequently over-estimated because they assume free-flowing traffic. The economic cost of reducing a speed limit typically peaks when flow is approximately two thirds of capacity.

2.8 Changes in operating speed resulting from speed limit change

2.8.1 Rural roads or roads of unspecified designation

Olde et al (2005) in a study on freeways in the Netherlands found that an 80 km/h speed limit reduced from 100 km/h or 120 km/h led to an unspecified decrease in the average speed of traffic and that the largest decreases were observed in free-flow situations.

Arona et al (2014) in a study of a speed limit reduction of 20 km/h from 130 km/h to 110 km/h on an inter-urban [ie rural?] motorway in the north of France concluded the average operating speed decrease would be 13.7 km/h.

Nilsson (1990) considers the effects of a speed limit reduction from 110 km/h to 90 km/h on motorways and main roads (including rural roads) during two months of the summer of 1989. Nilsson’s report is written in Swedish, so presumably this study was carried out on data for roads in Sweden. Nilsson concludes the actual speed reductions were greatest on motorways, but an overall effect could be noticed not only on the road sections involved but also on other main roads.

Fildes et al (2005) report on an Austroads project aiming to develop a speed setting methodology for Australasia to achieve a 10% reduction in the cost of casualty crashes. Strategies for managing compliance with posted limits include: (1) enforcement, (2) education/advertising, (3) road design and infrastructure, and (4) vehicle design.

Cameron (2009) envisages a 5 km/h actual reduction for a 10 km/h posted speed reduction on 100 km/h and 110 km/h rural Tasmanian roads.

Rossy et al (2012) state previous studies have indicated that on high-volume, high-speed roads a reduction in speed is not commonly attained by reducing the posted speed limits alone.

DeerCrash.ORG (nd) reports the design of the roadway (versus the posted speed limit) has the largest impact on speed. This result is generally supported by past transportation research, and is related to driver expectations, topography and a number of other factors.

ECCI (nd) in a study on Scotland roads observes: (1) reducing a speed limit alone typically results in a change in average speed of as little as a quarter of the change in speed limit, and (2) the use of speed zones with additional physical measures are effective at reducing vehicle speeds.

Climo clearly has much knowledge in the issues surrounding setting speed limits for New Zealand roads (eg Foley and Climo 2013), but frustratingly no sufficiently detailed literature relevant to this review by this author could be found on the internet.

FHWA, T-FHRC (1992) discusses findings of research aimed to determine the effects of raising and lowering posted speed limits on driver behaviour and crashes for rural and urban highways in the US. Results indicate lowering posted speed limits by as much as 20 miles/h (32 km/h), or raising speed limits by as much as 15 mi/h (24 km/h) have little effect on motorists' speeds.

The RAC Foundation (2013) states: 'Meta-analyses show that the speed limit alone by 10 km/h leads to a decrease in mean speeds of 3–4 km/h…'.

Hashim (2011) references Harkey et al (1990) and reports they analysed collected USA speed data and found, on average, 70% of motorists exceed the posted speed limit.

2.8.2 Urban roads

The following references are thought to have little relevance to the topic of this review, which focuses on the open speed (ie rural) environment, but are included as they may be useful to some readers.

Excell (2005) presents the results of market research on public opinion undertaken to gauge the acceptability of speed limit reductions on roads in Australia. A key finding of this public opinion survey was that between 71% and 79% either 'opposed' or 'strongly opposed' the reduction of speed limits from 60 km/h to 50 km/h. However, this finding is thought to be of little relevance to this review as it pertains only to 60 km/h urban limits and deals only with the results of survey opinions.

Taylor (2000) presents study results for a computer-simulated speed limit reduction in several urban networks. Conclusions were: (1) journey speeds in these urban networks were considerably less than the set speed limits; (2) differences in overall travel speeds and journey times were much less than the differences in the speed limits themselves; (3) signal coordination offered significant advantages for delays and traffic progression, except at higher congestion levels where some oversaturation was experienced; and (4) an argument for public acceptance of lower speed limits could be based on an improvement in traffic progression and quality of traffic flow possible under the lower speed limit regimes.

McCoy et al (1993) report on methods of establishing speed zones on Nebraska Department of Roads state highways passing through urban areas and state: 'This use of the test-car speed runs is not recommended; because it is inherently biased toward the lower speed limits requested by local officials and serves no useful purpose in the determination of the most appropriate speed limit'.

Dreesen and Nuyts (2006) in their study of lowering the speed limit in Flanders (Belgium) found in a school zone with a 70 km/h speed limit where a permanent 30 km/h zone was installed, the daily mean speed decreased significantly (by 20.3 km/h) for the first few months after installation. The mean speed decreases between 8am and 9am were 24.8 km/h. However, some months later, these speed reduction effects were much less pronounced.

2.9 Crash prediction models

The review in this section below is an updated and supplemented version of that in Cenek et al (2012a).

2.9.1 Post-2004 Australasian crash prediction models

Crash prediction models have been developed in New Zealand for application in New Zealand (eg Cenek et al 2012a) and in Australia for application in Australia. A summary of pre-2009 models developed in both countries has been given by Turner and Wood (2009a).

Post this Turner and Wood summary, a crash risk prediction model was developed by Austroads (Jurewicz et al 2014 or Jurewicz 2013) called the Australian National Risk Assessment Model (ANRAM). This model is intended to assist Australian road agencies to identify fatal and serious injury crash risk across all parts of their road network (ie rural roads, urban roads and intersections). This model seems very comprehensive, but may be better suited to detailed crash analysis at specific locations than for curve crash studies on reasonably long lengths of rural undivided carriageways. The model of Davies as described in Cenek et al (2012a) may be better suited for this. A key feature of the model of Davies relevant to rural curve crash risk modelling on rural highways is the inclusion of the difference between the curve approach speed and the curve speed. The ANRAM model does not appear to include this information.

2.9.2 Post-2004 New Zealand crash prediction models

When first published, the Poisson regression crash prediction model specifically developed for the New Zealand state highway network outlined by Davies et al (2005) was notable for its sophistication; it utilised traffic flow, road geometry and road condition as inputs and predicted the number of fatal/injury crashes on rural road networks with any moisture level, and the number of fatal/injury crashes on wet rural road networks. When it was released, this model, as far as the report authors are aware, was the sole Australasian crash prediction model to consider road condition. In their paper, as well as detailing the model, Cenek and Davies (2006) presented the results of a case study where the model was applied to the Karangahake Gorge in New Zealand.

Recent developments of this model have taken place. These developments include incorporating the difference between the curve approach speed and the curve speed (Cenek et al 2012a).

Turner et al (2003) provide details on other crash prediction models developed from reported injury crash data and traffic counts in New Zealand for major crash types and total crashes. Possible applications of the models were discussed, including economic evaluation, developing performance measures, assessing safety management systems and optimisation of network flow patterns to improve safety.

Turner et al (2004) discuss a model that focuses on predicting crash risk as a function of roadside hazard. Such models were claimed by the authors to be useful in targeting resources to remove roadside hazards.

Turner et al (2006a) discuss crash model predictions for cyclists and pedestrians and report a noticeable ‘safety in numbers’ effect.

Turner et al (2006b) note that crash occurrence is typically low at rural priority-controlled intersections, compared with priority-controlled urban intersections, due to low traffic volumes. They go on to discuss the production of crash prediction models for rural priority-controlled intersections based on traffic volume, sight distance, approach speed and geometric design. The paper also outlines some of the more important statistical methods used to assess the quality of the models produced.

Turner et al (2007a) observe that over the previous 15 years (from 2007), a multitude of crash prediction models were developed for rural roads, urban intersections and mid-block sections in New Zealand. They emphasise there is a growing need for more-comprehensive models similar to those developed internationally (eg in the US and Europe).

Partially addressing this perceived need, Turner et al (2007b) present New Zealand-based comparisons of selected crash prediction models from New Zealand, the US, Sweden and Australia. Their results suggest it is possible to transfer models from one country to the next, but there are a number of differences between countries that need to be accounted for.

In their technical note, Turner et al (2007c) identify the important attributes required in a road safety evaluation tool for New Zealand conditions. They review overseas software, considered whether it could be applied in New Zealand, and what local software could be required. They place particular emphasis on the issues involved in New Zealand adopting the Australian package SIDRA.

Notably, crash modelling work by Turner and Tate was incorporated in the EEM.

Turner et al (2008) focus on crash prediction models for intersections and note a significant proportion of urban crashes occur at traffic signals, and many of the 'black spots' in both Australian and New Zealand cities coincide with high-volume and/or high-speed traffic signals. These crash prediction models are thought to have enabled a better understanding of the impact of various factors on safety to be quantified. Turner et al (2009) note management of speed is considered an important safety issue at roundabouts. They discuss crash predictions for pedestrian-versus-motor-vehicle and cyclist-versus-motor-vehicle crash types.

Turner and Wood (2009a) present an overview of the statistical methods they used in their development of crash prediction models, and present many key findings from New Zealand and Australian crash prediction models.

In another publication, Turner and Wood (2009b) summarise a crash prediction model for intersections. Before describing this model they state that a large number of crash prediction models have been developed in New Zealand for different road elements and speed limits. Such models are deemed to have potential use, among other things, to predict the reduction in crashes that might result from an engineering improvement.

In the final publication included here by these two authors, Turner and Wood (2009c) appear to have covered the same information that was in a previous paper (2009a), presenting a literature review of New Zealand and Australian crash risk models and focusing on the statistical methods used in their predictive models.

In a PowerPoint presentation, Turner (nd) overviewed New Zealand land transport organisations, New Zealand and US crash trends, crash prediction models, and the application of crash prediction models in economic evaluation.

Roozenburg and Turner (2005) state that Beca, Carter, Hollings and Ferner Ltd have developed a number of crash prediction models for crashes at signalised intersections. Non-flow variables such as intersection geometry and signal phasing are concluded to be important predictor variables. They present and discuss in detail those models developed for signalised intersections in New Zealand. A predecessor of such models was included in appendix 6 of Transfund NZ's now-superseded *Project evaluation manual* (Transfund NZ 1997).

Harper and Dunn (2005) detail findings of research to develop more-advanced urban roundabout crash prediction models. Crash prediction model forms and findings from studies in the UK, Australia and New Zealand are briefly outlined. The authors comment that a considerable database has been collated, including traffic movement volumes, geometric site characteristics and crash data for 95 urban roundabouts throughout New Zealand. Employing this database, advanced conflicting-flow crash prediction models have been developed to predict major vehicle crash types on urban roundabouts in New Zealand in relation to traffic volumes and geometric variables.

Koorey (2006 and 2010) notes that crash prediction models are an increasing feature of rural highway design practice internationally. Koorey considers the issues involved with New Zealand using the Interactive Highway Safety Design Model (IHSDM) (FHWA 2006). For an excellent case study of the application of the IHSDM, refer to Bansen and Passetti (2005).

2.9.3 Post-2004 Australian crash prediction models

Many of the Australian-developed crash prediction models, or models applied to parts of Australia, appear to be co-authored or authored by Blair Turner, although there are some notable exceptions to this generalisation.

The recent report of Jurewicz et al (2014) is focused on setting speed limits at high-risk locations in Australia. However, part of their paper presents a reasonably comprehensive review of many crash risk models. Their report is worthwhile consulting for this review alone.

Affum and Goudens (2008) discuss the application of Australia's NetRisk Road Network Safety Assessment Tool as they envisage it applying to the North Coast Hinterland District in Australia.

Bobevski et al (2007) discuss the application of generalised linear models of road trauma outcomes, to assess the safety benefits of countermeasures in Victorian roads in Australia from 1998 to 2003. They conclude that generalised linear models of crash outcomes as a function of potential explanatory factors need realistic assumptions to be made about viable functional forms connecting each factor and the outcomes.

Cossens and Cairney (2008) summarise literature covering crash risk and road characteristics. Skid resistance is concluded to be the best-established of the road surface condition characteristics to affect crash risk. This conclusion is in agreement with the crash prediction model of Cenek and Davies (2006).

McInerney et al (2008) focus on risk maps and the AusRAP star ratings initiative for state highways in Australia that was launched by the Australian Automobile Association in October 2006, whereby roads are rated with between one and five stars according to their crash risk, based on road design elements that are known to affect crash risk. The analysis provides a strong indication of the improvement in crash costs that could be expected as a road network improved from a two-star, to three-star, to four-star, and ultimately a five-star road.

Prinsloo and Chee (2005) provide details of a project undertaken for the Roads and Traffic Authority, New South Wales, Australia. They define the approach and method for the derivation and computation of rural road crash rates.

Prinsloo and Goudanas (2003) present the tools and results achieved from the prediction of the safety performance of rural highways in New South Wales, Australia. The crash rate prediction model setup consists of a series of base models relying on a plethora of roadway parameters. Notably, road condition variables does not appear to be among them.

Blair Turner has published very widely on the Australian Road Research Board's (ARRB's) NetRisk Manager safety initiative (eg Turner 2007, 2008a and 2008b). Among these publications is the paper of Turner and Jurewicz (2008) who, in their PowerPoint presentation, give an overview of ARRB's activities, including road safety improvements. They mention AusRAP and NetRisk as two of these initiatives.

2.9.4 Selected international crash prediction models

While some of the following references do not address crash prediction models explicitly, they are included in this review as they cover some of the important risk factors for crashes.

Chen et al (2006) discuss a non-country-specific hypothetical conceptual framework for a system whereby in-vehicle sensors and computation algorithms would warn the driver of places where the crash risk was relatively high. This approach was judged by the paper's authors to give the driver sufficient time to react promptly, and would potentially promote safe driving and decrease curve-related injuries and fatalities.

Easa and You (2009) describe crash prediction models developed using Washington State road data collected from 2002 through to 2005. In total, the authors developed five statistical models for different combinations of three-dimensional alignment (eg curve on crest).

Elvik (2008) compares five techniques for identifying locations with a high expected number of crashes. These techniques were tested and evaluated by using data for Norwegian roads. It was concluded that hazardous road locations were most reliably identified by the empirical Bayes technique.

Hildebrand et al (2008) presents comparisons of crash prediction estimates for three models with actual crash data for sections of rural two-lane arterial highways in the province of New Brunswick, Canada. All three models were found to overestimate actual crashes. Notably, none of the models appeared to include any road surface condition variables.

In a US National Highway Traffic Safety Administration report, Liu and Subramanian (2009) conclude that the significant factors relating to the high risk of fatal single-vehicle run-off-road crashes are driver sleep deprivation, alcohol use, roadway alignment with curves, speeding and adverse weather.

Montella (2010) compares and evaluates various black-spot identification methods for allocating resources to improve the safety management of roads. He uses data from Italian roads and concludes the Empirical Bayes estimate of total crash frequency performed the most consistently of the methods evaluated.

Montella et al (2008) described the development of a crash prediction model for the rural motorway between Naples and Canona in Italy (the A16). Notably, the model did not appear to include any road surface condition variables.

In their National Cooperative Highway Research Program (NCHRP), Pigman and Agent (2007) sought to formalise US Department of Transport (DOT) reconstruction activities and concluded that very few state DOTs conducted crash reconstructions on a routine basis.

2.10 Conclusions

Based partly on the preceding literature review, the following models are recommended for investigating the effect of changes in route operating speed on crash risk, VOC and travel time.

2.10.1 Crash risk

The report of Cenek et al (2012a) is the only reference that could be found in the literature search for a rural crash prediction model that considers the difference between curve approach speed and curve speed. The model appears to be particularly convenient for analysing reasonably long lengths of state highway as it does not rely on external input from separate traffic flow simulation software. Given these two facts, it appears the best choice for the crash risk prediction modelling required to assess trade-offs between reduced operating speed and crash risk on sections of rural state highway.

2.10.2 Vehicle operating costs

The VOC model outlined in appendix A5 of the EEM is commendably comprehensive and so is a good choice for quantifying the impact of changes in operating speed on VOC.

2.10.3 Skid resistance deterioration

Skid resistance performance deteriorates with cumulative axle passes and not with time alone. According to Cenek et al (2012b), the variation of SCRIM coefficient with ADT is approximately '-0.004 per 100,000 vehicle passes' (refer Cenek et al 2012b, p22).

2.10.4 Travel time

A good choice for the travel time modelling required to quantify the impact of changes in route operating speed is to combine 10 m estimates of free-flow speed calculated using the model presented in Wanty et al (1995) with the analysis procedures detailed in appendices A2 and A4 of the EEM. This recommendation is based on:

- the ease with the free-flow speed model can be applied, as it utilises horizontal curvature, gradient and cross slope data that is readily accessible through the RAMM geometry table
- the model's extensive validation under New Zealand conditions (refer Koorey et al 2002)
- the model's use in a number of curve safety management initiatives undertaken by the Transport Agency (refer Cenek and Davies 2006 and Cenek et al 2011).

3 Analysis framework

It was decided to base the proposed analysis framework for investigating operating speed/road surface skid resistance trade-offs primarily around New Zealand work. This decision was guided by:

- the findings of the literature review, which showed the New Zealand-derived curve risk models and the procedures contained in the EEM for quantifying crash, VOC, CO_2 emissions and travel time costs to be among the most advanced and comprehensive
- ready availability and accessibility of required input data
- consistency with economic evaluation procedures employed by the Transport Agency when choosing between activity alternatives.

Therefore, the analysis framework comprised the following six steps:

1. Extraction of required input data from the RAMM geometry and curve context tables for the rural state highway route of interest.
2. Application of the curve crash model used to generate predicted collective and personal risks tabulated in the RAMM curve context table (refer Cenek et al 2012a) to identify the operational speed to the closest 5 km/h that for a skid resistance value of 0.4 equilibrium SCRIM coefficient (ESC) achieves the same or lower level of cumulative curve crash risk over the route of interest than if the skid resistance value at each curve with a horizontal radius of curvature 400 m or less was equal to the assigned IL. Note that a skid resistance value of 0.4 ESC is at the lower end of what can be expected for a seal constructed from natural aggregates commonly used in New Zealand in road surfacings.
3. Calculation of the 10 m averaged 85th percentile travel speed over all sections of the route with a horizontal radius of curvature equal or greater than 800m, ie over the length of the route considered to have a straight alignment.
4. Calculation of travel time disbenefit resulting from the longer time needed to travel the route because of the lowered speed limit as per appendices 2 and 4 of the EEM.
5. Calculation of social crash cost, VOC, CO_2 emissions and longer seal life benefits resulting from lowering of the speed limit as per appendices 5, 6 and 9 of the EEM.
6. Performing a check to see if the combined benefits are greater or equal in value to the disbenefits.

Each of these six steps is expanded on below.

3.1 Input data from RAMM

All the input data required to perform the analysis is available in RAMM. The tables and associated fields that need to be accessed are as follows:

- carriageway table:
  - road ID, road name, displacement, width, urban/rural classification, and ADT. Data from this table is used for determining lane widths needed for estimating crash rates for rural two-lane mid-blocks and also transitions from rural to urban speed environments.

- sign table:
  - roadID, road name, location, class ('regulatory general') and type ('speed limit xx km/h'). Data from this table is used to confirm existing speed limits that apply to the rural section of state highway being analysed and their start/end locations.
- geometry table:
  - road ID, road name, start, end, lane, survey, gradient, crossfall, HCurvature (ie horizontal curvature). Data from this table is used to calculate the 10 m average 85th percentile speeds and identify straight road sections along the route where operating speeds will be reduced.
- curve context table:
  - road ID, road name, start, end, curve radius, grad inc (ie gradient in increasing direction), grad dec (ie gradient in decreasing direction), approach speed increasing, approach speed decreasing, curve speed, AADT, recommended IL, estimated personal risk, estimated collective risk. This table is used to locate the curves with a horizontal radius of curvature 400 m or less and also to identify the operational speed required to achieve the same cumulative crash risk as if all the curves had skid resistance levels that were at the recommended IL.
- high-speed roughness table:
  - road ID, road name, start, end, lane, survey, vehicle speed. The SCRIM survey vehicle speed data from this table is used to moderate the theoretically calculated 10 m average, 85th percentile speeds.

3.2 Curve crash risk investigations

3.2.1 Curve crash risk model

The curve crash risk model used to generate the estimates of personal and collective risks tabulated in RAMM's curve risk table is also used to investigate the impact of both increasing the skid resistance from the default value of 0.4 ESC to the recommended IL value for the curves of interest and reductions in operating speed from the default rural road speed limit value of 100 km/h to 50 km/h in increments of 5 km/h.

The derivation of this model is detailed in Cenek et al (2012a, appendix G) along with the process employed for identifying curves on rural state highways using 10 m horizontal curvature data held in RAMM's geometry table.

The model assumes each side of each curve can generate crashes at the rate (per year) according to the following relationship:

$$a \times L_1 \times \exp(L_2) \qquad \text{(Equation 3.1)}$$

where: a = ADT

L_1 and L_2 are linear combinations of transforms of the road characteristics as follows:

For L_1 a constant, square root of curve length (sqrt_lengthR)

For L_2 OOCC (ie difference between the approach and curve speeds)
curve speed (AS)
skid resistance (SCRIM ESC)
approach gradient (gradient_app)
log10(ADT)
year
NZ Transport Agency administration region.

The following equation is used to calculate the overall number of crashes per 100 million vehicles passing through the curve for the side of the road of interest:

$$\frac{10^8}{365} \times L_1 \times \exp(L_2) \qquad \text{(Equation 3.2)}$$

The overall personal risk associated with a particular curve is obtained by averaging the crash rate calculated from the above equation for each side of the road.

Particular points to note when using the curve crash risk model to investigate skid resistance/operating speed trade-offs are as follows:

1 The quantities being modelled are personal risk in units of 'all' casualty crashes per 100 million vehicles entering the curve and collective risk in units of annual number of 'all' casualty crashes per curve.

2 The regions variable in the model refers to Transport Agency administration regions as at 2002 where R1 = Auckland (0); R2 = Hamilton (0.13161); R3 = Napier (0.38803); R4 = Whanganui (0.40065); R5 = Wellington (0.28962); R6 = Christchurch (0.33949); and R7 = Dunedin (0.43579). (Bracketed value is corresponding model input value.)

3 The year variable is fixed to a value of 0.25136, corresponding to year 2002.

4 The default skid resistance value is 0.4 ESC and this has been used in generating the estimated personal and collective risk values in the RAMM curve context table and is the recommended value for investigating the impact of changes in operating speed.

5 The approach speed in the increasing lane is the average of the 10 m 85th percentile speeds from 500m prior to the start of the curve to the start of the curve. The approach speed in the decreasing lane is the average of the 10 m 85th percentile speeds from 500m prior to the end of the curve to the end of the curve.

6 A number of model input variables have limits placed on the values they can take, these being:

   a 20 km/h ≤ curve speed ≤ 110 km/h

   b 0 m ≤ curve length ≤ 800 m

   c 100 vpd ≤ ADT ≤ 50,000 vpd

   d -15% ≤ grade ≤ 15%

   e 0 km/h ≤ OOCC ≤ 50 km/h where OOCC is difference between the approach and curve speeds.

Whenever a variable value falls outside the limit, it is set to the limit.

An illustrative application of the model using the following input values is provided in table 3.1.

year	2002
region	R2
OOCC	30 km/h
AS	80 km/h
SCRIM	0.4 ESC
ADT	1,000 veh/d
gradient	0%
length	100 m

Table 3.1 Example application of curve crash risk model

Variable	Value of input variable	Processed value of input variable	Model coefficient	Product (value × coefficient)
L_1				
constant		1	1.77E-05	1.77E-05
(sqrt(lengthR)-15)**1	100	-5	1.61E-06	-8.04E-06
(sqrt(lengthR)-15)**2	100	25	6.84E-09	1.71E-07
				∑ = 9.84E-06
L_2				
year:2002		1	0.25136	2.51E-01
region:R2		1	0.13161	1.32E-01
(OOCC-30.0)**1	30	0	0.04387	0.00E+00
(OOCC-30.0)**2	30	0	0.00039	0.00E+00
(OOCC-30.0)**3	30	0	-1.24E-05	0.00E+00
(AS-50.0)**1	80	30	0.01570	4.71E-01
(AS-50.0)**2	80	900	-9.43E-05	-8.48E-02
(AS-50.0)**3	80	27,000	-9.87E-07	-2.66E-02
(SCRIM-0.5)**1	0.4	-0.1	-2.17050	2.17E-01
(SCRIM-0.5)**2	0.4	0.01	-1.14390	-1.14E-02
(log10_ADT-3.0)**1	1000	0	-0.05904	0.00E+00
(log10_ADT-3.0)**2	1000	0	-0.17294	0.00E+00
(log10_ADT-3.0)**3	1000	0	-0.08039	0.00E+00
(gradient_app)**1	0	0	-0.02628	0.00E+00
(gradient_app)**2	0	0	0.00035	0.00E+00
				∑ = 9.70E-01
			Personal risk:	7.11[(a)]
			Collective risk:	0.026[(b)]

[(a)] From equation 3.2
[(b)] From equation 3.1

3.2.2 Target crash risk

The target personal and collective curve crash risk for a rural state highway route of interest is calculated as follows:

- The start and end location and modelling input data for all high crash risk curves along the route, ie curves with a horizontal radius equal or less than 400 m are obtained from RAMM by selecting the state highway of interest and extracting the data fields detailed in section 3.1 from the curve context table between the start and end locations of the route.
- Equations 3.1 and 3.2 are applied to each high crash risk curve with the default SCRIM skid resistance value of 0.4 ESC replaced by the recommended IL, to calculate the individual target personal and collective risk values for each curve. The recommend IL is provided in the RAMM curve context table as well as all other model inputs, ie OOCC, curve speed, curve length, ADT, gradient.

- These individual target risk values are summed to give the target personal and collective curve crash risks for the route that in turn will be used to identify the optimal operating speed for the route.

3.2.3 Impact of approach speed

To determine what operating speed is needed to achieve the target crash risk, the maximum curve approach speed for all the high-risk curves along the rural state highway route of interest is reduced from 100 km/h to 50 km/h in increments of 5 km/h and the individual curve and route crash risks are calculated for the revised OOCC value, and a SCRIM skid resistance value of 0.4 ESC, this being representative of roading aggregates commonly used in New Zealand (refer Cenek et al 2012b).

3.2.4 Determination of desired operating speed

Selected curve crash statistics for each approach speed investigated are compared with the target crash statistics to identify which capped approach speed results in the best match as determined by the minimum sum of the absolute differences.

The crash statistics considered are the predicted maximum, average and median of the crash rates and crash numbers calculated for each of the high-risk curves along the route plus the predicted total annual number of curve crashes for the route, which is just the summation of each high-risk curve's predicted annual crash number.

Results of the Ministry of Transport's annual survey of vehicle speeds (MoT 2015) show that for cars, 85th percentile open road free speeds have been trending downwards since 1995 and as at 2015 were 101 km/h. Similarly, this downward trend has been observed for rigid and articulated trucks, the corresponding 2015 85th percentile open road free speeds being 93 km/h for both truck types.

The 2015 85th percentile open road speeds are very close to the open road speed limits of 100 km/h for cars and 90 km/h for all heavy vehicle except school buses, which are limited to 80 km/h. Therefore, in the absence of any other New Zealand specific information, it is reasonable to assume that for rural state highways the posted speed limit equates to the 85th percentile free speed.

On this basis, the speed limit should be reduced from 100 km/h to as close as possible to the capped approach speed that results in the closest match to the target crash statistics. This is because all the approach and curve speeds used in the curve crash risk modelling are based on the theoretically derived 10 m 85th percentile speeds.

With reference to *Land Transport Rule 5400 – Setting of Speed Limits 2003* (MoT 2003), the lowered speed limit must conform to the following three requirements:

1 The speed limit must be one of the following:

   a 20 km/h

   b 30 km/h

   c 40 km/h

   d 50 km/h

   e 60 km/h

   f 70 km/h

   g 80 km/h

   h 100 km/h.

2 The minimum length of state highway the lowered speed limit must apply to unless this requirement is impractical for that road is as follows:

Speed limit (km/h)	Minimum length (m)
50	500
60	500
70	500
80	800
100	2,000

3 The point at which a speed limit changes must be at, or close to, a point of significant change in the roadside development or the road environment.

3.3 Calculation of 85th percentile speeds

A critical element of the analysis framework is the calculation of the free speed profile over the length of rural state highway of interest.

Previous research (Wanty et al 1995) has shown the 85th percentile speed can be reasonably determined by inputting 10 m radius and crossfall data from the geometry table in the Transport Agency's RAMM database into the formula below:

$$S_{85}(km/h) = -\left(\frac{107.95}{H}\right) + \sqrt{\frac{107.95^2}{H} + \left(\frac{127{,}000}{H}\right) \times \left(0.3 + \frac{X}{100}\right)} \quad \text{(Equation 3.3)}$$

where: S_{85} = 85th percentile speed (km/h), capped at maximum of 100 km/h

X = crossfall (%)

H = absolute curvature (rad/km) = 1,000/R

R = horizontal radius of curvature (m)

Equation 3.3 expects the sign of X to be the same as the sign of R. Whenever this is not the case, the road is tilting in the opposite direction from which it should tilt to help offset the transverse forces developed as a vehicle goes around a curve (ie adverse crossfall). In such situations the crossfall needs to be set to 0% in order to apply equation 3.3.

Equation 3.3 gives very high values for speed on straight road sections so the 85th percentile speed on rural sections of state highway is capped at 100 km/h.

The following limits were placed on the 85th percentile speed values calculated by equation 3.3 to better approximate actual changes in speed that a vehicle will make.

3.3.1 Limiting acceleration

Acceleration of a vehicle is higher in initial gears (low speed) and it reduces as the speed increases. From a study of speed and acceleration characteristics of different types of vehicle under mixed traffic conditions, Mehar et al (2013) present the following model for estimating average acceleration:

$$a_{av}(m/s^2) = 1.65e^{-0.04V} \quad \text{(Equation 3.4)}$$

where: a_{av} = average acceleration, m/s^2

V = speed of vehicle, m/s

Equation 3.4 implies that the average vehicle acceleration at 50 km/h is 0.95 m/s^2, whereas at 80 km/h it reduces to 0.68 m/s^2.

Therefore, the maximum increase in speed over a 10 m section of state highway is as follows:

$$\Delta S_{85}(km/h) = \left(3.6 \times \sqrt{\left(\frac{S_{85(i-1)}}{3.6}\right)^2 + 2 \times 10 \times 1.65e^{-0.4(\frac{S_{85(i-1)}}{3.6})}}\right) - S_{85(i-1)} \quad \text{(Equation 3.5)}$$

where: $S_{85(i-1)}$ = the 85th percentile speed of the (i-1)[th] 10 m section of state highway, km/h

S_{85i} = the 85th percentile speed of the I[th]10 m section of state highway, km/h

ΔS_{85} = S_{85i}-$S_{85(i-1)}$

3.3.2 Limiting deceleration

In a deceleration manoeuvre, a vehicle's deceleration rate initially increases, attains a maximum and decreases afterwards. Maurya and Bokare (2012), in a study of deceleration characteristics of 110 cars traversing 1.5 km stretch of highway, derived a second order polynomial model with the following form to describe the observed deceleration-speed behaviour:

$$d_{av}(m/s^2) = -0.005V^2 + 0.154V + 0.493 \quad \text{(Equation 3.6)}$$

where: d_{av} = average deceleration, m/s^2

V = speed of vehicle, m/s

Equation 3.6 implies that the average vehicle deceleration at 50 km/h is 1.67 m/s^2, whereas at 80 km/h it reduces to 1.45 m/s^2. These deceleration values are about twice the acceleration values given by equation 3.4 but comparable with Bennett and Dunn's (1995) study conducted on a New Zealand freeway.

The maximum decrease in speed over a 10 m section of state highway is as follows:

$$\Delta S_{85}(km/h) = \left(3.6 \times \sqrt{\left(\frac{S_{85(1-I)}}{3.6}\right)^2 - 2 \times 10 \times (-0.005\left(\frac{S_{85(i-1)}}{3.6}\right)^2 + 0.154(\frac{S_{85(i-1)}}{3.6}) + 0.493)}\right) - S_{85(i-1)}$$

(Equation 3.7)

where: $S_{85(i-1)}$ = the 85th percentile speed of the preceding 10 m section of state highway

S_{85i} = the 85th percentile speed of the i[th]10 m section of state highway, km/h

ΔS_{85} = S_{85i}-$S_{85(i-1)}$

3.3.3 Gradient effects

Gradient effects can also be incorporated into the calculation of the 85th percentile speed using the following simple formula derived from Bennett (1994) to limit speeds on uphill sections of state highway:

$$S_{85i}(km/h) \le 125 - 5 \times G \quad \text{(Equation 3.8)}$$

Where: S_{85i} = the 85 percentile speed of the i[th]10 m section of state highway, km/h

G = uphill gradient (%)

Equation 3.8 helps dampen speeds on uphill slopes. For example, on an 8% uphill grade, the 85th percentile speed cannot exceed 85 km/h.

3.4 Conversion of 85th percentile speed to mean speed

The majority of vehicle speed-based procedures in the EEM for estimating benefits and costs are based on mean traffic speeds rather than the 85th percentile speeds. New Zealand-specific travel speed data reported in Koorey et al (2002) and the road user behaviour survey spreadsheet[1], accessed through the Ministry of Transport's 'Research' web page, were used to derive relationships between 85th percentile speeds and mean speeds for curves and straight road sections, respectively. The resulting conversions are:

For horizontal radius of curvature ≤ 400m $S_{mean} = 0.8951 \times S_{85}$ (Equation 3.9)

For horizontal radius of curvature > 400m $S_{mean} = 0.000694 \times S_{85}^2 + 0.878 \times S_{85}$ for $S_{85} \geq 50\ km/h$ (Equation 3.10)

where: S_{mean} = mean travel speed (km/h)

S_{85} = the 85th percentile travel speed (km/h)

With reference to equations 3.9 and 3.10, the following points are of note:

- The difference between mean and 85th percentile speed is greater on curves than straight road sections.
- The ratio of mean to 85th percentile speed reduces with reducing posted speed limit (ie 0.916 at 50 km/h cf 0.948 at 100 km/h) whereas the ratio for curves is invariant with design curve speed.

3.5 Calculation of route mean speed

The mean speed over the portion of rural state highway of interest is calculated using the following formula from section A2.9 of the EEM:

$$S_{\text{mean\_route}}(km/h) = \frac{N}{\left[\frac{1}{S_{\text{mean}(1)}} + \frac{1}{S_{\text{mean}(2)}} + \frac{1}{S_{\text{mean}(3)}} + \cdots + \frac{1}{S_{\text{mean}(N)}}\right]} \quad \text{(Equation 3.11)}$$

where: $S_{mean(i)}$ = the 85th percentile speed of the i^th 10 m section of state highway calculated using the procedure outlined in section 3.3, converted to mean speed using either equation 3.9 or 3.10 depending on the horizontal curvature of the 10 m section

N = total number of 10 m sections comprising the portion of state highway of interest.

3.6 Travel time disbenefits

A consequence of lowering the speed limit of a portion of rural state highway to mitigate the increased crash risk associated with the provision of lower levels of skid resistance is an increase in the time needed to travel this portion. This increase in travel time is valued as follows:

1 The time taken to travel the route of interest is calculated using the following equation:

$$RTT = \sum_{i=1}^{i=N} \frac{10}{\left(\frac{S_{\text{mean}(i)}}{3.6}\right) \times 60} \quad \text{(Equation 3.12)}$$

where: RTT = route travel time (minutes)

$S_{mean(i)}$ = the mean speed of the i^th 10 m section of state highway calculated using the procedure outlined in section 3.5.

N = total number of 10 m sections comprising the portion of state highway of interest.

[1] www.transport.govt.nz/research/roadcrashstatistics/motorvehiclecrashesinnewzealand/motor-vehicle-crashes-in-new-zealand-2015/, accessed 4 May 2017

2 The increase in travel time is the difference between RTT calculated with $S_{mean(i)}$ capped at the existing speed limit and with $S_{mean(i)}$ capped at the lowered speed limit ie:

$$\Delta RTT = RRT_{OSL} - RRT_{LSL} \qquad \text{(Equation 3.13)}$$

where: ΔRTT = change in route travel time (minutes), calculated for the increasing and decreasing directions separately

RRT_{OSL} = route travel time with $S_{85(i)}$ capped at original speed limit

RRT_{LSL} = route travel time with $S_{85(i)}$ capped at lowered speed limit.

3 The average change in travel time, ΔRTT_{av}, is the average of ΔRTT calculated for the increasing and decreasing directions, ie:

$$\Delta RTT_{av} = \frac{(\Delta RRT_{Increasing\ Dirn} + \Delta RRT_{Decreasing\ Dirn})}{2} \qquad \text{(Equation 3.14)}$$

4 The increased travel time cost is:

$$Travel\ Time\ Cost\ (\$) = ADT \times BVT \times UF \times \left(\frac{\Delta RTT_{av}}{60}\right) \qquad \text{(Equation 3.15)}$$

where: ADT = average daily traffic over the route of interest

BVT = the base value of time ($/h) which from table A4.3 of the EEM equals $23.25c for rural strategic all periods and $22.72c for rural other all periods, these being values at July 2002

UF = July 2015 update factor, which equals 1.44 (refer table A12.2 of the EEM)

ΔRTT_{av} = lane averaged change in travel time (minutes).

3.7 Benefits

3.7.1 Crash cost

The costs used to value changes in crash numbers and crash severity resulting from a lowering of the rural speed limit to mitigate increased crash risk associated with managing skid resistance on curves to a readily achievable level of 0.4 ESC rather than the recommended IL are given in table 3.2.

Table 3.2 Cost per reported injury crash (after table A6.5(a) of the EEM

Crash site/type	Speed limit area ($000, May 2015)			
	50 km/h	70 km/h	100 km/h (near rural)	100 km/h (remote rural)
Mid-block crashes	275	435	585	850

Lowering of the speed limit impacts on rural road crash numbers and crash severity as follows:

1 Cumulative curve crashes. It is unlikely a reduction in speed limit can produce precisely the same level of cumulative curve crash risk over the route of interest than if the skid resistance value at each curve with a horizontal radius of curvature 400 m or less was equal to the assigned IL. The resulting cumulative curve crash numbers will be either higher (disbenefit) or lower (benefit).

2 Severity of straight road crashes. Both injuries per crash and injury severity are very much impacted by operating speed. Lowering of the speed limit will result in the largest reductions in operating speed on straight road sections. Therefore, the severity of road crashes on straight sections of rural state highway and their associated cost can be expected to decrease.

3.7.1.1 Curve crashes

The annual number of crashes at each curve along the rural state highway route of interest is calculated using equation 3.1 for two scenarios:

1. Skid resistance at the recommended skid resistance IL
2. The 85th percentile speed capped at the desired operating speed and skid resistance level of 0.4 ESC (refer section 3.2.4).

Summing the individual curve crash numbers gives an estimate of the annual number of curve crashes for the route. If the total annual curve crash number for scenario 1 is greater than scenario 2, then a benefit results. This benefit can be costed simply by subtracting the scenario 2 total curve crash number from scenario 1 and multiplying by the appropriate EEM crash cost value.

3.7.1.2 Straight road crashes

Procedures provided in section 3.1 of the NZ Transport Agency's (2016b) *Crash estimation compendium (New Zealand crash risk factors guideline*) and section A6.6 of the EEM are combined to quantify the impact of a lower speed limit on straight road crash numbers.

Specifically, for two-lane rural roads, the typical crash rate (reported injury crashes per year) is calculated using the exposure-based equation:

$$Injury\ crashes\ per\ year = (AF \times b_0) \times X \times CMF \qquad \text{(Equation 3.16)}$$

where:

AF = adjustment factor applied to b_0 to account for crash trends (see the EEM, pp5–292)
= 0.8 for the crash rate to pertain to 2016

b_0 = crash rate for straight road sections = 13 (for road type/One Network Road Classification of national strategic and regional strategic)

X exposure (midblocks)
= $L \times AADT \times 365/10^6$, where L = length (km) and AADT = annual average daily traffic

CMF = crash modifying factor from table 5 from NZ Transport Agency (2016b)
= 0.89 for seal shoulder width = 1.5 m and lane width = 3.5 m
= 1.22 for seal shoulder width = 0.75 m and lane width = 3.5 m
= 1.79 for seal shoulder width = 0.25 m and lane width = 3.25 m

Note: The seal shoulder widths and lane widths selected for assigning values to CMF coincide with group1 (over 2,000 vpd), group 2 (500 to 2,000 vpd) and group 3 (up to 500 vpd) minimum seal widths for state highways ie 10 m, 8.5 m and 7 m respectively of the NZ Transport Agency's (2002) *State highway geometric design manual* (draft).

Evidence indicates that both injuries per crash and injury severity increase with speed. Accordingly, the cost per injury crash increases with increasing speed as can be seen in table 3.2. With reference to section A6.6 of the EEM, crash costs for a specific mean speed can simply be calculated by linear interpolation of the values given in table 3.2, ie:

$$for\ 50 < V < 70km/h{:}\ C_V = C_{50} + (C_{70} - C_{50})(V - 50)/20 \qquad \text{(Equation 3.17a)}$$

$$for\ 70 < V < 100km/h{:}\ C_V = C_{70} + (C_{100} - C_{70})(V - 70)/30 \qquad \text{(Equation 3.17b)}$$

where:

C_V = cost of crashes for the mean speed V

C_{50} = cost of crashes in 50 km/h speed limit areas

C_{70} = cost of crashes in 70 km/h speed limit areas

C_{100} = cost of crashes in 100 km/h speed limit areas

V = mean speed of traffic (km/h)

Therefore, to value the effect of the lowering the speed limit on the straight sections of the route of interest, the following steps are taken:

1 All 10 m sections with a horizontal radius of curvature equal to or greater than 800 m are identified as having a straight alignment.

2 The summation of these 10 m lengths provides the value of L for input into equation 3.12.

3 The mean speed over these 10 m lengths is calculated for both the original speed limit and the desired speed limit using equation 3.3, capped at the respective speed limits.

4 The crash costs associated with both these mean speeds are calculated through linear interpolation of the crash cost values in table 3.2.

5 The resulting crash cost saving is calculated by multiplying the number of injury crashes per year derived from equation 3.12 by the difference of the two crash costs.

3.7.2 Vehicle operating costs

With reference to section A5 of the EEM, the VOC components affected by a lowering of speed limit comprise:

1 Base running costs, which include fuel, tyres, repairs and maintenance, oil and the proportion of depreciation related to vehicle use

2 Additional VOC that results when a vehicle travelling at its cruise speed has to decelerate to a minimum speed before returning to the original cruise speed when encountering situations such as curves. This variation in operating speed is termed speed change cycle.

As additional VOC due to speed change cycle will only pertain to curves, it is important not to double count the base running cost benefits resulting from a reduction in operating speed. This has been achieved by separately considering straight sections (ie 10 m road sections with horizontal curvature greater than 800 m) and curved sections (as determined by RAMM's curve context table) of the rural state highway route of interest.

The VOC values provided in the EEM are given in terms of vehicle classes and standard traffic compositions. However, to simplify the calculation of changes in VOC that result from a lowering of the open road speed limit on rural state highways, only two standard traffic compositions are considered, these being 'rural strategic' and 'rural other'.

The procedures adopted for calculating the VOC benefits are outlined below.

3.7.2.1 Change in base running costs

This only applies to straight sections of the route as this is where the largest changes in operating speed will occur as a result of the speed limit being reduced. Therefore, a filter is applied to the RAMM geometry data generated for the route to extract all 10 m sections with a horizontal radius of curvature greater than 800 m. This 800 m threshold has been selected because the start and end locations of the high-risk curves tabulated in RAMM's curve context table are defined by when the horizontal radius of curvature equals 800 m. The average speed and average absolute gradient of these straight 10 m sections are calculated separately for the increasing and decreasing lanes for input in the VOC cost equation for speed and gradient given in table A5.11 of the EEM. This VOC cost equation is reproduced below for ready reference.

$$VOCB_B = a + b \times GR + c \times ln(S) + d \times GR^2 + e \times [ln(S)]^2 + f \times [GR \times ln(S)] + g \times GR^3 + h \times [ln(S)]^3 + i \times [GR \times [ln(S)]^2] + j \times [GR^2 \times ln(S)]$$ Equation 3.18

where: *VOCB* = base vehicle operating costs in cents/km

GR = absolute value of average gradient (ie >0) over range of 0–12%

S = mean speed in km/h over range of 10–120 km/h

Ln = natural logarithm.

a,b,…j = constants given in table 3.3

The benefit in base running cost that results from lower operating speeds is the sum of the values calculated separately for the increasing and decreasing lanes using equation 3.19.

$$Running\ Cost\ Benefit\ (\$) = \frac{(VOCB_{OSL} - VOCB_{LSL})}{100} \times L_s \times \frac{AADT}{2} \times 365$$ (Equation 3.19)

where: $VOCB_{OSL}$ = base vehicle operating costs in cents/km calculated for original speed limit

$VOCB_{LSL}$ = base vehicle operating costs in cents/km calculated for lowered speed limit

L_s = length of route with horizontal radius of curvature > 800 m (km)

$AADT$ = annual average daily traffic

Table 3.3 VOC running cost by speed and gradient regression coefficients

Coefficient	Rural strategic	Rural other
a	13.79870979	15.72643344
b	0.73448090	0.48929243
c	48.56043376	44.72442039
d	0.05235459	0.03965776
e	-20.74309230	-19.47519438
f	-0.31061873	-0.18238796
g	-0.00464374	-0.00358501
h	2.43466309	2.29963206
i	0.03033303	0.01427468
j	0.02795727	0.02396070

3.7.2.2 Change in speed change cycle costs

The speed change cycle cost is calculated for each curve along the rural state highway route of interest, using VOC cost data provided in table A5.41 (for rural strategic traffic composition) and table A5.43 (for rural other traffic composition) of the EEM.

Two speed change cycles are associated with each curve, when entering the curve and when exiting the curve. Therefore, with reference to tables A5.41 and A5.43, when entering the curve in the increasing direction the initial speed is the curve approach speed and the final speed is the curve speed taken from RAMM's curve context table. When exiting the curve, the approach speed is the curve speed and the final speed is assumed to be the same as the approach speed from the decreasing direction.

The approach and curve speeds need to be rounded down to the nearest 5 km/h to facilitate ready application of tables A5.41 and A5.43 of the EEM without the complication of bi-linear interpolation.

As speed change cycle costs need to be considered for both increasing and decreasing directions, the analysis simplifies to calculating the speed change cycle cost associated with entry in both the increasing direction and decreasing direction. The total speed change cycle cost ($) per year is then simply the sum of both these costs multiplied by the following factor, (AADT × 365)/100, where AADT is the average annual daily traffic from the RAMM curve context table.

To determine the change in speed change cycle cost that results from lowering the speed limit, the calculation is performed a second time with the approach speed in either direction capped at the lowered speed limit.

The difference in speed change cycle cost for each curve is summed to provide the total benefit.

3.7.3 Carbon dioxide emissions

There is a direct relationship between CO_2 emissions and fuel consumption.

In accordance with section A9.7 of the EEM, the predicted value change in CO_2 emissions resulting from a lowering of the speed limit is simply 4% of the change in VOC running cost as calculated by equation 3.16 (refer section 3.6.2.1).

The reduction in the amount of CO_2 emissions produced, in tonnes, can be simply calculated as the $ value of the reduction in CO_2 emissions divided by 40.

3.7.4 Extension of seal life

The Transport Agency's T10 Specification for state highway skid resistance management (NZ Transport Agency 2013) highlights that cornering forces can significantly influence the rate of polishing of a surfacing aggregate, thereby directly impacting on the useful life of the road surface. As cornering forces are directly related to velocity squared (Dukkipati et al 2008), even a small reduction in the speed the curve is negotiated can produce an appreciable reduction in the rate of polishing. For example, a 10% reduction in speed will result in a 20% reduction in cornering force.

To account for this effect, it is assumed the expected seal life of all curves along the route of interest where the curve speed given in the RAMM curve context table is higher than the proposed lowered speed limit can be increased according to the following formula:

$$Extended\ Seal\ Life\ (years) = Expected\ Seal\ Life\ (years) \times \left[\frac{V_C}{V_{LSL}}\right]^2 \qquad \text{(Equation 3.20)}$$

where: $V_C \geq V_{LSL}$

V_C = 85 percentile curve speed from the RAMM curve context table (km/h) converted to mean curve speed using equation 3.9

V_{LSL} = lowered speed limit (km/h)

The RAMM seal lives tabulated in table 2.1 are too generic for use with equation 3.20. Therefore, the Transport Agency's surface longevity model, commissioned by SLAG, was used to obtain estimates of the expected seal life of rural high-risk curves (ie curves with a horizontal radius curvature equal or less than 400m) for input into equation 3.15. Table 3.4 provides the seal life estimates provided by the model for all types of chipseal surfaces combined (ie single coat and two coat seals) in terms of the four skid resistance ILs assigned to rural high-risk curves.

Table 3.4 Estimated life of chipseal surfaces on rural state highway high- risk curves

Rural curve IL	Estimated life for chipseal surface (years)		
	Average	50 percentile	75 percentile
0.55	6.71	6.68	7.33
0.5	7.10	7.07	7.76
0.45	9.03	8.99	9.87
0.4	6.95	6.92	7.60

With reference to table 3.4, the expected average seal life in all cases is seven years when the life is rounded to a whole number apart from curves assigned an IL of 0.45 ESC, which have an expected average seal life of nine years. The reason for this longer seal life is not readily apparent but in the absence of any other information has been adopted.

The benefit is derived from the cost of the sealing being able to be amortised over a longer time period. This is calculated as the difference in annual cost as follows:

$$Benefit\ of\ Longer\ Seal\ Life\ (\$) = Sealing\ Cost\ (\$) \times r \times \left[\left(\frac{1}{1-(1+r)^{-L_o}}\right) - \left(\frac{1}{1-(1+r)^{-L_n}}\right)\right]$$ (Equation 3.21)

where: $Sealing\ Cost\ (\$) = (Shoulder\ Width(m) + Lane\ Width(m)) \times curve\ lenth\ (m) \times Chipsealing\ Rate\ (\$/m^2)$

r = discount rate = 0.06 ie 6%

L_o = expected seal life ie 7 years (IL=0.4, or 0.5 or 0.55) or 9 years (IL=0.45)

L_n = extended seal life from equation 3.5 rounded to whole number

The discount rate has been set at 6% per annum as this has been effective since 1 July 2013 and is considered in the EEM to be appropriate for transport investment.

With reference to the sealing cost, the recommended chipsealing rate for use with equation 3.21 is $5/m^2 as this is the 50th percentile value of contracted rates throughout the country. For consistency, the sealing cost is based on the shoulder width and lane width being limited to the same three options as for the crash modifying factor (CMF) in equation 3.12, ie:

- 1.5 m shoulder width and 3.5 m lane width giving a seal width of 10 m, ie group 1 SH
- 0.75 m shoulder width and 3.5 m lane width giving a seal width of 8.5 m, ie group 2 SH
- 0.25 m shoulder width and 3.25 m lane width giving a seal width of 7 m, ie group 3 SH

3.8 Calculation of overall benefit

The annual cost of benefits associated with crashes, VOC, CO_2 and extended seal life is summed to determine if the result is greater or less than the annual cost disbenefit resulting from the increase in travel time associated with the lowering of the speed limit. If the result is greater, lowering the speed limit and using locally sourced sealing chip becomes an alternative option for achieving the desired levels of safety to sealing with aggregates with a high polishing resistance transported from outside the region.

4 Trial application

Separate MS Excel spreadsheets were written to perform each of the main calculation steps detailed in chapter 3 and applied to SH58, north of Wellington. The 100 km/h speed limit on this state highway extends from its intersection with SH2 to just before the Pauatahanui roundabout, a distance of 9.946 km. There are 28 high-risk curves (ie curves with a horizontal curvature of 400 m or less) along this length. Figure 4.1 shows the alignment of the section of SH58 of interest. This state highway is classified as 'regional strategic'.

Figure 4.1 100 km/h speed limit section of SH58 (increasing direction towards the left)

Because a reduction in operating speed will impact on the severity of crashes occurring on straight road sections as well as loss of control crashes on curves, crash numbers over the entire section of SH58 of interest should be considered and not limited just to crashes near curves.

The wet road crash history over the five-year period 2011 to 2015 for this section of SH58 is 17 injury crashes and 64 all (ie injury and non-injury) crashes corresponding to annual crash densities of 0.34 wet road injury crashes/km/year and 1.29 wet road all crashes/km/year. These crash figures along with the large number of high-risk curves make SH58 a good state highway to trial the analysis framework detailed in chapter 3.

The findings of this trial application are presented below.

4.1 Desired operating speed

Predicted collective and personal risk for each of the 28 high-risk curves located on the open road speed section of SH58 was calculated for the following situations:

- Open road speed limit and IL set to recommended values in RAMM curve context table, ie benchmark situation.
- Skid resistance set at 0.4 ESC and maximum 85th percentile speed reduced in increments of 5 km/h from 100 km/h to 50 km/h. The 100 km/h situation is equivalent to that used to generate the predicted collective and personal risk values tabulated in the RAMM curve context table. Therefore, the 100 km/h risk values calculated by the spreadsheet were compared with those in RAMM curve context table to ensure crash risk calculations performed by the spreadsheet were consistent with RAMM.

The calculated personal and collective curve crash risks are summarised in tables 4.1 and 4.2.

Table 4.1 Predicted personal curve crash risk for SH58, influence of speed limit and skid resistance

Statistic	Predicted personal curve crash risk (all injury crashes per 10^8 vehicles entering curve)											
	Target	Maximum 85th percentile speed (skid resistance = 0.4 ESC)										
		100 km/h	95 km/h	90 km/h	85 km/h	80 km/h	75 km/h	70 km/h	65 km/h	60 km/h	55 km/h	50 km/h
Max	9.23	9.90	9.29	8.89	8.05	8.05	8.04	8.04	8.04	8.04	8.04	8.04
Mean	4.84	5.64	4.97	4.56	4.30	4.13	4.04	3.98	3.94	3.90	3.88	3.87
Median	4.45	5.23	4.42	4.04	4.06	4.07	4.00	3.74	3.79	3.79	3.79	3.79

Table 4.2 Predicted collective curve crash risk for SH58, influence of speed limit and skid resistance

Statistic	Predicted collective curve crash risk (annual all injury crashes per curve)											
	Target	Maximum 85th percentile speed (skid resistance = 0.4 ESC)										
		100 km/h	95 km/h	90 km/h	85 km/h	80 km/h	75 km/h	70 km/h	65 km/h	60 km/h	55 km/h	50 km/h
Max	0.46	0.50	0.47	0.45	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
Mean	0.24	0.28	0.25	0.23	0.22	0.21	0.20	0.20	0.20	0.20	0.20	0.19
Median	0.22	0.26	0.22	0.20	0.20	0.20	0.20	0.19	0.19	0.19	0.19	0.19
Total no.	6.79	7.91	6.98	6.40	6.03	5.79	5.68	5.59	5.53	5.48	5.45	5.43

Comparing the column titled ‘target’ with that titled ‘100 km/h’ in table 4.1, it can be seen that for the speed limit of 100 km/h, managing the skid resistance of the high-risk curves to the default value of 0.4 ESC instead of the recommended IL, results in the mean personal crash risk increasing from 4.84 to 5.64 all injury crashes per 100 million vehicles entering the curve. This corresponds to an increase of about 16.5%, highlighting the need for alternative safety interventions if the desired skid resistance levels for high-risk curves cannot be economically maintained.

It can also be seen, as the maximum 85th percentile speed reduces, the maximum personal crash risk continues to reduce until it plateaus at a value of 8.04 all injury crashes per 100 million vehicles entering the curve once a speed of 75 km/h is reached. Similarly, the median personal crash risk plateaus at a value of 3.79 all injury crashes per 100 million vehicles entering the curve once a speed of 65 km/h is reached.

The best equivalency with the target values of personal crash risk is achieved when the maximum 85th percentile speed is 90 km/h so this is the desired operational speed for SH58 if sealing chip is sourced from local aggregate.

The trends observed in table 4.1 are also observed in table 4.2, which is concerned with curve crash numbers. This suggests either personal crash risk or collective crash risk can be used to identify the operational speed required to provide the same level of safety with poorer performing aggregates that are readily accessible as that for aggregates which maintain skid resistance at or above IL but whose availability is limited.

From CAS, the average annual number of all injury crashes over the 9.946 km of SH58 of interest is 7.20 over the five-year period 2011 to 2015. With reference to table 4.2, the predicted annual injury crash number for the 85th percentile speed limited to 100 km/h ranges from 6.79 for skid resistance at IL to 7.91 for skid resistance at 0.4 ESC. These predicted annual all-injury crash numbers span the actual crash number of 7.20, so there can be a degree of confidence in the curve crash risk model given by equation 3.1.

4.2 Travel time disbenefit

The base 85th percentile speed was calculated using equation 3.3 for each 10 m section comprising the 9.946 km of SH58 of interest, in both the increasing (towards SH1 with reference to figure 4.1) and decreasing (towards SH2 with reference to figure 4.1) directions. To obtain realistic speed profiles over the 9.946 km route, the acceleration, deceleration and gradient effects given by equations 3.5, 3.7 and 3.8 were applied to the base 85 percentile speeds.

When the SCRIM vehicle undertakes its annual high-speed pavement condition survey of the state highway network[2], the average survey speed over each 10 m section is recorded for data quality assurance purposes. This survey speed data is tabulated in RAMM's skid resistance table. Despite pertaining to a truck with a limited maximum speed of 80 km/h, the opportunity was taken to use this 'actual' speed profile data to assess how realistic the calculated speed changes are.

In addition, it is common practice to take the 'local speed' as the average speed over a 100 m to 200 m length surrounding the 10 m section of interest as this 'irons out' any irregularities in the data and approximates the actual speed changes a vehicle makes. Therefore, the opportunity was also taken to investigate if applying a moving average to the calculated 85th percentile speeds results in a speed profile that better matches the observed speed profile on SH58 than when acceleration/deceleration limited speed changes are employed.

The resulting speed profiles are shown in figure 4.2 for the increasing direction and figure 4.3 for the decreasing direction. An additional 3.069 km of SH58 has been plotted bringing the total length to 13.015 km as this 3.069 km contains a high concentration of tight curves (21 curves with a horizontal radius of curvature less than or equal to 400 m over the 3.069 km). This additional 3.069 km allows assessment of the proposed speed modelling's ability to deal with a quick succession of speed changes.

[2] www.nzta.govt.nz/roads-and-rail/road-composition/pavement-condition-surveys/

Figure 4.2 Comparison of theoretical and actual speed profiles for SH58 – increasing direction

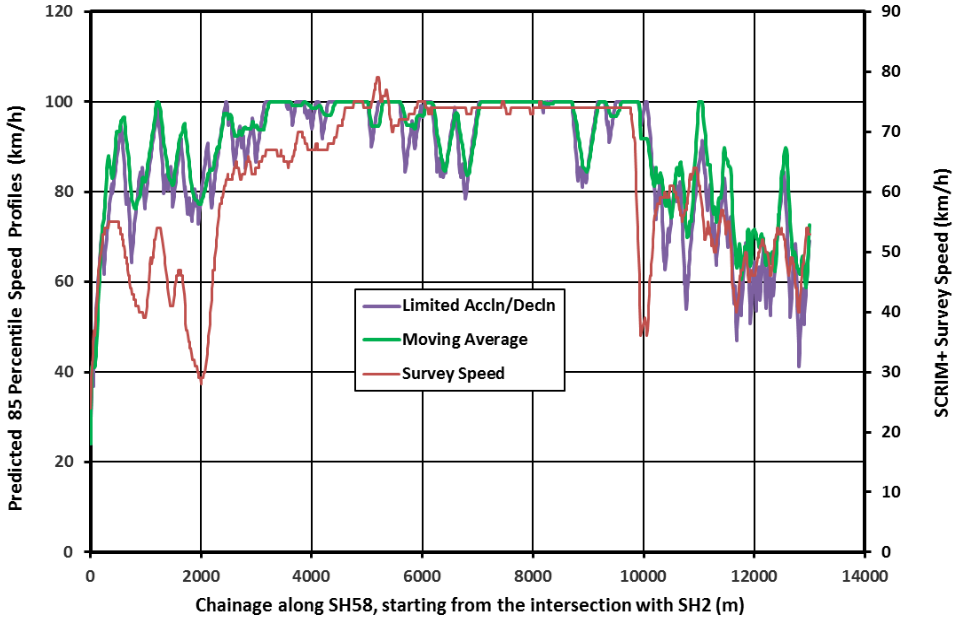
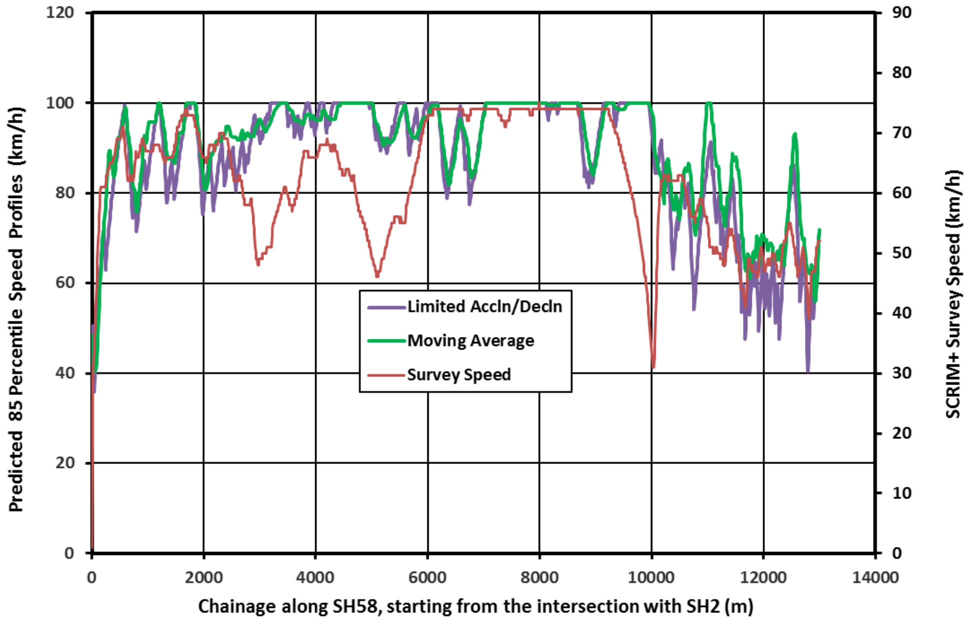


Figure 4.3 Comparison of theoretical and actual speed profiles for SH58 – decreasing direction

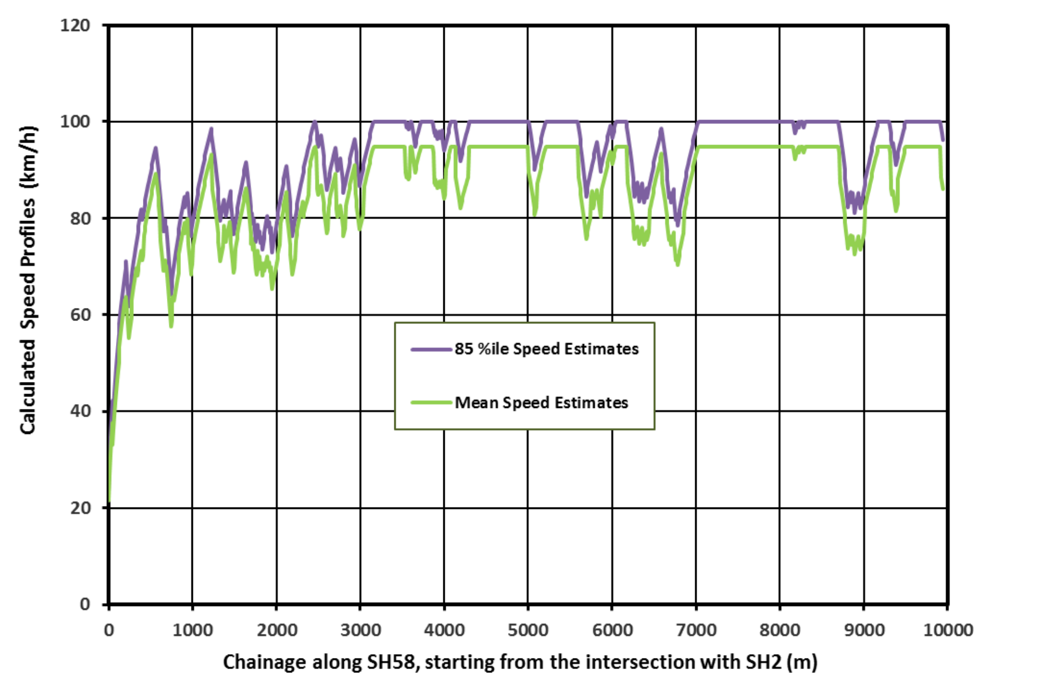


With reference to figures 4.2 and 4.3, it can be seen that applying a 200 m moving average has almost an identical effect in smoothing the speed profile to that of applying acceleration and deceleration limits as defined by equations 3.4 and 3.6. The resulting 'smoothed' speed profiles derived from the RAMM geometry table data provide a good match with the speed fluctuations observed in the SCRIM+ survey speed profile for SH58. Accordingly, estimates of travel time increases resulting from the lowering of the open road speed limit can be expected to be reasonably accurate.

Although applying the 200 m moving average provided a similar result to applying acceleration and deceleration based limits on the speed changes possible, it was decided to use the latter because the limit approach was easier to implement in a spreadsheet format and had a sounder basis.

Figure 4.4 shows the effect of applying the relationships given in section 3.4 for converting 85th percentile speed to mean speed over the 9.946 km section of SH58 with a 100 km/h speed limit. As expected, the speed profile remains relatively unaffected. However, the length averaged mean speed at 86 km/h is 6 km/h or about 7% less than the length averaged 85th percentile speed of 92km/h.

Figure 4.4 Comparison of theoretical and actual speed profiles for SH58 – decreasing direction



Reducing the open road speed limit from 100 km/h to 90 km/h over the 9.946 km route was shown to increase the travel time by 0.66 minutes in the increasing direction and 0.67 minutes in the decreasing direction. The cost associated with these increases in travel time was calculated to be $1,930,569.60c assuming an AADT of 14,254 vehicles per day and a composite value of travel time (all occupants and vehicle types combined) of $33.48c (July 2015).

4.3 Benefits

4.3.1 Reduction in curve crashes

With reference to table 4.2, reducing the speed limit to 90 km/h is predicted to result in 6.40 injury curve crashes per year. By comparison, managing the skid resistance of all the high-risk curves on the 9.946 km section of SH58 of interest to their recommended IL is predicted to result in 6.79 injury crashes per year. Therefore, the lowered speed limit option results in 0.39 less injury curve crashes per year.

This translates to a social cost saving of $228,150 per year assuming the EEM cost of a near rural mid-block injury crash of $585,000 (July 2015) (from table A6.5(a) of the EEM).

4.3.2 Reduction in severity of crashes occurring on straight road sections

The total length of straight road (ie horizontal radius of curvature greater than 800 m) over the 9.946 km route of interest is 5.11 km. This corresponds to about 51% of the route by length. The mean speed over this 5.11 km length is calculated to be 90.6 km/h for the 100 km/h speed limit situation (ie 90.42 km/h in the increasing direction and 90.80 km/h in the decreasing direction). For the 90 km/h speed limit situation the mean speed reduces to 83.4 km/h (ie 83.23 km/h in the increasing direction and 83.63 km/h in the decreasing direction).

Application of the procedure detailed in section 3.7.1.2 results in a crash rate estimate over the straight section of SH58 of interest of 3.4 injury crashes per year. This estimate is based on a cumulative length of 5.11 km, a seal width of 8.5 m (ie 0.75 m shoulder width and 3.5 m lane width), and traffic volume of 14,254 AADT.

As SH58 is considered to be a near rural road, the mid-block crash cost is calculated to be $538,033 (July 2015) for the estimated mean speed of 90.6 km/h, ie the existing 100 km/h speed limit situation. This compares to a mid-block crash cost of $503,148 (July 2015) for the lowered speed limit of 90 km/h.

The resulting annual saving in societal costs amounts to $118,609 (ie $3.4 \times (\$538{,}033 - \$503{,}148)$).

4.3.3 Reduction in base running costs

The inputs required to calculate the change in base vehicle running cost over the straight road sections of the route of interest using equations 3.18 and 3.19 are as follows:

Road classification:	rural strategic
Cumulative length:	5.11 km
AADT:	14,254 vehicles/day
Average speed (increasing direction):	90.4 km/h (100 km/h speed limit) 83.2 km/h (90 km/h speed limit)
Average speed (decreasing direction):	90.8 km/h (100 km/h speed limit) 83.6 km/h (90 km/h speed limit)
Average absolute gradient:	2.84% (increasing direction) 2.65% (decreasing direction)

The base vehicle running costs calculated from equation 3.18 are:

Increasing direction:	35.37c/km (100 km/h speed limit) cf 34.62c/km (90 km/h speed limit), resulting in a difference of 0.75c/km
Decreasing direction:	35.25c/km (100 km/h speed limit) cf 34.50 c/km (90 km/h speed limit), resulting in a difference of 0.75c/km.

The vehicle running cost benefit calculated from equation 3.19 is $98,832.78c per year in the increasing direction and $100,023.56c per year in the decreasing direction giving a combined total of $198,856.35c per year (July 2015).

4.3.4 Reduction in speed change cycle costs

For the 100 km/h speed limit, the sum of speed change cycle costs for the 28 curves along the route of interest amounts to $7,622,882.31c per year. This reduces to $4,358,200.22c when the speed limit is lowered to 90 km/h. Therefore, the resulting reduction in speed change cycle costs is $3,264,682.09c per year (July 2015).

By lowering the speed limit to 90 km/h, 3 out of the 28 high-risk curves on the portion of SH58 of interest, corresponding to about 11% of the curves by number, have no difference between the approach speed and the curve speed so there is no speed change cycle cost for these curves.

4.3.5 Reduction in carbon dioxide emission costs

The reduction in carbon dioxide emission costs is simply 4% of the reduction in base running costs taken from section 4.3.3. This equates to $7,954.25c per year. The corresponding reduction in the amount of carbon dioxide emissions produced is 199 tonnes per annum.

4.3.6 Reduction in maintenance costs due to extended seal life

Only 3 out of the 28 high risk curves located on the portion of SH58 of interest have a curve speed that is above the proposed lower speed limit of 90 km/h. The extended seal life for these curves ranges from a factor of 1.28 to 1.42, corresponding an additional two to three years to the expected seal life of seven years (refer table 4.3).

With reference to table 4.3, the equivalent annual sealing cost has been calculated using a discount rate of 6%, a sealing cost rate of $5 m^2, and a seal width of 8.5 m (ie 0.75 m shoulder width and 3.5 m lane width).

The overall calculated benefit is $506.12c per year. This low figure is attributed to the relatively low sealing cost rate coupled with short seal lengths of between 60 m and 150 m and the relatively long time period to amortise the cost.

Table 4.3 Effect of extended seal life on equivalent annual sealing cost

SH58 Curve start (m)	SH58 Curve end (m)	Curve radius (m)	Curve speed (km/h)	Rec. IL (ESC)	Ratio of curve speed to 90 km/h speed limit	Expected seal life (years)	Rounded extended life (years)	Equivalent annual sealing cost ($) (expected life)	Equivalent annual sealing cost ($) (extended life)	Difference ($)
1,730	1,790	398	96	0.4	1.42	7	10	$456.79	$346.46	$110.33
9,300	9,450	292	91	0.4	1.28	7	9	$1,141.99	$937.27	$204.72
3,490	3,630	297	91	0.4	1.28	7	9	$1,065.85	$874.78	$191.07
									Total:	$506.12

4.3.7 Overall benefit

The calculated component annual cost benefits associated with reducing the 100 km/h speed limit on SH58 to 90 km/h range from hundreds to millions of dollars. Their combined total amounts to $3,818,757.81 per year. This compares with the travel time disbenefit of $1,930,569.60 per year. Therefore, the combined annual cost benefit exceeds the travel time disbenefit by $1,888,188.21, indicating that reducing the 100 km/h speed limit on SH58 is a very effective safety measure if the skid resistance of the high-risk curves cannot be maintained at or above their recommended IL.

4.3.8 Effect of mismatched RAMM curve context and geometry tables

The RAMM curve context table was generated using geometry data acquired as part of the 2010 SCRIM$^+$ annual condition survey of the state highway network. However, in trialling the proposed analysis framework on SH58, the geometry data used for calculating travel time and straight road section crash and VOC costs was from the 2016 SCRIM$^+$ annual condition survey. This was done to replicate the expected application of the analysis framework, which involves combining data from the RAMM curve context table and RAMM geometry table. This contains data by default from the most recent SCRIM+ annual condition survey (in this case 2016).

To assess the effect of this mismatch in survey dates between the RAMM curve context and geometry tables, the benefit/disbenefit analysis for SH58 was repeated using curve context data generated from the 2015 survey geometry data so there was perfect matching between the curve context and geometry datasets. The routines used to generate the 2015 curve context data for SH58 were the same used to generate the 2010 curve context data. A comparison of the results is provided in table 4.4.

Table 4.4 Comparison of benefit/disbenefit costs

Component	Calculated annual cost ($)	
	2010 curve context & 2016 geometry	2015 curve context & 2015 geometry
Disbenefit		
Travel time	$1,930,569.60	$1,840,195.90
Benefit		
Curve crashes	$228,150.00	$260,325.00
Severity of crashes	$118,609.00	$117,171.46
Base VOC	$198,856.35	$193,959.42
Speed change cycle	$3,264,682.09	$2,772,591.78
CO_2 emissions	$7,954.25	$7,758.38
Extended seal life	$506.12	$ 91.94
Subtotal:	$3,818,757.81	$3,351,897.98
Benefit- disbenefit:	**$1,888,188.21**	**$1,511,702.08**

With reference to table 4.4, there is relatively good agreement between the component costs apart from speed change cycle costs. The calculation of this component cost requires the approach and curve speeds to be rounded down to the nearest 5 km/h. Differences between the 2010 and 2015 approach and curve speeds appear to have been magnified by this rounding process, resulting in a difference of $492,090.31 in the speed change cycle cost. This largely accounts for the difference of $466,859.83 observed between the net annual benefit calculated using 2010 curve context data and 2016 geometry data and that calculated using matched 2015 curve context and 2015 geometry data. This difference amounts to about 20% of the 2010/2016 net annual benefit.

The other important point to note with reference to table 4.4 is that travel time and speed change cycle annual costs dominate the analysis by a considerable margin, both being in the millions for SH58. The travel time cost is derived from data extracted from the geometry table, which is updated on an annual basis so is current. However, the speed change cycle cost is derived from the curve context table, which has an expected update cycle of five or more years.

Ideally, the time period between the geometry data that forms the basis of the curve context table and that contained in the current RAMMM geometry table should be as short as possible. However, from table 4.4, a difference of six years does not appear to pose much of an issue so long as no major road alignment has taken place in this intervening period. This can be checked easily by extracting geometry data for both the current and the 2010 survey contained in the RAMM geometry table and comparing curvature profiles for the length of state highway of interest.

5 Curve crash risk analysis tool

5.1 Tool design

It was decided to base the curve crash risk analysis tool around MS Excel, largely because of the flexibility this affords, and also because of the consistency with spreadsheet tools developed to implement EEM analysis procedures, which have found ready acceptance by the industry.

The format of this prototype for the Wellington region (and SH58 in particular) allows for a future Access-Excel approach where the user first selects the region with the sections of interest.

A macro would interrogate the MS Access™ database(s) for the records for the region, these having first been stored in Access form from a structured query language (SQL) routine run on the RAMM database to extract the appropriate RAMM tables and data fields in a prescribed manner.

For the prototype stage, the macro for interrogating the Access databases has not been developed but the SQL routine has. Appendix A shows what data fields are extracted by the SQL from RAMM's 'carriageway', 'curve context', 'high-speed data (HSD) geometry', 'sign' and 'skid resistance' tables and the overall structure of the input datafile for the prototype spreadsheet allowing the required input data to be manually extracted from RAMM if desired. However, a complete listing of the SQL has been provided in appendix B to facilitate automatic extraction of all the RAMM inputs required by the MS Excel-based spreadsheet tool developed to perform the analysis framework detailed in chapter 3. The spreadsheet tool is available as appendix D at www.nzta.govt.nz/resources/research/reports/636.

The RAMM HSD geometry data is split by direction with the increasing direction sorted in ascending sort order into one worksheet tab and the decreasing direction sorted in descending order in another worksheet tab.

The curve context table is based on historical (2010) data and ideally the timeframe for it should match that of the high-speed road condition data. However, in developing and testing the prototype, HSD road condition information from the 2016 HSD survey has been used.

Key features of the MS Excel based spreadsheet tool are detailed in the following sections. The item numbers bracketed in the section heading refer to row identifiers of the example prototype input-output worksheet screen dump provided in figure 5.1 at the end of this chapter.

5.2 Highway section selection [items 1 and 2]

As noted above, the user first specifies in the Input_output worksheet tab (refer figure 5.1) the region the network outcomes contract (NOC)/network management area (NMA) of interest is located in.

The user then specifies which state highway is of interest by selecting from a dropdown box (to avoid issues with texts and numbers). Because the original curve crash risk modelling work was based on the seven Transit NZ regions, there are many more highways in the drop down list to choose from than are current for the NOC/NMA region selected.

The user then specifies the start reference station (RS) and route position (RP) and the end RS (might be the same as the start RS) and end RP.

The user is also able to specify the start RS/RP and end RS/RP within the selected section to be excluded from the analysis. This was originally done to exclude the urban western half of SH58 without having to manually enter the data for the urban/rural field, but can be useful for excluding a short section of highway for a variety of reasons.

The user is also asked whether to include adjustment of the HSD standard 10 m length if there is missing HSD geometry data.

It should also be noted that the length of the beginning and end straights related to the selected curve data is derived from the start RS/RP and end RS/RP, while the lengths between the curves are derived directly from the HSD geometry data. Accordingly, it is important for the analyst to input the correct end RP and not arbitrarily enter 9999 for instance.

5.3 Speed model values [item 3]

For the section chosen, the analyst specifies the posted speed limit or speed environment as appropriate. Currently this is a dropdown box with 60,70,80,90,100 km/h being the only options but this could be modified to reflect speed environment values to the nearest 5 km/h.

The model outputs the crash risk for different target speed values and matches with the modelled crash risk for the existing (higher speed) situation. The default range tested is from 5 km/h above the existing situation to 30 km/h below the existing situation in 5 km/h intervals. There is no reason why these should be changed except when an error occurs in which case the user is alerted to increase or decrease the input value(s).

The user is asked to specify which speed model to use, the choices being that embedded (hard-coded) in the RAMM curve data, an adjusted speed model developed and used in testing to gauge the effect of reducing the high approach speeds on short straights, and the limiting acceleration/deceleration models based on work in India.

The preference is not to use the model in the RAMM curve data as it appears to be based wholly on equation 3.3 with the gradient correction equation 3.8 included. It has near instantaneous acceleration to the input (straight) speed which is considered unrealistic.

The adjusted speed took the speed of the approach straight as x records before the start of the curve, where the default was x=5 or 50 m. At the same time the radius for the curve was taken to be the third smallest radius (or second smallest for short curves).

The RAMM curve context table splits a curve into two where there is a change in the RAMM roadname along the curve and the start/end records of some curves were not multiplies of 10 m so some allowances were made to incorporate these.

5.4 Target investigatory level [item 4]

The default target IL is set as 0.40 and there is no particular reason why this should be changed. It is a value entered at present but in due course the user should choose from a dropdown list.

5.5 Curve model parameter values [items 5, 6 and 7]

The model limits are given in item 5 while three of the key parameter values are given in item 6, including a year and regional value.

The lower and upper percentiles for the curve risk equilibrium results can be altered if need be. The results are presently output in columns BW to CZ of the ‘CurveAnal’ worksheet tab for rows 203 to 208.

Note that at present the spreadsheet accommodates up to 199 curves (rows 3 to 201), while it accommodates up to 1,508 analysed HSD records (rows 3 to 1510 in the HSDanal_ worksheet tabs).

5.6 Curve model key results [item 8]

The key results for the curve risk equilibrium modelling are given. The equilibrium speed is derived from linear interpolation of the model values (at 5 km/h interval) according to the statistic chosen by the user (default is sum) and the nearest speed limit (rounded or rounded down to nearest 5/10 km/h as selected by the user) stated along with its computed curve risk. This lowered speed limit (currently input_output worksheet tab cell C62) is a critical value used in the economic evaluation models. It can vary according to the statistic chosen and, depending on its value, the rounding choice selected. It also depends if it is applied to other than the predicted number of injury crashes, which is the default (item 8f). Injury crash rate can also be selected (but do not apply to the 'Sum' statistic) as it is a user definable variation on the crash number (not implemented as yet but allowed for).

Furthermore, the resulting critical speed value can vary along the route selection so that if the user, for example, splits the route into say 2 km lengths, different equilibrium speed values can result. This can particularly be true if the speed environment varies significantly along the route. Of course, the equilibrium speed along with the posted speed limit, should generally be constant along the whole route selected by the user.

5.7 Journey time analysis [item 9]

The first three parameters relate to the speed model and may possibly be moved to be with the other speed model parameters.

Output results for the next eight parameters are given by direction, the first being the difference in travel time (given in minutes) for the existing situation compared to that for the lower speed limit.

Currently, for testing purposes to check the spreadsheet tool results with those for the test case, the user could input 'test' in cell E79.

5.8 Economic evaluation input [items 10a-e and 10f]

In order to undertake an indicative cost-benefit analysis of introducing a lowered speed limit, the user needs to input the following:

- One Network Road Classification (ONRC) – dropdown list (Access is not included)
- horizontal topography of the straight sections – dropdown list (default is straight)
- road category - dropdown list (default is rural strategic except if ONRC secondary collector is chosen in which case it is rural other)

If the average ADT does not tally the range expected for the input ONRC, then a warning message appears. Likewise, if the road category selected does not match that expected for the input ONRC.

For baseline VOC calculations, the two options available are 'rural strategic', which pertains to arterial and collector roads connecting main centres of population and carrying traffic of over 2,500 motorised vehicles/day) and 'rural other', which pertains to rural roads other than rural strategic.

For the speed change cycle calculations, the worksheet tab 'EEM16_TabA543' contains the parameter values and the lookup table for approach speeds 65 to 120 km/h in 2.5 km/h intervals (interpolation averaging applied) and curve speeds 30 to 115 km/h. The range could be extended to match that in the EEM (ie 5 and 0 km/h) but the model is for rural curves only. Certainly, a check will be needed in due course but none of the SH 58 curves had a curve speed slower than 30 (it is conjectured the HSD table removes such instances).

The user also inputs the average traffic lane and shoulder widths for the section, the defaults being based on those for the chosen ONRC category. These are used to derive the CMF for estimating the mid-block injury crash rate using the long established EEM model (refer item 10 f).

The CMFs from the EEM have been interpolated with values for lane widths at 0.1 m intervals from 2.8 to 3.6 m plus the EEM 2.75 and 3.25 m values. The values for shoulder widths have also been interpolated to give them for 0 to 2.0 m at 0.25 m intervals. The user input shoulder width value is rounded to the nearest 0.25 m and set at a maximum of 2 m.

5.9 Economic evaluation crash cost output [item 10g]

The derived crash rate in item 10 f is applied to the EEM crash costs adjusted for the average speed on the straight (item 10 g) to give the crash cost saving on the straight (item 10 h). In a similar vein, the crash cost savings on the curves (item 10 j) are derived from the EEM crash costs on the curve adjusted for the average curve speed. The default crash values are based on the 100 km/h 'near rural' values although for 'rural other' it is for the 100 km/h remote higher value.

Item 10 k is the sum of the crash cost savings with the lower speed limit.

5.10 Economic evaluation travel time cost output [item 10l]

The evaluation of the travel time disbenefits arising from the lower speed limit is given in item 10 l, noting however, if the reduction in speed limit is considered to be part of the do minimum situation (for example in accordance with a strategic plan or district plan policy), then it could be ignored.

5.11 Economic evaluation VOC output [items 10m-o]

The evaluation of the VOC benefits arising from the lower speed limit is given for the speed change cycles for the curves (assessed on an individual basis) as well as for the reduced average speed along the straights.

The VOC speed change benefits can be considerable and so care was taken in computing them.

5.12 Economic evaluation CO_2 output [item 10p]

The evaluation of the carbon dioxide (CO_2) benefits arising from the lower speed limit is given based on the VOC benefits for the reduced average speed along the straights.

The current EEM is inconsistent in how CO_2 is evaluated, referring to earlier EEM values. The approach used is effectively the same as for the 2013 EEM whereby the CO_2 benefit, as a %VOC based on traffic links, is generally 3.0% +/- 0.1%. 2.9% was chosen for the 10-12 %HV range for 'rural strategic' and 'rural other' with adjustment to compensate for the reduced %VOC attributable to fuel. The result was that the ratio of the CO_2 benefit to the calculated tonnes CO_2 was very close to the EEM default value of $40 per tonne. No CO_2 update factor was applied (ie inferred 1.0 update factor).

5.13 Economic evaluation resurfacing cost output [items 10q to 10v]

The evaluation of the benefits arising from reduced skid resistance demand on the tighter curves is given, based on the IL for the curves with speeds above the reduced speed limit. The approach involves inputting the expected life for the curves based on the IL and the expected resurfacing cost, which suits analysis of RAMM in a similar manner to that previously undertaken by Wanty (2014) on behalf of the Transport Agency. The expectation is that curves with low IL are resurfaced less frequently than those with higher IL and the resurfacing cost per square metre is also expected to be less. It should be noted that the overall benefits (item 10 u) are relatively low and not particularly sensitive to life-cost assumptions.

5.14 Economic evaluation summary output [item 11]

The evaluation benefits are summed and travel time disbenefits repeated with the overall (indicative) net present value (NPV) annual savings given (item 11c).

5.15 Urban/rural assumptions [item 12]

The default assumption is that the 10 m sections after the last curve and before the first curve are rural [items 12a and 12b]. The 10 m sections between curves have the same urban/rural assumptions as the preceding curve [item 12c]. If for some unknown reason an error occurs, an ‘X’ is output by default [item 12d].

5.16 85th percentile speed to mean speed [item 13]

The model parameters for the two equations (straights and curves) used for converting the 85th percentile speed to the mean speed for the purposes of economic evaluation are specified here (refer section 3.4 for the conversion equations employed). Item 13a is the curve/straight radius threshold (default is 400 m) while item 13e also relates to when the curve equation is applied (low speeds, the default speed threshold is 50 km/h). The model coefficients are given in items 13b, 13c and 13d.

Based on the conversion, the mean speed for the curve risk adjusted (design) speed [item 8d] is output in item 13e. Its potentially different rounded value [item 13f] is not used.

5.17 Notes worksheet

A notes worksheet has been included to provide helpful referencing related to key variables and factors. It splits the notes into three main parts, namely relating to the SQL input data, the EEM input data and the analysis tabs (worksheets). The notes are reproduced as part of figure 5.2 below.

5.18 Other issues and comments

The spreadsheet based tool is at the prototype stage and has not been tidied up to remove redundant cells and EEM extracts.

Undoubtedly some issues will arise, which potentially could be from the urban/rural field and the rural other/rural remote scenarios but these can be readily rectified because of the spreadsheet format.

The tool is only meant to apply to rural sections of highway and has not been tested on expressways as it is assumed expressways would not usually be the target of this type of assessment. Any 10 m section coded as urban in the HSD geometry data will be ignored.

NOC/NMA regions need to be mapped to the seven 2002 Transit NZ management areas employed in the curve crash risk modelling (ie equation 3.1), so appropriate region constants required by the model can be assigned.

The currently allocated number of analysis rows might need increasing in due course for assessing a long section of windy highway.

With respect to the SQL, the order of downloading the RAMM curve data fields is important. The SQL might also not sort the HSD decreasing data by the start_m field as a matter of course, so manual intervention might be required after copying the SQL decreasing data into the *HSDdataD* worksheet. In terms of maximum allowed input data, there is no limit aside from the Excel default of 1,048,576 rows but there is an analysis limit of 2,499 analysed HSD records (that is 24.99 km) within the *HSDdataI* & *HSDdataD* worksheets and 199 curves within the *CurveAnal* worksheet (noting this includes contiguous curves with different road_name). Note also that as RAMM fields can change their names from year to year, the current SQL might require future updating.

Figure 5.1 Input- output worksheet tab print area

original	INPUT PARAMETERS:	Change required	=	Change required		
29/07/16	RURAL CURVE CONTEXT	May need to change	=	May need to change		
latest	RISK EQUALISATION TOOL	Change unlikely	=	Change unlikely		
6/07/17	Created by David Wanty, WTC	Should not change	=	Should not change		
ver7l	models by Peter Cenek, Opus Research	Cannot change or n/a	=	Cannot change or n/a		
A	**Key input parameters**	dropdown box				
				CurveContext		
	Influence Database choice			RAMMdata		
1a	NOC or NMA region	NOC	NOC	CurveData	-1	0
1b	NZTA RAMM Region	Wellington		20	20	21
				CurveContext	HSDdataI	HSDdataD
	Filter RAMM curve data			0	0	0
2	Section of highway to analyse			236	21249	19597
a	State Highway	2	1N	2	2	9979
b	start RS	921	0	21	6046	12509
c	start RP (m)	0	0	21	6046	13554
d	end RS	931	0	94	7091	11150
e	end RP (m)	99999	99999	140	8449	11150
	Excluding the sub-section		X			
f	start RS	0	0	#N/A	#N/A	#N/A
g	start RP (m)	0	0	#N/A	#N/A	#N/A
h	end RS	0	0	#N/A	#N/A	#N/A
i	end RP (m)	0	0	#N/A	#N/A	#N/A
j	Apply HSD (missing) data length modification ?	Yes	Yes	Yes	No	
3	*Influence Model target values*					
a	Section speed envt/posted speed limit	70	100	30		
b	Upper speed limit test	75	75	4		
c	Analysis output interval (km/h)	5	5	5		
d	Use CurveData or adjusted speeds	Indian2 adj	Indian2 adj	Adj Speed	50	0
4	Target default IL (for curves)	0.40	0.40	4		
B	**Key speed model parameters**					
5	Model limits					
a	Curve speed : min	20	20			
	Curve speed : max	110	110			
b	Curve length: min	0	0			
	Curve length: max	800	800			
c	ADT : min	100	100			
	ADT : max	50,000	50000			
d	Grade : min	-15	-15			
	Grade : max	15	15			
e	OOCC speed differential: min	0	0			
	OOCC speed differential: max	50	50			
f	Curve/Straight radius threshold (m)	800	800			
6	Curve Model parameter values					
a	Constant	0.0000177	0.0000177	crash rate		
b	Year (2002)	0.25384	0.25384	crash number		
c	Region	0.28962	0.28962	crash no. RMS		
7	Curve Crash results percentiles					
a	Lower, Upper Percentiles	0.25	0.75	0.25	0.75	
8	Curve Analysis Results (inj ax/yr)	round down 5	Sum	5	6	Max
a	Input posted speed IL	9.38	inj ax/yr	round to 5		Average
b	Input posted speed IL adjusted	10.06	inj ax/yr	round down 5		Median
c	Predicted speed for IL adjusted	67.03	km/h	round to 10		Lower %
d	Risk equalised speed limit	**65**	km/h	round down 10		Upper %
e	Inj crashes at reduced speed limit	8.91	inj ax/yr	65		Sum
f		crash number		2		

Figure 5.1 continued (1)

9	Journey Time analysis						
a	Rawlinson equation constant	107.95	107.95				
b	Koorey grade modification max spd	125	125	km/h			
c	Urban limit applied to speed	70	70	U			
d	Route difference jny time (mins) - inc,dec	0.870	0.310				
e	Curve data: km , number (RAMM, unadj)	13.95	120	11.77,8.95	120		
f	Calc distance of straights (km) - incr,decr	8.65	6.64	-0.36			
g	Avg absolute gradient straight - incr,decr	3.55	3.77	0.09	0.03	0.10	0.16
h	Avg jny speed: straight current - incr,decr	62.60	61.18	33.86	34.04	35.17	34.67
i	Avg jny speed: straight risk equil - incr,dec	59.10	60.00	33.77	34.02	35.08	34.50
j	Avg jny speed: curve current - incr,decr	51.33	47.66				
k	Avg jny speed: curve risk equil - incr,decr	50.27	47.15				
l	VKT on curves 100 M veh-km/yr, AADT	0.282	5547				
m	VKT on straights 100 M veh-km/yr, AADT	-0.009	-285				
n	Total vehs entering curves (100 M / yr)	2.433	5512	365			
C	Key economic model values & output						
10	Economic evaluation for section(s)	expect Arterial	Arterial	4			
a	ONRC for selected section	Regional Strategic	23.25	2	Rural Strategic		
b	Horizontal Alignment: straights	Straight	straight	Crash Estimation Compendium, b0			
c	EEM Road Traffic Category	Rural Strategic	Rural Strategic	Rural Strategic	Rural Other		
			2500				
d	Traffic lane average width on straights	3.50	3.50				
e	Shoulder average width on straights	2.00	2.00	10			
f	inj crashes on straights: b0, exposure/yr	13	-0.008	26			
	crash mod factor (CMF), rate adj factor	0.66	0.800	2016	0.66		
	EEM injury crash rate/yr: straight 100MVKT	-0.05					
g	EEM default injury crash cost, 70 mid-blk	$435,000		11			
	EEM default injury crash cost, 100 mid-blk	$585,000		6			
	adj crash cost for straight (70-100): current	$394,455			61.9		
	adj crash cost for straight (70-100): risk eq	$382,753			59.6		
h	NPV Cost saving/yr: straight	-$ 638	not updated				
i	adj crash cost for curve (70-100): current	$332,485			49.5		
	adj crash cost for curve (70-100): risk equil	$328,559			48.7		
j	NPV curve ax/yr saving for risk equil	$ 191,217	not updated				
k	NPV approx. cost saving/yr: route	**$ 196,296**	1.03	Crash UF	12		
l	NPV extra travel time cost/yr: incr & decr	**$ 667,049**	1.45	TTC UF	8		
m	NPV VOC saving/yr: speed change curve	$ 2,229,783	0.98	VOC UF	9		
n	NPV VOC saving/yr: $/km route inc & dec	-$ 864	1	5	round	0.10	5
o	NPV VOC saving/yr	**$ 2,228,919**	0	3.1%	2.9%	4.0%	
p	NPV CO2 saving/yr, tonnes/yr (route)	**-$ 25**	**-1**	0.0009	0.0016	0.00098	
				39.8	40	0.740	
q	Cost of curve surfacing: current, risk equil	$ per sqm	$ per sqm	5.00		4.15	
	IL = 0.55	$ 16.00	5.00	16.00		0.53	
	IL = 0.50	$ 8.00	5.00	8.00		1.82	
	IL = 0.45	$ 5.00	5.00	5.00		0.29	
	IL = 0.40	$ 5.00	5.00	5.00		1.51	
r	Life of curve surfacing: current, risk equil	years	years	7.0		5.58	
	IL = 0.55	4.0	7.0	4.0		3.46511	
	IL = 0.50	5.5	7.0	5.5		4.56998	
	IL = 0.45	7.0	7.0	7.0		5.58238	
	IL = 0.40	7.0	7.0	7.0		5.58238	
s	Assumed curve carriageway width (m)	11.0	11.00	30	1.06	6%	0
t	NPV resurfacing costs at current IL	$ 79,700	not updated	$ 79,700			
u	NPV resurfacing costs at target IL	$ 24,984	not updated	$ 40,888			
v	NPV approx resurfacing cost savings/yr	**$ 54,716**	1.00	32			
11	Economic Summary			3.87		24.00	
a	Crash, VOC, CO2, Resurfacing savings/yr	$ 2,479,907		3.11		12.00	
b	NPV Travel time cost disbenefits/yr	$ 667,049		2.60		8.00	
c	NPV benefits/yr (-ve means disbenefit)	**$ 1,812,858**		2.40		5.50	

Figure 5.1 continued (2)

NB:	The above analyses include approximations and make a number of inherent assumptions						
	The HSD data can be missing a few records and due to oddities length is taken as end-start						
	The analysis includes checking on curves being artificially split in RAMM (diff roadnames)						
	Currently the speed change cycle table is reduced to min app spd=25 and min curve spd=0						
	This tool is still at the prototype development stage - user care and sense is required						
	(check also to see if the update factors inter alia need updating - refer Notes tab)						
D	**Added (mid-2017) model parameters**						
12	Urban/Rural assumptions (June 2017)						
a	After last curve in CurveContext data	R					
b	Before first curve in CurveContext data	R					
c	Between curves-uses the preceding curve	U or R					
d	If unexpected error encountered	X					
13	85% speed to mean speed formulae						
a	Curve radius threshold	400	m	10			
b	Coeff for (tight) curves [Ref Eqn 3.9]	0.8951		60.0			
c	Coeff for easy curves and straights	0.878		62.0			
d	Coeff 2 for easy curves and straights	0.0006942		0.000694			
e	Curve equation applies if 85% speed <	50	km/h	85			
f	Mean speed limit for risk adj PSL	60.0	67.0	65			
g	Rounded Speed limit to apply (not used)	60	round to 10				1

Figure 5.2 Prototype 'notes' worksheet tab print area

original	INPUT PARAMETERS:		
29/07/16	**RURAL CURVE CONTEXT**		
latest	**RISK EQUALISATION TOOL**		
6/07/17	Created by David Wanty, WTC		david@transportconsultant.co.nz
ver7l	models by Peter Cenek, Opus Research		peter.cenek@opus.co.nz
mid-June 2017	**NOTES: SQL Input Data**		Comments
Tab	CurveContext		
1	Field order differs from CurveData and RAMM data previously		Can now longer use CurveData option
2	No longer includes curve direction LH or RH but not used		
3	Appears based on the increasing direction		
4	Where radius goes from >800 to < 800 record is included as curve		
5	Curve may be arbitraily split in RAMM if different roadname		Rectified later is same geometry
Tab	HSDdataI		Copy/del formulae to suit no. records
1	Field order differs from previously so CurveAnal row 2 changed		
2	Assumes sorted by start_m in ascending order (Column T)		
3	Urban_Rural field based on CurveContext and for straights based		
	on the preceding curve and user input before first/after last curve		
Tab	HSDdataD		Copy/del formulae to suit no. records
1	Assumes sorted by start_m in decending order		
2	Prototype SQL had some extra -I incr dirn data, removed manually		Since rectified
	NOTES: EEM Input Data		
Tab	EEM16_TabA543		
1	Copy of additional VOC speed change cycle (SCC) costs: rural		
2	Rural Strategic is default, other choice is Rural Other		
3	Table A5.41 and 5.43 values entered in full (5 km/h interval)		
4	Currently tool uses subset of min app spd 65, min curve 30 km/h		Amended to min app spd 20, curve 0
5	Uses interpolation of app spd to give values at 2.5 km/h interval		
6	The values used are in cell range A63:S86		
	I think if get error then adopts value for 20 km/h [CHECK]		
Tab	VOCtabA511		
1	Newly added tab with the EEM formulae coeff for VOC cent/km		
2	Opus supplied the coeffs as EEM 2016 has them wrong! (OK 2017)		
3	Rural Strategic is default, other choice is Rural Other		refer input_output!C86 (item 10c)
4	The values used are in cell range D9:D18		
5	The derviation of the VOC cruise spd costs whole route included		
6	Based on average cents/km for straights and curves by direction		
Tab	input_output		
1	Various factors given in cell range I64:O82; and thence P64:P82		
2	2016 Update factors are updated to 2017 (range P72:P76)		Refer NZTA EEM website (Table A12.2)
3	CO2 $ proportion of VOC factors in range E107:G107 (user-input*)	*	~3% (not 4%) based on EEM formulae
4	CO2 tonnes factors from CO2 $ in range E108:G108 (uses %HV)		
5	crash mod factors (CMF) for lane/shdr width in range W211:AH220		
6	discount factor & analysis years in cells G120 & E120 respectively		
7	update factor for surfacing costs in cell d123 (default is 1.00)		
8	days per year in cell E80 (default is 365.0)		
9	Additional VOC speed change cycle costs rural in range P11:Q20		

Figure 5.2 continued (1)

	NOTES: Analysis Tabs		
Tab	CurveAnal		
1	For equating the risk equilised IL and thence crash costs		
2	And also the VOC SCC costs independent of curve length ($/veh)		
3	Column EZ adjusts for arbitrarily split contiguous curves		
4	Uses range of 8 speeds at 5 km/h interval and does error check		
5	Code for excluded curves specified in input_output!D21		
6	Cols FA to FF calc curve resurfacing cost difference		
7	Curve direction given in CurveData not given in CurveContext		
8	Adverse & absolute crossfall fields not given in CurveContext		
9	Est. personal & collective crash fields not given in CurveContext		
Tab	HSDanal_i		Currently formulae to row 2501
1	Row 2 formulae adjustment but tool based on CurveContext only		
2	The tool uses the Indian2 procedure for adjusting the speeds		
	(CurveData option) and there was another (Adj Speed) option		
3	The derivation of the Risk equalised speed applies to 85th %ile		
	whereas the economics uses the mean derived from 85th speeds		
	(two eqns, one for curves <= 400m)		
4	The length of the straight after the last curve is given in cell AM2		
5	The Okay field col AB shows 10 m records excluded from analysis		Can enter an "X" manually if so desire
Tab	HSDAnal_d		Currently formulae to row 2501
1	As for HSDanal_i with v minor diff in formulae as desc order		
Tab	input_output		
A	Items 1-2 relate which rural section (suggest 2-20 km) to analyse		
	One segment within it can be excluded (urban auto-excluded)		
	3a: Enter the posted speed limit or envt speed for the section		
B	Items 5-6 and 9a to 9c relate to the curve model		
	Items 7-8 relate to curve risk equilisation model		
	Items 9d to 9n are calc. jny dist/time/spd for curves & straights		
C	Items 10 to 10c, 10d+10e relate to the section category		
	Items 10f to 10k are the calc crash costs from CurveAnal		
	Items 10m to 10p are the calc TTC, VOC+CO2 costs from HSDdata_		
	Items 10q to 10s are the calc srufacing costs from CurveAnal		
D	Items 12 & 13 relate to the curve & risk equilisation models		
	calculating the mean speed from the 85th %ile for EEM costing		
Other	cells H106, H120, H150 relate to rounding [+ dropdown cell C62]		
Tab	**NOTES: Earlier/hidden worksheets**		These might be deleted in due course
RAMMdata	Original mid-2016 RAMM data		
CurveData	Mid-2016 generated data		
SH58test	Mid-2016 data only for SH58		Deleted
HSDtest	Mid-2016 HSD data for SH58 (data differs from mid-2017)		Deleted
Comp	Mid-2016 comparing results with those of Peter Cenek		Deleted
	His mid-2017 files used the previous HSD data since changed		
	His rounding was to nearest 5 km/h (I used 2.5) or initial 1 km/h		

6 Conclusions and recommendations

Because of financial and resource constraints, there is sometimes a need to manage the wet friction of state highways to values below those specified by the NZ Transport Agency's T10 specification. This results in an adverse impact on crash-risk that can be negated through reducing speed limits with attendant benefits (ie reduced VOC and longer seal lives) and dis-benefits (ie longer travel times). Research was therefore undertaken to develop an analysis framework that allows the impact of lowering both wet friction levels that state highways are managed to and speed limits to be assessed on the basis of crash risk and road user costs.

The main conclusions and recommendations arising from the development and trial application of this analysis framework are as follows.

6.1 Conclusions

1. The analysis framework could be formulated entirely around procedures in the EEM for calculating speed effects on travel time, crash severity, and baseline and speed cycle change VOC supplemented by Transport Agency research on seal lives and curve crash risk. As a consequence, the calculation of benefits resulting from the combined lowering of maintenance levels of skid resistance and speed limits are pertinent to New Zealand conditions and can be directly compared with other competing asset preservation or safety projects.
2. Application of the analysis framework to SH58, a state highway classified as 'regional strategic' with an average crash density of 0.72 casualty crashes per kilometre per year, showed the analysis is dominated by a significant margin by speed change cycle and travel time considerations. This highlights the need for accurate estimation of free-flow vehicle speeds along rural state highways, especially in the vicinity of curves, to allow robust economic assessment of adopting lower speed and skid resistance limits.
3. For SH58, a decrease in operational speed of only 10 km/h from 100 km/h to 90 km/h was found to be sufficient to completely negate the increased curve crash risk that results when the skid resistance of the high crash risk curves is managed to a level of 0.4 ESC rather than the recommended IL, which on 19 out of 28 curves was 0.5 ESC or greater. However, this finding should be treated with caution until such time as the curve crash risk model used is independently validated.
4. The associated user benefit was significant, corresponding to an annual cost saving of about $1.5 million. This indicates that, for SH58, reducing the 100 km/h speed limit is a very effective safety measure if the skid resistance of the high-risk curves cannot be maintained at or above their recommended IL.

6.2 Recommendations

1. For ease of application, the SQL macro written to automatically extract, format and order all the data fields from RAMM required by the spreadsheet tool to perform the economic assessment of lowering speed and skid resistance limits should be able to be called within RAMM. The only user input required would be the section of state highway of interest (ie SH, and start and end locations of the section of the state highway to be analysed).

2 The current (2016) version of RAMM contains both speed and speed limit tables but these have not yet been populated. Having both validated speed and speed limit data would considerably simplify and improve the robustness of both the speed limit setting and annual cost analysis performed by the spreadsheet tool. Therefore, it is recommended priority should be given to populating these two RAMM tables, particularly as they would also be beneficial for any updating of RAMM's curve context table. Ideally, the free vehicle speeds in the speed table would be derived from actual speed measurements of both passenger and heavy commercial vehicles rather than from theoretically derived vehicle speeds.

3 Apart from the Ministry of Transport annual speed survey results, there appears to be very little New Zealand-specific information available in the public domain to assist in quantifying the expected change in operating speed characteristics following a change in speed limit, apart from weather activated variable speed limit trials in the Kaimai ranges. Therefore, a key element missing from the research is some hard evidence of what happens to speed profiles over a tortuous route when the 100 km/h speed limit is reduced. There is a valid concern when lowering speed limits that, while the mean and 85th percentile speed appropriately lower, the variability in speeds increase and the upper 15 percentile speeds remain high. If SCRIM ILs are lowered over time, for example to 0.4 ESC, there will still be a significant proportion of drivers travelling at the previous operating speeds (at least initially without significant additional enforcement) that could result in crash rates not decreasing or remaining the same but potentially increase as ESC levels drop over time. Therefore, additional research is needed to improve our understanding of the relationships between operational traffic speed characteristics and speed limits for the six functional categories of the ONRC adopted by the Transport Agency.

4 Until the RAMM speed table is populated, it is recommended estimates of mean and 85th percentile speeds should be based on road geometry with appropriate limits placed on average acceleration and deceleration rates and steep (>8%) uphill and downhill grades. These limits should be validated for New Zealand conditions, and ideally related to the functional categories of the ONRC.

5 The existing RAMM curve context table needs to be updated as it has been based on 2010 geometry data and curve entry and departure speeds that assume near instantaneous acceleration/deceleration. The opportunity should also be taken to incorporate 'wet road' curve crash risk modelling to supplement the existing 'all' curve crash risk modelling so the assigning of curve investigatory skid resistance levels and route speed limits can be made on the basis of wet road conditions, rather than all road conditions as at present.

6 The spreadsheet tool has been formulated to calculate the operating speed over the route specified by the user that results in the same level of cumulative curve crash risk for a skid resistance level of 0.4 ESC as if the skid resistance of each curve was at the recommended IL. A suggested refinement is that the route is automatically divided into shorter subsections, to explore how variable the desired operating speed is over the route. The minimum subsection length would be 1,000 m to allow compliance with Land Transport Rule 5400 for the setting of speed limits. This would help the user determine whether the route selected was appropriate or if it would be better to split the route up into shorter sections with potentially different speed limits applying. When splitting the route into subsections, care would be needed to ensure no curve spans over two adjacent subsections.

`

7 References

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Appendix A: RAMM table layouts

The expected layouts of the extracted RAMM tables are as follows:

A1 Geometry table

A	B	C	D	E	F	G	H	I
Road name	Start	End	Lane	Survey	Reading date	Gradient	Crossfall	H curvature

A2 Curve context table

A	B	C	D	E	F	G	H	I
Road name	Road ID	Start	End	Curve radius	Curve direction	Curve direction	Grad incr	Grad dec

J	K	L	M	M	O
Approach speed increasing dire	Approach speed decreasing	Curve speed	Aadt	Adverse crossfall	Absolute crossfall

P	Q	R	S
Recommended IL	Curve classification	Estimated personal risk	Estimated collective risk

A3 Modified curve context table:

In order to manually run the spreadsheet tool, the RAMM context table has to be reorganised as below with the urban/rural field manually populated.

A	B	C	D	E	F	G	H	I
Road name	Road ID	Start	End	Curve radius	Grad incr	Grad dec	App speed inc	App speed dec

J	K	L	M	N
Curve speed	AADT	Region	Rec IL	Urban/rural

Appendix B: SQL for extracting RAMM data

The SQL listed below outputs data used for the speed-vs-IL trade-off calculations. This SQL outputs data in four tabs:

1. Geometry data for 'I' direction lanes.
2. Geometry data for 'D' direction lanes.
3. Inputs for Matlab's implementation of the crash prediction model.
4. Data on speed limit signs. (This information is used for assessing the magnitude and location of speed limit changes.)

```
==========================================================================================
{5-29E30.00
RJH
from 01.05.2017}
--------------------------------------------------------------------------------
{instructions:
{- *** ALTER LIST IN LINES 28 TO 34 THE SH-RP's YOU WANT TO ANALYSE ***}
{- set 'Y->Yes' button to off (i.e. no change in shading from the other buttons}
{- run this sql and copy results (4x tabs) to excel}
--------------------------------------------------------------------------------
{to do:
- nothing}
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
set LANGUAGE BRITISH;
--------------------------------------------------------------------------------
if OBJECT_ID('tempdb.dbo.#tmp_roads_to_analyse') is not null
drop table #tmp_roads_to_analyse;
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
select distinct roadnames.*, carr_way.cway_area
into #tmp_roads_to_analyse
from roadnames, carr_way
where roadnames.road_id = carr_way.road_id
and right(roadnames.road_name,2) not in ('-I', '-D')
and carr_way.road_name in ('054-0000', '060-0118');
{*** ALTER THIS LIST TO SH-RP LIST YOU WANT TO ANALYSE ***}
--------------------------------------------------------------------------------
select #tmp_roads_to_analyse.road_name,
hsd_geometry.start_m, hsd_geometry.end_m, hsd_geometry.lane,
hsd_geometry.survey_number, hsd_geometry.reading_date, hsd_geometry.gradient,
hsd_geometry.crossfall, hsd_geometry.curvature {i.e. horizontal curvature}
from #tmp_roads_to_analyse, hsd_geometry
where
#tmp_roads_to_analyse.road_id = hsd_geometry.road_id
and left(hsd_geometry.lane,1) = "L" {increasing direction}
and hsd_geometry.latest = "L"
order by #tmp_roads_to_analyse.road_name asc, hsd_geometry.start_m asc, hsd_geometry.lane
asc;
--------------------------------------------------------------------------------
select #tmp_roads_to_analyse.road_name,
hsd_geometry.start_m, hsd_geometry.end_m, hsd_geometry.lane,
hsd_geometry.survey_number, hsd_geometry.reading_date, hsd_geometry.gradient,
hsd_geometry.crossfall, hsd_geometry.curvature {i.e. horizontal curvature}
from #tmp_roads_to_analyse, hsd_geometry
where |
#tmp_roads_to_analyse.road_id = hsd_geometry.road_id
and left(hsd_geometry.lane,1) = "R" {decreasing direction}
and hsd_geometry.latest = "L"
order by #tmp_roads_to_analyse.road_name desc, hsd_geometry.start_m desc,
hsd_geometry.lane desc;
--------------------------------------------------------------------------------
select
#tmp_roads_to_analyse.road_name,
ud_curve_context.road_id, ud_curve_context.start_m, ud_curve_context.end_m,
ud_curve_context.curve_radii, ud_curve_context.grad_incr,
```

```
ud_curve_context.grad_dec, ud_curve_context.approach_speed_inc,
ud_curve_context.approach_speed_dec, ud_curve_context.curve_speed,
ud_curve_context.aadt,
{nzta/tnz administration region: a number between 1&7}
administration_region =
case #tmp_roads_to_analyse.road_region
  when 1 then 1
  when 2 then 1
  when 3 then 2
  when 4 then 2
  when 5 then 3
  when 6 then 3
  when 7 then 4
  when 8 then 4
  when 9 then 5
  when 10 then 5
  when 11 then 6
  when 12 then 6
  when 13 then 7
  when 14 then 7
end, {case}
ud_curve_context.rec_il,
'R' as urban_rural {the matlab OoCC software reports only 'Rural' curves}
from #tmp_roads_to_analyse, ud_curve_context
where
#tmp_roads_to_analyse.road_id = ud_curve_context.road_id
order by #tmp_roads_to_analyse.road_name asc, ud_curve_context.start_m asc;
--------------------------------------------------------------------------------
select
#tmp_roads_to_analyse.sh_state_hway, #tmp_roads_to_analyse.sh_ref_station_no,
#tmp_roads_to_analyse.road_name, #tmp_roads_to_analyse.road_id,
sign.location, sign.sign_type, sign.side, sign.offset,
replace(replace(sign.legend_note, '[', ''), ']', '') as legend,
#tmp_roads_to_analyse.sh_ref_station_no*1000 +
  sign.location as cumulative_location_in_m
from #tmp_roads_to_analyse, sign
where
#tmp_roads_to_analyse.road_id = sign.road_id
and (
  sign.sign_type in ('RG1', 'RG2')
  or
  left(sign.sign_type,4) = 'RG1-'
  )
and sign.offset >= 0
and sign.offset <= 10
and sign.side = 'L'
and sign.replace_date is NULL
order by #tmp_roads_to_analyse.road_name, sign.location;
```

Appendix C: Glossary

AC	asphaltic concrete
ADT	average daily traffic
ANRAM	Australian National Risk Assessment Model
ARRB	Australian Road Research Board
AusRAP	Australian Road Assessment Program
CAS	Crash Analysis System (NZ Transport Agency)
CMF	crash modifying factor
CO_2	carbon dioxide
DOT	Department of Transportation (US)
EDAS	electronic driver assistance systems
EEM	*Economic evaluation manual* (NZ Transport Agency 2016)
ESC	equilibrium SCRIM coefficient
FHWA	Federal Highway Administration (an agency of the US DOT)
GPS	global positioning system
HMA	hotmix asphalt
HSD	high-speed data (ie road condition and geometry data collected by automatic means at normal traffic speeds)
IHSDM	Interactive Highway Safety Design Model
IL	(SCRIM) investigatory level
LoC	loss of control
MS	Microsoft™
MoT	Ministry of Transport
nd	no date (date unknown of a publication)
NHTSA	National Highway Traffic Safety Administration (US)
NMA	network management area
NOC	network outcomes contract
NPV	net present value
NZ	New Zealand
OOCC	out of context curve effect, ie difference between approach and curve speeds
OGPA	open graded porous asphalt
ONRC	One Network Road Classification
RAC	[The United Kingdom's] Royal Automotive Club
RAMM	Road Assessment and Maintenance Management System
RP	route position
RS	reference station
SCRIM	sideways-force coefficient routine inspection machine
SH	state highway
SLAG	Seal Life Advisory Group
SMA	stone mastic asphalt

SQL	structured query language
Transport Agency	New Zealand Transport Agency
TRL	Transport Research Laboratory (a UK based organisation)
US	United States of America
USA	United States of America
VOC	vehicle operating cost/s
VPD	vehicles per day
VTPI	Victoria Transport Policy Institute (Canada)

Appendix D: Spreadsheet tool for investigating operating speed/road surface skid resistance trade- offs

Appendix D can be accessed at www.nzta.govt.nz/resources/research/reports/636.