A PRIORITISATION SCHEME FOR THE SAFETY MANAGEMENT OF CURVES

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ABSTRACT

This paper reports an attempt to fit a Poisson linear/log-linear crash prediction model to curves on New Zealand's rural state highways with horizontal radius less than 500m for use in priority assessment and ranking of improvement schemes. The key predictors of crash risk were found to be the curve speed, the difference between the approach speed and the curve speed, the curve length, the annual daily traffic (ADT), the skid resistance of the road surface, and the approach gradient averaged over 100m before the start of the curve. Of all these parameters, the curve length term dominates the magnitude of the predicted curve crash risk.

The model was applied to the entire rural state highway network of New Zealand to gauge the level of agreement with observed loss of control crashes on curves and to assess its potential to assist in the safety management of curves. Loss of control on curves remains the largest cause of crashes on rural state highways with a large proportion of these (over 1200 per annum) occurring on wet roads. The number of curves investigated amounted to 18,771 of which 11,800 (65%) were classified as having a high friction demand.

The paper discusses how the prioritisation scheme evolved and the associated cost-benefit implications.

Key Words: crash-risk modelling, curves, skid resistance, road safety management

INTRODUCTION

Loss of control on curves remains the largest cause of crashes on New Zealand's rural state highways, comprising 1309 reported injury crashes in 2009. This represents 49% of reported injury crashes on rural state highways and 36% of all reported injury crashes. Of these 1309 crashes, 1210 (92%) occurred on curves classified by New Zealand Police as either moderate or easy and 471 (36%) occurred in wet conditions.

Since 1997/98, with the issuing of the T10 specification for skid resistance investigation and treatment selection, curves with a horizontal radius of curvature less than 250 metres have been effectively managed to a skid resistance level that is 25% greater than for all other curves on rural state highways. This was a consequence of the T10 specification, which aimed to equalise the risk across the state highway network of a skidding crash in the wet by assigning investigatory skid resistance levels (in terms of equilibrium skid resistance, ESC) for different site categories, which are related to different friction demands. A description of these site categories and associated investigatory levels (IL) are summarised in Table 1 below. As can be seen, curves below 250 m horizontal radius of curvature are assigned a higher IL than curves with a horizontal curvature of radius 250 m or greater.

By incorporating the concept of a "threshold level" (TL) for skid resistance, the policy effectively sets a minimum level of service. The TL is the trigger level at which urgent remedial work should be undertaken. The TL is currently set at 0.1 ESC below the IL. In practice, the policy results in curves below 250 m horizontal radius of curvature being immediately investigated and treated when the skid resistance falls below the TL of 0.4 ESC. Curves equal or greater than 250 m horizontal radius of curvature are immediately treated only when the skid resistance falls below the TL of 0.3 ESC.

Table 1: T10:2002 skid site categories

Site Category	Description	Notes	Investigatory Level (ESC)
5	Divided carriageway		0.35
4	Normal roads	Undivided carriageways only.	0.4
3	Approaches to road junctions. Down Gradients 5% -10%	Includes motorway on/off ramps	0.45
2	Curve < 250m radius Down Gradients > 10%		0.5
1	Highest priority	Railway level crossing, approaches to roundabouts, traffic lights, pedestrian crossings and similar hazards.	0.55

As not all small radius curves constitute a safety hazard, nor do all moderate to large radius curves have a low crash risk, statistical modelling was undertaken to allow estimation of crash risk for any curve less than 500 m horizontal radius on New Zealand's rural state highways (Brodie et al, 2009).

This paper covers subsequent refinements to this crash risk model for curves associated with the addition of an approach gradient term and making curve length a linear term; the application of the model to assist in the safety management of curves; and an investigation of the relationships between curve radius, personal and collective risk, and crash severity to identify where effort should be focussed.

CURVE IDENTIFICATION

Curves on rural state highways were identified using 10 m horizontal curvature data in the "high speed" (HS) geometry table found in the NZ Transport Agency's Road Assessment and Maintenance Management (RAMM) database. After a number of iterations, the following set of rules was settled on.

1. What Constitutes a Curve?

Curves are defined as consisting of at least three sequential 10 m segments in the same lane which have a 30 m rolling average radius less than the threshold of 500 m and the sign of the radius is the same for all three segments. For simplicity, this is referred to as the curve apex (see figure 1).

2. Start and End Points

For a lane, the start and end of a curve is when the average radius value over three consecutive 10 m readings (recorded at the middle reading i.e. the 30 m rolling average comprises the 10 m section before, the 10 m section under consideration, and the next 10 m section) is greater than 800 m. This takes account of the curve transition/spiral and the braking zone leading into a curve.

For the carriageway, the start point of a curve is the lane curve start location with the lower chainage and the end point is the lane curve end location with the higher chainage.

3. Compound and Reverse Curves

If there is more than one instance within the length of the curve where condition (1) is met (i.e. three sequential 10 m segments where the 30 m rolling average radius is less than 500 m and all are of the same sign), these are to be treated as part of one large curve provided the following condition is met:

> *For one of the lanes, there are no instances throughout the length of the curve where there are more than 2 sequential 10 m segments with a 30 m moving average radius greater than 800 m. For simplicity a gap is defined as being a 10 m segment whose 30 m moving average radius is greater than 800 m. Therefore it is possible to have one or several 10 m or 20 m gaps in one lane of a compound or reverse curve for this condition to be met. In other words, gaps of 10 m or 20 m can be ignored.*

Provided this condition is met, any size gap can be tolerated in the other lane.

A compound curve is when the sign of curvature at the apexes doesn't change throughout the curve length. A reverse curve is when there is a change in the sign of curvature between successive apexes.

4. Start and End Points of Reverse Curves

The point where one curve ends and the next curve begins is defined by splitting the difference between:

i. the latest point in the first curve where the curvature is in the same direction and the 30 m moving average radius is less than or equal to 800 m in both lanes; and

ii. the earliest point in the second curve where the curvature is in the same direction and the 30m moving average radius is less than or equal to 800 m in both lanes.

If the split point is found to be in the centre of a 10 m section, the displacement is round down, i.e. the split 10 m section is added to the second curve.

5. Minimum Separation Distance Between Curves

A curve is regarded as being isolated if the length of break (i.e. radius greater than 800 m) between curves is 20 m or greater.

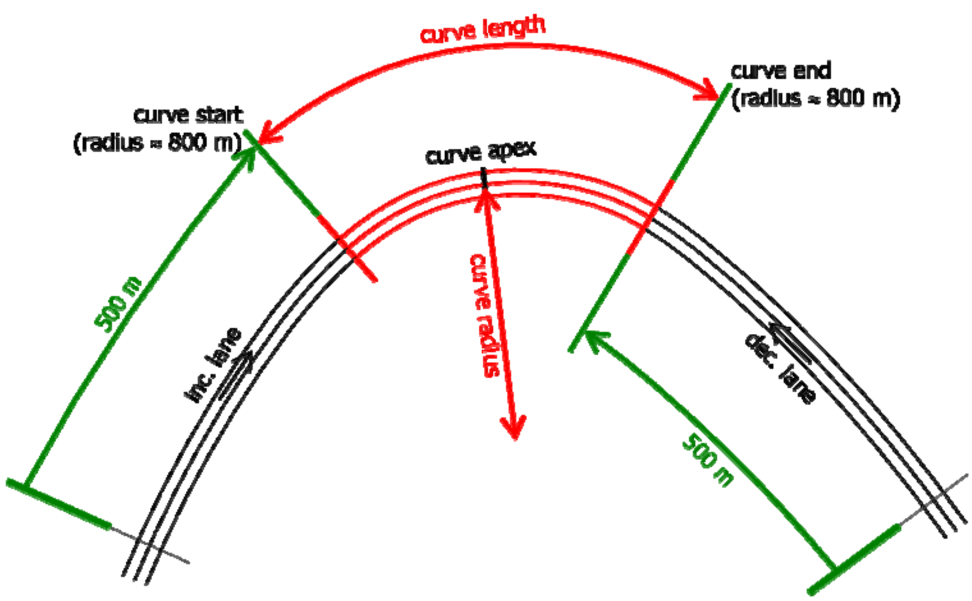


Figure 1: Schematic of curve

The above curve identification process was validated by comparing the derived curve extents with those manually determined from true tangent points (i.e. where there is the first indication of deviation from the straight approach) for curves located at SH5/RS29, SH30/RS158 and SH30/RS170. A total of 55 curves, including a number of compound and reverse curves were used in this validation exercise. In the majority of cases, the start and end locations agreed to 20 m or closer.

A program has been written in Matlab®, version R2007b, to automatically generate a report of rural curves with a radius of less than 500 m. The program can process curves on both divided and undivided carriageways. It also flags curves whenever the location of the increasing lane apex is 40 m or greater than the location of the decreasing lane apex to indicate a possible concern with the geometry data. There is an expectation that the difference in curve location between increasing and decreasing directions should not be greater than 10-20 m in the majority cases as surveys of road geometry made since 2009 have employed a GPS based location referencing system so the start point is common for both increasing and decreasing directions.

Application of the above curve identification process to New Zealand's rural state highways yielded the following results:

- There are a total of 18,771 curves with a horizontal radius of curvature less than 500 m, of which about 65% (11,800) are < 250 m radius.
- The combined length of < 250 m radius curves amounts to 1699.02 km and the curves between 250 m and 500 m radius amounts to 1138.04 km. This gives a total of 2837.06 km, which equates to about 26% of the entire state highway network.
- The average length of a <250 m radius curve is 144 m compared to 163.3 m for curves between 250 m and 500 m radius.

OBSERVED CRASH RISK ON CURVES

The T10 skid site categories given in Table 1 have mirrored those adopted in UK's skid resistance policy. Recent TRL research (Viner et al, 2005) has resulted in the UK extending their "curve" category from less than 250 m radius curves to less than 500 m radius curves. To establish if this may equally apply to New Zealand's rural state highways, crash data for curves with radius less than 500 m radius were investigated.

An analysis of injury crash data over the 5 year period 2004 – 2008 extracted from the Ministry of Transport's Crash Analysis System (CAS) indicates that about 1000 injury crashes per year occur on curves less than 500 m radius located on rural road sections of the state highway network, with about 37% of these on curves with a radius between 250 – 500 m.

With reference to Figure 2, the average annual crash number is shown as a function of curve radius. This plot shows that the T10 horizontal radius of curvature demarcation of < 250 m correctly identifies curves with high crash numbers. However, high crash numbers continue for curve radii up to 350 m radius and so considerable safety benefits are likely to accrue by extending the demarcation from less than 250 m radius to less than 500 m radius and by better matching IL's to friction demand.

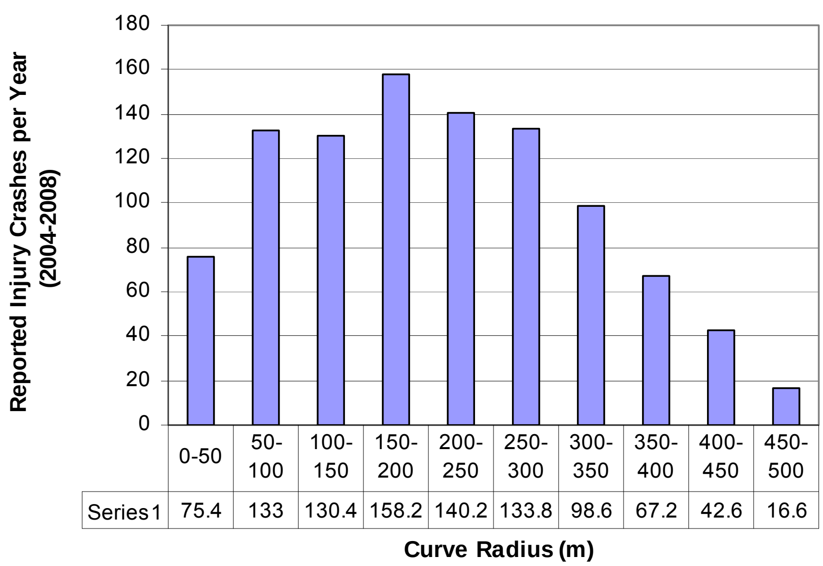
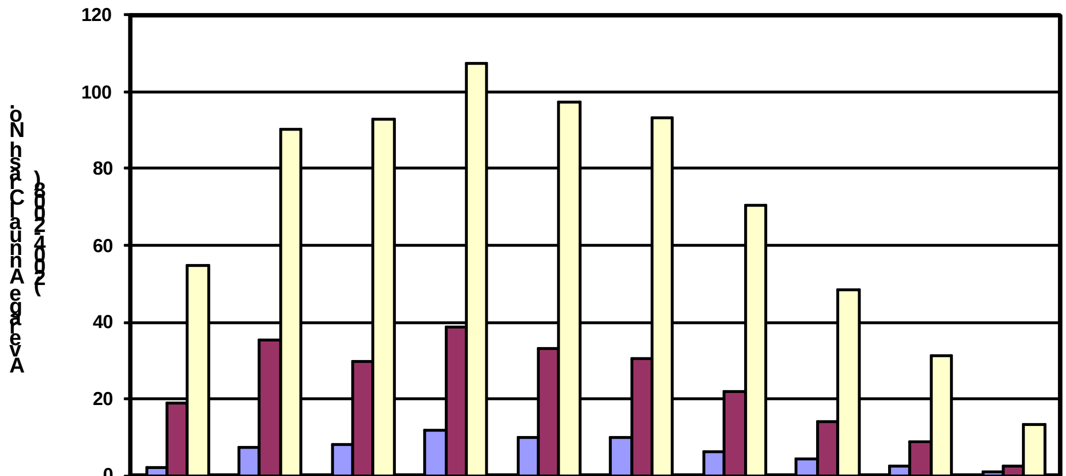


Figure 2: Relationship between curve radius & average injury crashes per year

Figure 2 shows that there are actually more crashes in the 250 m - 350 m radius range (232.4) than in the 0 m – 100 m radius range (208.4) and almost as many in the 250 m – 400 m radius range (299.6) as in the 0 - 150 m radius range (338.8). Consequently, there is as strong an argument for better managing skid resistance on curves over 250 m radius as there is for managing curves below 150 m radius.

The histogram plot given in Figure 3 breaks down the curve related crashes in terms of severity. This shows that about 38% of fatal crashes and 33% of serious injury crashes occur on curves with a radius between 250 – 500 m.

Again, more fatal crashes occur in the 250 m to 400 m radius range (20.4 fatal crashes) than in the 0 m to 150 m range (17 fatal crashes).



	0-50	50-100	100-150	150-200	200-250	250-300	300-350	350-400	400-450	450-500
Fatal	2	7.2	8	11.8	10	10	6.2	4.4	2.6	1
Serious Injury	18.8	35.4	29.6	38.8	33	30.6	22	14.2	8.8	2.4
Minor Injury	54.6	90.4	92.8	107.6	97.2	93.2	70.4	48.6	31.2	13.2

Curve Radius (m)

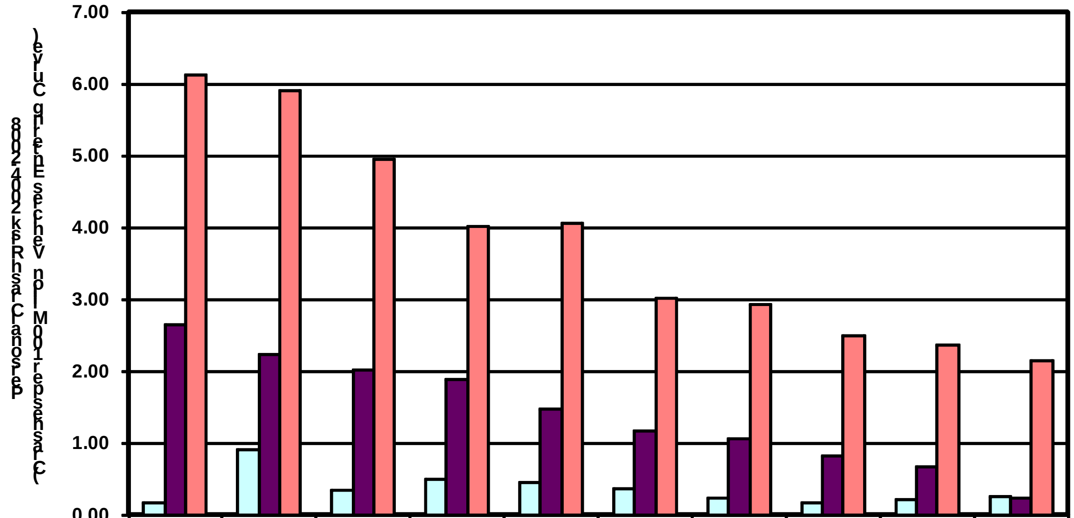
Figure 3: Relationship between curve radius & collective risk by crash severity

The relationship between curve radius and average personal risk for rural state highway curves less than 500 m radius is shown in Figure 4. For curves, personal risk is defined as the number of crashes per 100 million vehicles entering the curve.

With reference to Figure 4, personal risk decreases monotonically with increasing curve radius for serious and minor injury crash types. By comparison, personal risk is relatively constant for fatal crashes at about 0.3 fatal crashes per 100 million vehicles entering the curve apart for a peak of 0.86 fatal crashes per 100 million vehicles entering the curve, which occurs for curves with a horizontal radius of between 50 and 100 m, and a lesser peak at around 0.48 fatal crashes per 100 million vehicles entering the curve, which occurs for curves with a horizontal radius of between 150 m and 250 m.

Figure 5 combines the crash number versus curve radius distribution of Figure 3 with the crash rate versus curve radius distribution of Figure 4, but in this case only all reported injury crashes are considered. This combined plot clearly shows that personal risk is highest at the smaller curve radii. However, greater crash numbers would be targeted by the larger curve radii.

Figure 5 confirms the potential to realise significant crash reductions through improved safety management of larger radii curves.



	0-50	50-100	100-150	150-200	200-250	250-300	300-350	350-400	400-450	450-500
Fatal	0.17	0.90	0.33	0.50	0.44	0.36	0.22	0.17	0.21	0.25
Serious Injury	2.65	2.23	2.00	1.88	1.46	1.16	1.06	0.81	0.65	0.22
Minor Injury	6.12	5.89	4.94	4.01	4.06	3.00	2.93	2.49	2.36	2.15

Curve Radius (m)

Figure 4: Relationship between curve radius & personal risk by crash severity

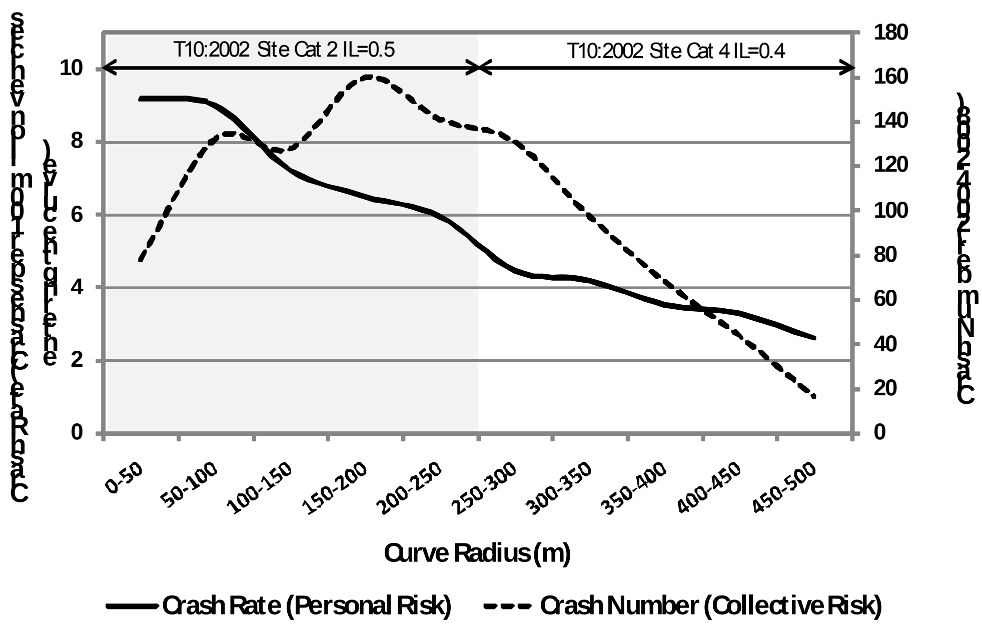


Figure 5: Relationship between curve radius, personal risk and collective risk for reported injury crashes

CRASH RISK MODEL FOR CURVES

The dataset used for deriving the crash prediction model for curves covered the period 1997 to 2002 and was the same as described in Davies et al (2005). The dataset comprised a total of 95435 curve-years (i.e. one line of data for each curve for each year) and 3244 crashes (all reported injury, including fatals). Curves at intersections were excluded from the dataset.

The statistical modelling attempted to fit the number of crashes in each curve (and in each year of the analysis period) to geometric elements of the curve, exposure (i.e. average daily traffic, ADT) and the difference between the approach and the curve speeds (i.e. out of context curve, OOCC).

Previous New Zealand research (Koorey and Tate, 1997) had identified that the risk and severity of crashes on curves was not only a function of absolute curve radius but also the difference between the approach speed and the curve speed. Furthermore, the crash rate was shown to increase significantly when the difference between the approach speed and curve speed exceeded 15 km/h. Therefore, determination of approach and curve speeds was seen as a critical input to the statistical modelling.

The approach and curve speeds can be reasonably determined by inputting 10 m radius and crossfall data from the geometry table in the NZ Transport Agency's RAMM database into the formula below:

$$AS = -\left(\frac{107.95}{H}\right) + \sqrt{\left(\frac{107.95}{H}\right)^2 + \left[\frac{127{,}000}{H}\right]\left[0.3 + \frac{X}{100}\right]}$$

where: AS = Advisory Speed (km/h)

X = % Crossfall (sign relative to curvature)

H = Absolute Curvature (rad/km) = 1000/R

R = Horizontal Radius of Curvature (m)

Tate and Turner (2007) tested a range of variables and among other things identified a strong correlation between curve crashes and the difference between the approach speed over a 500 m length and the minimum curves speed over a 30 m length.

For the statistical modelling, the approach speed in the increasing lane was defined as the average of the advisory speeds from 500 m prior to the start of the curve to the start of the curve. The approach speed in the decreasing lane was defined as the average of the advisory speeds 500 m prior to the end of the curve to the end of the curve. For the 500 m lead-in of an analysis, for which there is no data in either lane, the advisory speeds were assumed to be equal to 110 km/h, which is 10% above the open road speed limit. Setting AS to the maximum expected speed of 110 km/h ensures that calculated differences between approach speed and curve speed will err on the high side for situations when geometry data is not available over the entire 500 m lead in. The schematic in Figure 1 shows the position of the lead-in relative to the start of the curve.

The curve speed was defined as the minimum 30 m averaged advisory speed over the length of the curve. The 30m average is derived from the advisory speed calculated for the current and preceding two 10 metre sections.

For the rural environment, AS was capped at 110 km/h and for the urban environment it was capped at 70 km/h.

An issue with the statistical modelling is the difficulty in allowing for the errors in the location of the crashes. A partial solution has been to assign each crash that occurs within 50 m of a curve to that curve. Where this would result in a crash being assigned to two curves, it has been assigned to the one nearest to the curve.

A modification of a Poisson linear/log-linear model was fitted to the data. The modelling assumed that each side of each curve can generate crashes at the rate (per year) according to the following relationship:

$$a \times L_1 \times \exp(L_2)$$

where a is the average daily traffic (ADT) per lane and 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 (i.e. difference between the approach and curve speeds)
- curve speed (AS)
- skid resistance (SCRIM)
- approach gradient (gradient_app)
- $\log_{10}$(ADT)
- year
- NZTA administration region

Therefore, the fitted model is a combination of the linear model (the L_1 part) and the log-linear model (the L_2 part). The coefficients in the L_1 and L_2 linear combinations are the unknown parameter that had to be estimated.

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:

$$\frac{10^8}{365} \times L_1 \times \exp\left(L_2\right)$$

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.

Table 2 summarises the results of the analysis of variance of the model fit. To assist the fitting process, the 50 percentile value has been subtracted from the non-categorical variable apart from the approach gradient. Third degree polynomial transforms have been used for OOCC, curve speed and $\log_{10}$(ADT), whereas a second degree polynomial transform has been used for SCRIM skid resistance, approach gradient and square root of curve length.

The column SS(3) in Table 2 gives the chi-square value when the corresponding term is the last one added to the analysis and SS(1) gives the chi-square value when the terms are included sequentially. When calculating the SS(1) values for the L_1 and L_2 terms, it is assumed that the other linear set of terms has already been fitted.

Table 2: Table of Variance

Term	**Degrees of Freedom (df)**	**Chi-Square**	
		SS(3) (Term Added Last)	**SS(1) (Term added sequentially)**
	L_1		
poly2_((sqrt_lengthR)-15)	2	81.97	81.97
	L_2		
year	5	35.89	28.45
NZTA administration region	6	61.35	89.67
poly3_(OOCC-30)	3	189.66	457.41
poly3_(AS-50)	3	28.03	14.16
poly2_(SCRIM-0.5)	2	63.43	47.63
poly3_(log10(ADT)-3)	3	35.48	35.99
poly2_(gradient_app)	2	13.81	13.81

The 1% and 5% levels of significance are tabulated in Table 3. A comparison with the SS(3) values shows all the fitted variables are statistically significant at the 1% level if the Poisson model was valid. OOCC is shown to be the most significant predictor variable followed by curve length. However, the SS terms for the curve length are under-estimating its importance, probably because of the use of the L_1 and L_2 terms.

Table 3: Levels of Significance

Levels of Significance	Degrees of Freedom					
	1	2	3	4	5	6
5%	3.84	5.99	7.81	9.49	11.07	12.59
1%	6.63	9.21	11.34	13.28	15.09	16.81

It is likely that there is more variability in the data than the Poisson model implies because one can't expect the model to fit perfectly. So one likes the SS values in table 2 to be substantially above the critical levels in table 3 before declaring a term to be statistically significant. Table 4 is the resulting table of effects. Values greater than 2 under the column headed "Ratio" are statistically significant if the Poisson model is believed to be correct. This test should be applied only to the highest degree term in each polynomial.

Table 4: Model Coefficients

Variable	Model Statistics		
	Coefficient	Standard Error	Ratio
L_1:			
constant	1.77E-05	1.80E-06	9.9
(sqrt(lengthR)-15.0)**1	1.61E-06	1.92E-07	8.4
(sqrt(lengthR)-15.0)**2	6.84E-09	1.21E-08	0.6
L_2:			
year:1997	0		
year:1998	-0.02352	0.062	-0.4
year:1999	0.04360	0.063	0.7
year:2000	0.02011	0.063	0.3
year:2001	0.19874	0.061	3.3
year:2002	0.25136	0.061	4.1
region:R1	0		
region:R2	0.13161	0.064	2.1
region:R3	0.38803	0.080	4.8
region:R4	0.40065	0.074	5.4
region:R5	0.28962	0.079	3.7
region:R6	0.33949	0.079	4.3
region:R7	0.43579	0.076	5.7
(OOCC-30.0)**1	0.04387	0.004	10.6
(OOCC-30.0)**2	0.00039	0.000	3.7
(OOCC-30.0)**3	-1.24E-05	4.95E-06	-2.5
(AS-50.0)**1	0.01570	0.003	4.6
(AS-50.0)**2	-9.43E-05	1.71E-04	-0.6
(AS-50.0)**3	-9.87E-07	2.65E-06	-0.4

(SCRIM-0.5)**1	-2.17050	0.273	-8.0
(SCRIM-0.5)**2	-1.14390	2.159	-0.5
(log10_ADT-3.0)**1	-0.05904	0.094	-0.6
(log10_ADT-3.0)**2	-0.17294	0.206	-0.8
(log10_ADT-3.0)**3	-0.08039	0.155	-0.5
(gradient_app)**1	-0.02628	0.008	-3.4
(gradient_app)**2	0.00035	0.001	0.4

Note:

R1 to R7 are the following NZ Transport Agency administration regions:

- *R1=Auckland*
- *R2=Hamilton*
- *R3=Napier*
- *R4=Wanganui*
- *R5=Wellington*
- *R6=Christchurch*
- *R7=Dunedin*

The analysis was repeated for crashes likely to result from loss on control at a curve (i.e. all injury and fatal crashes with the movement codes A, B, C, D or F). However, only a small difference in the model form or model coefficients resulted. This was not surprising given that the selected crashes comprised about 91% of all the reported injury crashes.

PREDICTED EFFECTS ON CURVE CRASH RATES

The following graphs show the effect of the different variables varied one at a time. The variables not being varied have the following values:

Year:	2002
Region:	R2
OOCC:	30(km/h)
AS:	80(km/h)
SCRIM:	0.5(ESC)
ADT:	1000(v/d)
gradient:	0 (%)
length:	100(m)

With reference to Table 5, the quantity being modelled is personal risk in units of crashes per 100 million vehicles entering the curve.

Table 5: 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	0.5	0	-2.17050	0.00E+00
(SCRIM-0.5)**2	0.5	0	-1.14390	0.00E+00
(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
				∑ = 7.42E-01
			Personal Risk:	**5.66**
			Collective Risk:	**0.02**

Note: *Personal risk is in terms of injury crashes per 100 million vehicles entering the curve.*
Collective risk is in terms of annual number of injury crashes per curve.

The solid line in the graphs is the estimate of the effect of the parameter and the dotted lines are the associated 5% confidence intervals.

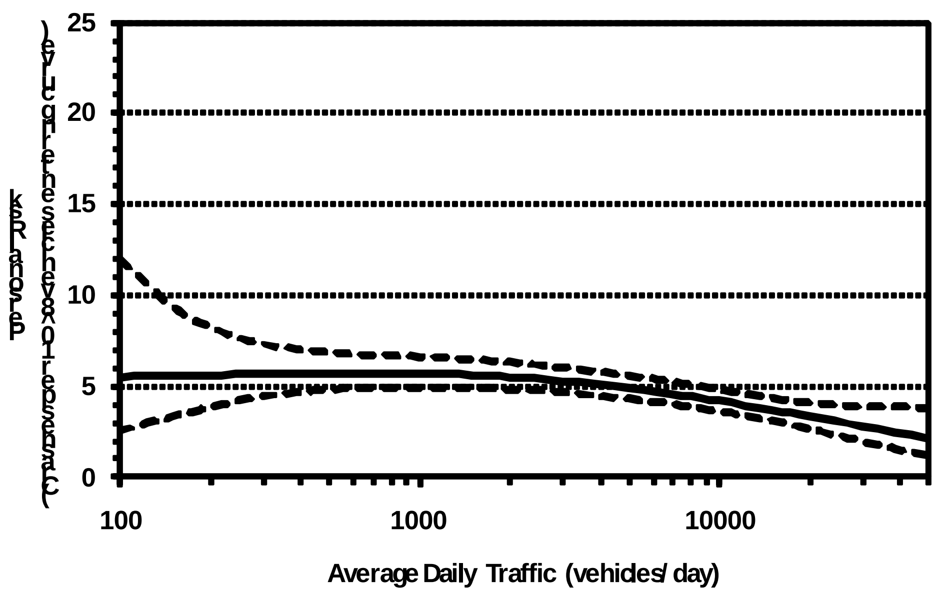


Figure 6: Crash rate versus ADT

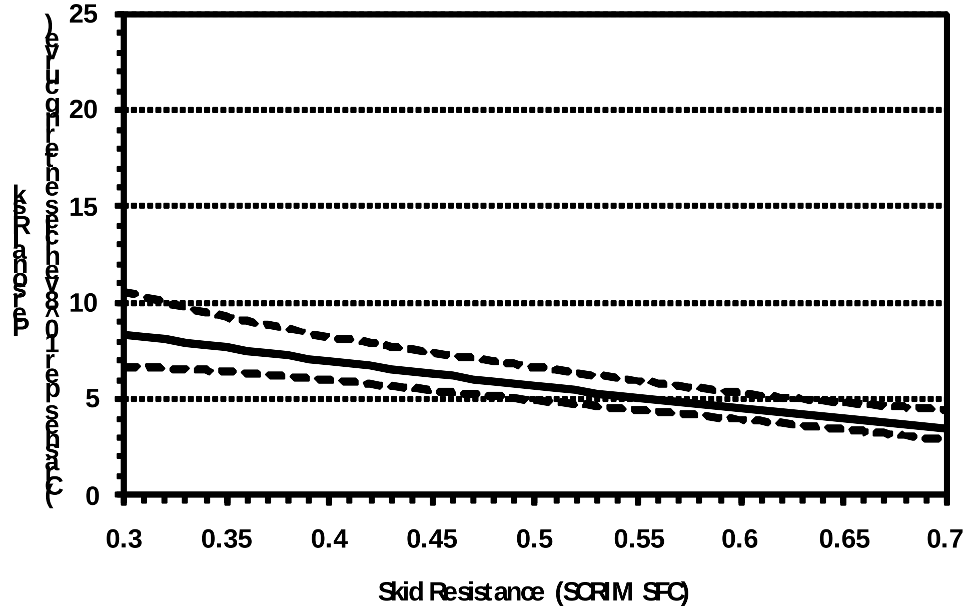


Figure 7: Crash rate versus skid resistance (SCRIM)

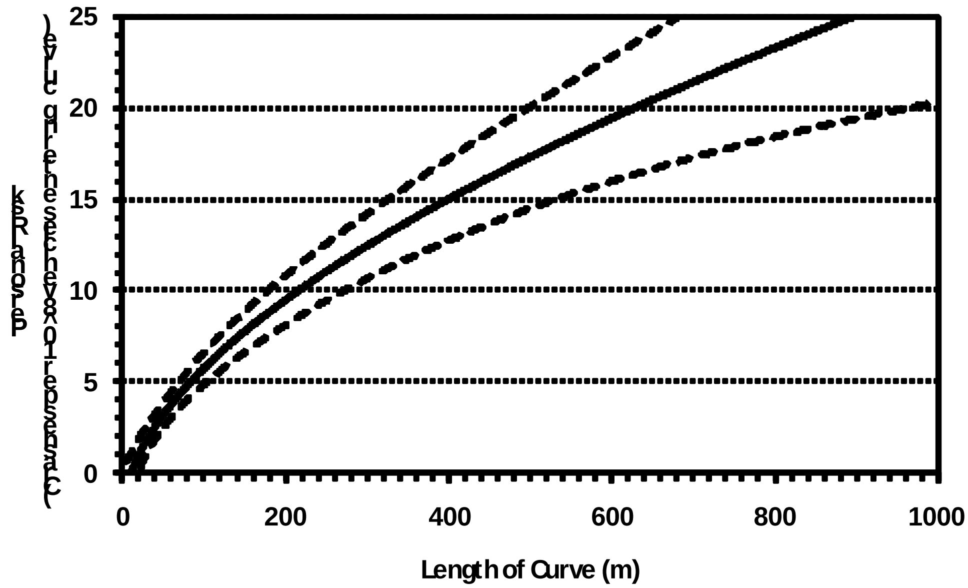


Figure 8: Crash rate versus length of curve

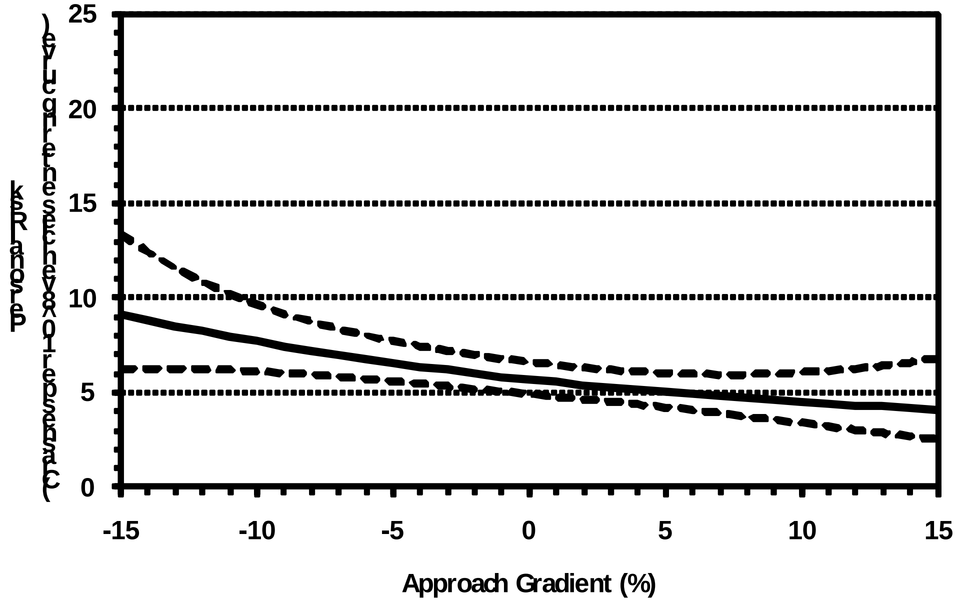


Figure 9: Crash rate versus approach gradient

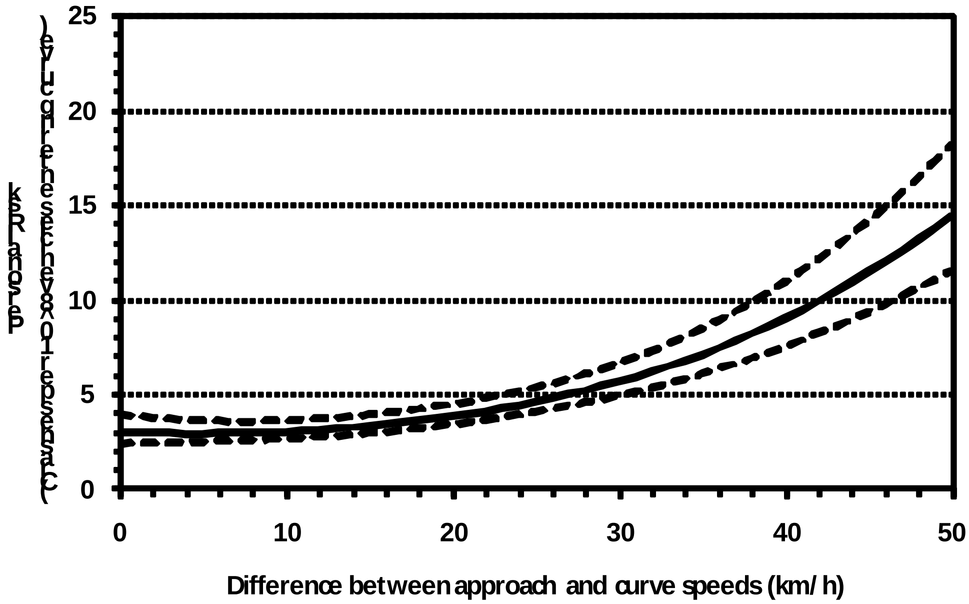


Figure 10: Crash rate versus OOCC

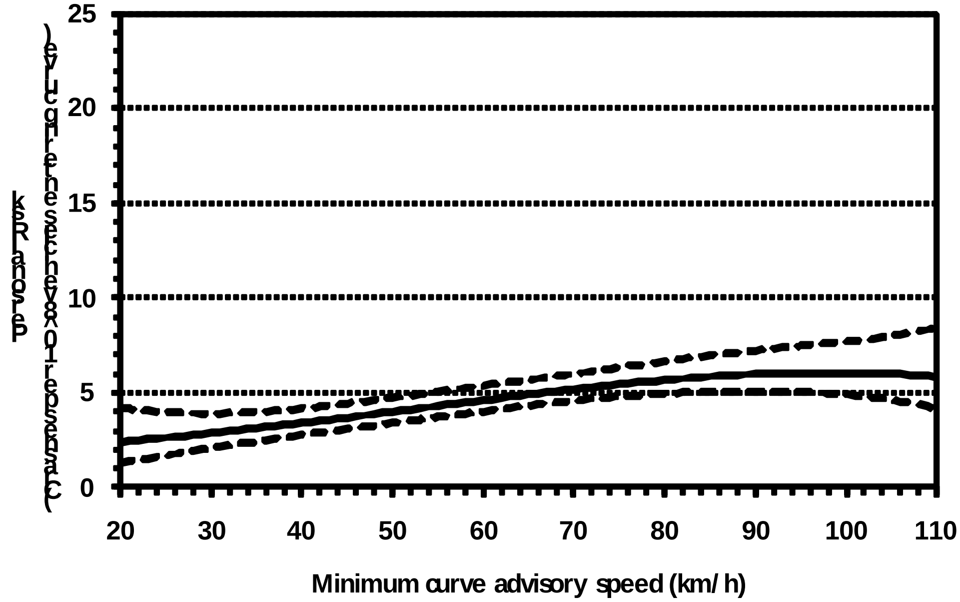


Figure 11: Crash rate versus curve speed

With reference to Figure 11 crash-rate is shown to increase as the curve eases (i.e. AS increases). The reason is that the OOCC variable is being held constant so the approach speed is also increasing.

The effects shown in Figures 6 to 11 are generally consistent with expectation and so this provides a degree of confidence in the derived crash prediction model for curves.

It is difficult to carry out a goodness of fit test on the model. The usual chi-squared goodness of fit test on the actual and fitted values does not work in the present situation because of the small expected number of crashes for most curves.

One approach is to divide the data into categories based on one or two of the predictor variables and then compare the observed and modelled numbers of crashes for each of the categories. Figure 12 shows the comparison when one categorises by length and curve speed.

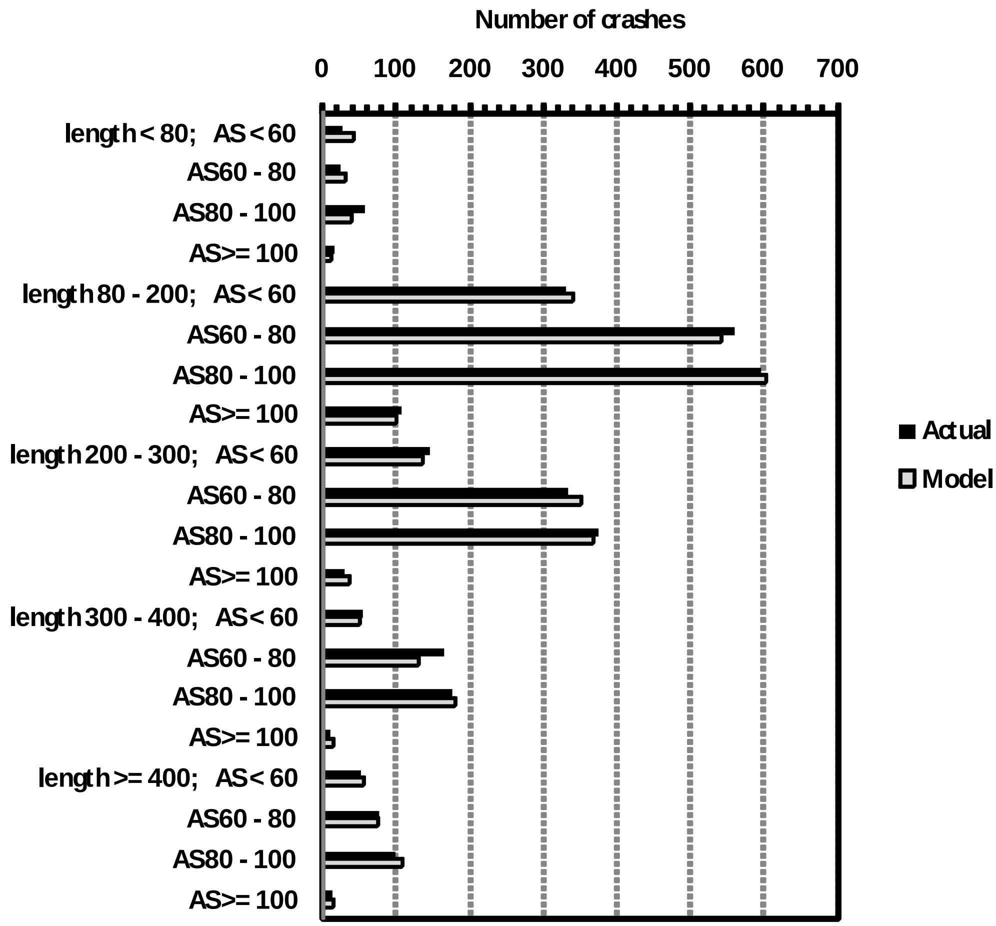


Figure 12: Predicted and actual crash numbers

Mostly, the agreement is good, but there appears to be an interaction between length and curve advisory speed for the shortest curves. The tightest curves are less dangerous than predicted by the model and the straighter ones are more dangerous.

Tests on length and OOCC and on curve speed and OOCC also gave good agreement.

On the basis of these results, the model appears sufficiently robust for prioritising curves for treatment and to investigate the cost-benefit of altering investigatory skid resistance levels.

SUGGESTED PRIORITISATION SCHEME

A cumulative histogram of the predicted personal risk of each state highway open road curve less than 500 m radius is given in Figure 13. The personal risk plotted assumes a skid resistance value of 0.4 ESC, corresponding to the T10 site category 4 investigatory level. This histogram plot was used to select values of personal risk corresponding to low and high risk curves on the basis of the 25 and 75 percentiles.

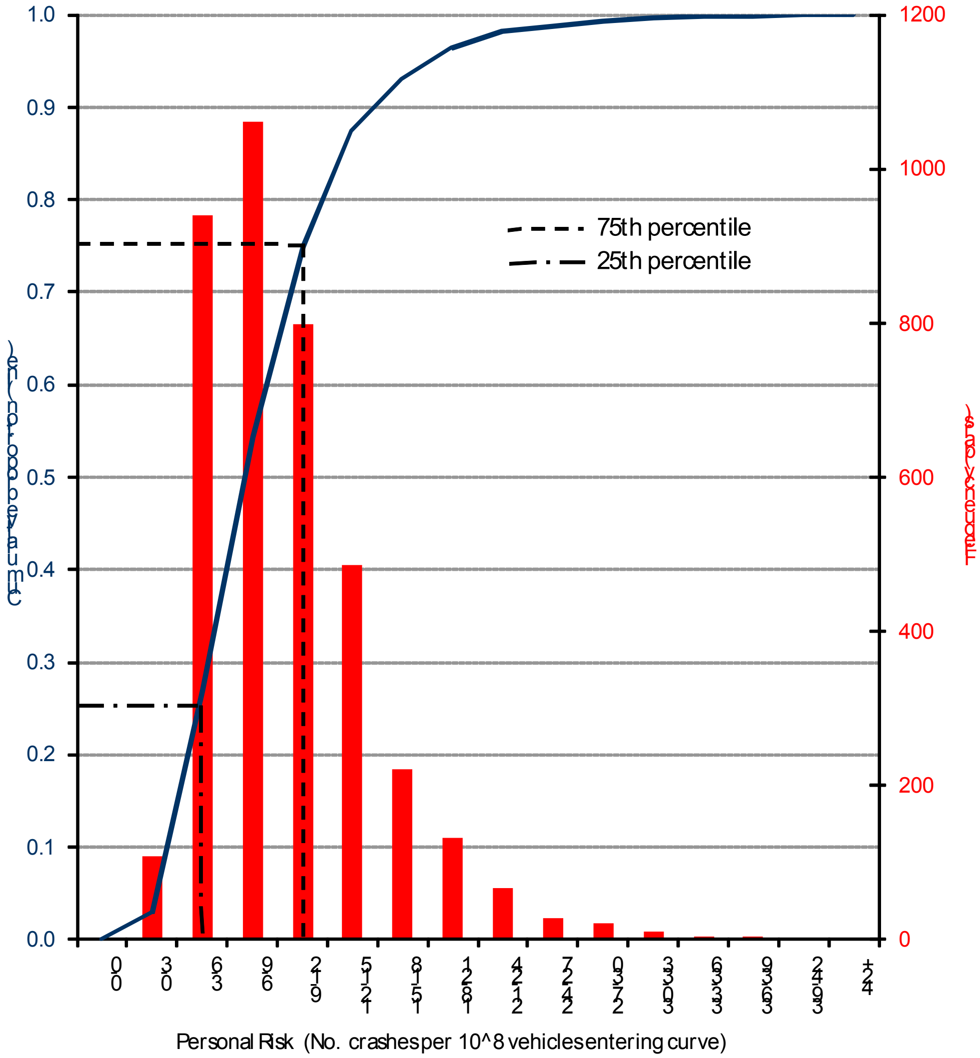


Figure 13: Cumulative histogram of curve personal risk

The prioritisation scheme suggested by Figure 13 is as follows:

- low risk curves, predicted personal risk (PPR) < 7
- medium risk curves, 7 ≤ PPR ≤ 14
- high risk curves, PPR > 14

PPR is in terms of number of injury crashes per 100 million vehicles entering the curve.

A key aspect of any refinement to the way curves are handled by the T10 specification is the improved targeting of curves with a large difference between the approach and curve speeds as this represents a high degree of "out of contextness." However, the curve crash prediction model was found to assign high personal risk to long curves, but not necessarily to those with a large speed difference. Therefore, additional conditions related to the difference between the approach speed and the curve speed were found to be necessary.

The speed difference (OOCC) related reclassifications decided on were as follows:

- If T10:2002 site category 2 curve (R<250m) rated medium risk but OOCC>35 km/h transfer to high risk.
- If T10:2002 site category 2 (R<250m) curve rated high risk but OOCC< 20 km/h transfer to medium risk.

- If T10:2002 site category 4 (250m<R<500m) curve rated high risk but OOCC<15 km/h transfer to low risk.
- If T10:2002 site category 4 curve (250m<R<500m) rated high risk but 15 km/h≤OOCC≤20 km/h transfer to medium risk.

The mapping of curve risk to T10 site category is tabulated below. The largest changes are that a site category 2 curve may have its IL reduced from 0.5 to 0.45 but a site category 4 curve may have its IL increased from 0.4 to 0.55.

Table 7: Suggested Skid Resistance Investigatory Levels for Different Curve Risk

Curve Risk Rating	Suggested T10 IL
High	IL = 0.55
Medium	IL = 0.50
Low, Radius<250m	IL = 0.45
Low, 250≤Radius≤,500m	IL = 0.40

To confirm that the proposed combined curve risk rating and speed difference criteria were correctly identifying high risk curves, the actual crash rates for curves in the various categories were derived from all injury crash data over the 5 year period 2004 – 2008. The expectation was that curves rated as being high risk would have the highest actual observed crash rate. The results from both a NZ Transport Agency administration region and national basis are summarised in Table 8. They are as expected with the high risk curves having over double the crash rate of the medium risk curves and the medium risk curves having over double the crash rate of the low risk curves.

Table 8: Observed Crash Rates for Different Curve Risk Ratings

NZTA Administration Region	Actual Annual Crash Rate Over Period 2004-2008 (injury crashes per 108 vehicles entering curve)			
	Low Risk (IL=0.4)	Low Risk (Il=0.45)	Medium Risk	High Risk
Auckland/Northland	3.51	2.47	7.54	9.50
Hamilton/Tauranga	2.9	1.48	4.87	10.75
Napier	2.62	2.66	6.45	9.19
Wanganui	2.8	0.56	5.55	9.81
Wellington	1.25	3.3	3.95	9.34
Christchurch	1.99	2.35	5.31	11.38
Dunedin	3.5	8.93	6.74	14.98
New Zealand	2.71	2.38	5.39	11.12

APPLICATION TO NZ'S RURAL STATE HIGHWAYS

The results of applying the proposed curve risk rating procedure to the New Zealand's rural state highways are summarised in Table 9.

It can be seen that most of the length of state highway classified as high risk comes from T10:2002 site category 2 curves with a horizontal radius of curvature between 50 m and 200 m. The contribution of T10:2002 site category 4 curves to the length of the state highway classified as high risk is derived largely from curves with a horizontal radius of between 250 m and 350 m.

To assess the cost effectiveness of the proposed curve risk rating procedure, a benefit-cost analysis was performed. The annual cost method was used in preference to the net present value method as this better matched the annual budgeting cycle for maintenance works and eliminated the need to account for deterioration of skid resistance over time, as the analysis takes place over the year the seal is put down.

Table 9: State Highway Lengths by Curve Risk Rating and Curve Radius

Curve Radius (m)	Length of State Highway (km)			
	LOW RISK (IL = 0.40)	**LOW RISK (IL = 0.45)**	**MEDIUM RISK (IL = 0.50)**	**HIGH RISK (IL = 0.55)**
0-50	0	4.16	36.43	77.88
50-100	0	16.01	107.48	196.48
100-150	0	22.43	160.25	202.26
150-200	0	28.14	256.53	152.66
200-250	0	41.14	321.26	75.91
250-300	142.25	0	226.98	33.52
300-350	130.21	0	161.9	9.72
350-400	117.26	0	97.77	0.88
400-450	93.81	0	63.93	0
450-500	37.03	0	22.59	0.19
Total SH Lengths (km)	520.56	111.88	1455.12	749.5

The key assumptions in the benefit-cost analysis were as follows:

- Two coat seals are almost exclusively used on curves less than 500 m radius to counteract variable texture present giving a seal cost: $6 per m2
- Carriageway width: 8.5 m
- Seal Life: Site Cat 2 (R<250m): 5 years, Site Cat 4 (250< R<500m): 7 years
- Additional cost of high PSV aggregate to achieve 0.55 ESC: NZ$0.70 per m2
- 0.5 ESC to 0.45 ESC extends seal life by 1 year
- 0.4 ESC to 0.5 ESC reduces seal life by 2 years
- Rate of return : 8%
- Sinking Fund Deposit Factor (SFDF) (uniform series whose future value is $1) used to annualise total seal costs
- Social cost of injury crashes taken from the economic evaluation manual to be NZ$840,000.

The estimated saving in social costs was estimated to be about NZ$61.5 million for an additional expenditure of NZ$2.4 million per annum in sealing cost, resulting in a benefit-cost ratio of 25.6. This demonstrates that targeted skid resistance management of curves can be a very cost-effective safety measure.

Reducing the cut-off curve radius from 500 m to 400 m was also considered. With reference to Figure 1, reducing the curve radius limit from 500 m to 400 m will only have a minor affect with only 59 out of 996 reported injury crashes being discounted, or about 6%. From Table 9, lowering the cut-off curve radius from 500 m to 400 m reduces the length of state highway affected by the out-of-context-curve initiative from 2,837.06 km to 2,619.51 km, a reduction of 217.55 km or approximately 8%. The majority of this 217.55 km comprises low risk (130.84 km) and medium risk curves (86.52 km) T10:2002 site category 4 curves. Therefore, on this basis it is recommended that for New Zealand, the T10 curve category be increased from 250 m radius to 400 m radius as this will be sufficient to address the majority of curve related crashes on the rural state highway network.

CONCLUDING REMARKS

The T10 specification for skid resistance management and treatment selection was updated in October 2010. On the basis of the analysis presented in this paper, the skid resistance of curves with horizontal radius of curvature less than 400 m will now be managed through the assignment of investigatory levels based primarily on predicted personal crash. A significant reduction in loss of control crashes on rural state highways is anticipated as a result of this change in the way skid resistance of curves is managed, and the RAMM database has now been modified to include a curve table to assist industry in meeting this expectation.

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ACKNOWLEDGEMENTS

The authors acknowledge the support of the NZ Transport Agency, MWH New Zealand Ltd, Opus International Consultants Ltd and Statistics Research Associates in preparing and presenting this paper.

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Colin Brodie has over 35 years roading experience in New Zealand and overseas working for Ministry of Works, Opus and Beca, prior to joining Transit New Zealand in 2005. He has specialised in traffic engineering and road safety for the last 25 years and is presently the Chief Safety Advisor to the Highways and Network Operations Division of NZ Transport Agency. He is New Zealand's representative on the Austroads Road Safety Task Force, Technical Director of the KiwiRAP Road Assessment programme, and a member of the Safe System Culture Change portfolio group. He was involved in the development of the current NZ and Austroads Crash Reduction and Safety Audit procedures and has been responsible for the technical direction of this workshop for the last decade. He has also been a member of the NZ Transport Agency's Skid Technical Advisory Group (STAG) for the past 4 years.

Dr Robert Davies is a director of Statistics Research Associates Limited. He has many years of experience in statistical data analysis including the exploratory analysis of data, graphical presentation of results, interpretation of results for clients, as well as the analysis itself. He has carried out numerous analyses on road surface data and road crash risk for Transit NZ, Transfund Land Transport Safety Authority, Land Transport NZ and the NZ Transport Agency. He is familiar with the types of measurements made on road surfaces, particularly with the high speed data collection. As part of this work he has developed the computer software required to analyse the volume of data such projects involve.

Dr Fergus Tate is the Technical Development Leader for MWH New Zealand. A Charted Professional Engineer, Fergus holds an NZCE, and graduated with a BE in Civil engineering in 1986. In 2001 Fergus was awarded an MSc (Eng) from the University of Leeds and PhD in 2003. These qualifications complement more than 30 years of experience in highway and traffic engineering, and in particular road safety engineering. His research interests include as included investigations into the relationship between highway curve context and crashes, and the relationship between road geometry, drivers speed choice and crashes. Fergus has also helped to developed road infrastructure safety assessment (RISA) methodologies and was a major contributor to the New Zealand KiwiRAP road assessment programme. His other interests include the development of the pedestrian crossing facilities selection guideline, work on minimising pedestrian vehicle conflicts in shared spaces, and studies of social severance.