Modelling of extreme traffic loading effects
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An important note for the reader

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Abbreviations and acronyms

AADT	annual average daily traffic
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
CBR	California bearing ratio
dTIMS	Deighton's Total Infrastructure Management System
EDA	equivalent design axle (this is based on an 80kN axle loading)
ESA	equivalent standard axles
ESAL	equivalent single axle load
FWD	falling weight deflectometer
HCV	heavy commercial vehicle
HCV I	heavy commercial vehicle (type I)
HCV II	heavy commercial vehicle (type II)
HDM-4	Highway Design and Maintenance Model 4
HDM-III	Highway Design and Maintenance Model III
IRI	International Roughness Index
LTPP	long-term pavement performance
NAASRA	National Association of Australian State Road Authorities
NZTA	New Zealand Transport Agency
PMS	pavement management systems
QADT	quad-axle with dual tyres
RAMM	road assessment and maintenance management system
RIMS	road information management systems
RP	route position
RS	reference station
SADT	single axle with dual wheels
SAST	single axle with single wheels
SH	state highway
SNC	modified structural number
SNP	adjusted structural number
TADT	tandem axle both with dual wheels
TAST	tandem axle both with single wheels
TRDT	tri-axles all with dual wheels

Contents

Executive summary

This study was undertaken to establish whether various pavement deterioration models incorporated into the New Zealand – Deighton's Total Infrastructure Management System (NZ-dTIMS) and Highway Design and Maintenance Model 4 (HDM-4) pavement management systems could be modified to reliably predict the condition of a pavement after it had been exposed to sudden extreme traffic loading, which can occur with the commencement of forestry logging, mining or enforced detours. Specifically, the deterioration models considered were for roughness progression (both the NZ-dTIMS and HDM-4 models) and rutting progression (NZ-dTIMS model only).

The research was based on:

- an on-road programme of falling weight deflectometer (FWD) measurements on selected local authority sites
- conversion of the FWD data into modified structural number (SNC) data
- comparison of roughness – structural strength relationships for the selected local authority sites
- assessment of the sensitivities of various pavement deterioration models to changes in traffic volume, traffic volume growth and the percentage of heavy commercial vehicles. Models considered included NZ-dTIMS, HDM-4, the Cenek–Patrick roughness model and the Ball alligator cracking model. Pavement deterioration measures varied with model and included roughness, rutting and cracking.

Conclusions

The comparison of measured structural and roughness data showed that:

- the structural strengths of the test pavements were not normally distributed, but positively skewed, with a lower SNC limit of around 1.8
- if this lower limit was used, the location of, and time (or vehicle passes) expected before pavement failure should be able to be modelled
- there were unexpected indications of a negative correlation between high roughness and low pavement strength, suggesting either spatial matching issues in the datasets or some other unknown issue/factor should be considered.

The assessment of the sensitivities of the pavement deterioration models showed that:

- the roughness pavement deterioration models displayed varying degrees of sensitivity to traffic loading and roughness variables, with most performing better with average roughness changes over longer averaging lengths (300m), but not so well for maximum roughness changes occurring over shorter averaging lengths (ie <300m)
- for the NZ-dTIMS Henning rutting model, when using the input settings used in this research, the relationship between rut depth and time was approximately linear from 10 to 25 years (ie for this period, no rapid exponential increase in rut depth with time was evident) for the first two stages of rutting.

An investigation of the road assessment and maintenance management system (RAMM) database suggests that:

- the consequences of extreme traffic loading may show up in RAMM maintenance cost data, but not in RAMM pavement condition data
- extreme traffic loading must be sustained in duration for its consequences to show in RAMM maintenance cost data.

Recommendations

It is recommended that the relationship between structural strength and roughness be investigated further.

It is also recommended that a wider range of sites, which are exposed to or going to be exposed to extreme traffic loading, need to be identified and monitored. If the existing pavement deterioration models are found not to adequately predict the relationships between structural strength and pavement failure, new models should be developed based on the extreme loading data obtained for these additional sites.

Indications are that employing pavement deterioration models may be the preferable analysis route for detecting the effects of extreme traffic loading rather than attempting to analyse pavement condition and reactive maintenance cost data in RAMM. To enable this, the available pavement deterioration models need to be improved to better model observed localised pavement damage resulting from extreme traffic loading.

Abstract

In New Zealand, premature failure of low volume, low strength state highways and local authority roads has sometimes occurred due to significant changes in heavy commercial vehicle traffic. Current New Zealand pavement deterioration models (eg NZ-dTIMS and HDM) were not designed to simulate these sudden increases in traffic loading and their effects over short distances.

The NZ-dTIMS and HDM models along with other pavement distress models were investigated to establish their suitability for modelling extreme variations in traffic loading. The relationships between measurements of structural strength and pavement condition data were investigated for selected pavements. The sensitivities of some pavement deterioration and pavement distress models to extreme traffic loading were also investigated.

The key finding was that the extreme traffic loading must be sustained for a lengthy duration to show up in RAMM pavement condition and reactive maintenance cost data. This observation indicates that employing pavement deterioration models may be the preferable analysis route for evaluating the effects of extreme traffic loading rather than analysing historical RAMM data. To enable this, the available pavement deterioration models need to be improved to better model observed localised pavement damage that results from extreme traffic loading, particularly edge break.

1 Introduction

1.1 Background

In recent years, premature failure of low volume, low strength state highways and local authority roads has occurred due to a significant change in commercial traffic volumes, usually related to forestry or dairy traffic. As many current pavement condition models were not designed to simulate sudden increases in traffic loading, this premature failure due to increased loading can go unpredicted, causing considerable rehabilitation costs. The aim of this research was to investigate the current NZ-dTIMS and HDM pavement deterioration models to establish their suitability for modelling extreme loading. This report also looked at several alternative distress models and their applicability to the problem.

To establish each model's potential for simulating the effects of extreme loading, sensitivity analyses were performed with relation to traffic loading variables, followed by retrospective application of the models to pavements previously exposed to intense traffic loading. The sensitivity analyses were done to reveal whether the model was capable of recognising significant changes in loading, whereas the retrospective application was to show the model's accuracy in modelling extreme loading.

The majority of all pavement deterioration models incorporate mean pavement structural strength as an input variable. As the mean structural strength is used, the within pavement variation of pavement strength is not accounted for. Although this variability can be calibrated out to a significant degree to allow reasonably accurate network condition predictions, it was hypothesised that maximum pavement deterioration would occur at the same position as minimum structural strength. If this hypothesis could be shown to be correct, the within pavement variance of pavement strength was likely to be an important predictor of pavement deterioration. To investigate the effects of this variation in strength, this research explored the relationship between roughness and structural strength and examined the structural strength distribution.

The results of the investigations are discussed in detail and recommendations made on the most useful models for predicting pavement deterioration due to extreme loading.

1.2 Definition of extreme loading

In the context of this report a typical extreme loading event is considered to cause a two- to five-fold increase in daily equivalent single axle loading (ESAL) mirroring the introduction of forestry or daily traffic to a low-volume rural road or detouring of general traffic.

1.3 Road network organisations' perspectives on extreme traffic loading

A selection of February 2010 to April 2010 email correspondence from Opus International Consultants staff and Inroads staff was reviewed as part of this project. Staff from these two organisations are involved in the day-to-day management of various road networks distributed throughout the North Island of New Zealand for New Zealand road authorities. This correspondence is thought helpful as it highlights issues that are considered by the industry in practice, rather than the research authors' speculation as to what might be considered. Key items of this review are summarised below.

Edge break and road-marking wear might be the first potential maintenance areas resulting from extreme traffic loading. An increase in roughness might be another.

Compensation for such damage to roads resulting from extreme loading can be denied by the organisation responsible for funding roads in New Zealand where the extreme loading results from an emergency outside their control (eg a traffic diversion due to a gunman on the loose or a detour due to flooding/slips on a state highway).

In contrast, compensation for the damage caused to a road by extreme traffic loading resulting from planned and programmed (ie non-emergency) situations (eg logging traffic, construction material transport, dairy traffic or scheduled state highway bridge replacement) is quite different and should/can be legitimately included in project costing.

However, a method for establishing a 'fair' compensation value for use of a bypass subject to extreme loading in such non-emergency cases is likely to be problematic. For example, before-and-after falling weight deflectometer (FWD) measurements potentially suffer from repeatability issues, pavement stiffness modulus changes and dependency on groundwater levels. A more robust compensation value-establishing method which may be better accepted would be to use pavement deterioration models with equivalent single axle (ESA) data. This approach seems attractive, but relies on the use of accurate pavement deterioration models subject to extreme traffic loading – the subject of this research.

Alternatively, visual before-and-after inspections could be done, but such subjective measures for setting compensation values may be open to question. A formal variation on this method might be to use before-and-after assessments of pavement condition via road assessment and maintenance management (RAMM) ratings, supplemented with roughness measurements. (Naturally, if inspections to assess damage to a bypass were decided upon, the cost of these inspections would need to be included in the overall compensation package.)

Finally, a difficulty in using before-and-after visual assessments for establishing compensation values would be in deciding how much the loading of the additional traffic contributed to wear, particularly if the normal level of traffic was appreciable in comparison to the additional traffic. This suggests the use of pavement deterioration models based on actual traffic flow data may be the preferable method for setting compensation values acceptable to all parties.

1.4 Scope of the report

This report, in overview, presents the findings of a study aimed at establishing whether the pavement deterioration and pavement distress models for roughness, rutting and cracking progression incorporated into pavement management systems, such as NZ-dTIMS and HDM-4, could be modified so they reliably predict the condition of a pavement after it has been exposed to sudden extreme traffic loading.

The layout of this report is as follows. Chapter 2 investigates pavement strength on six local authority roads in an attempt to determine a suitable default value for deterioration modelling. Chapter 3 builds on this work by investigating the spatial relationship between pavement structural strength and roughness. Chapter 4 looks at the NZ-dTIMS roughness model and the NZ-dTIMS Henning rutting model. Chapter 5 investigates alternative (ie non-dTIMS) pavement deterioration models. Chapter 6 considers how vehicle-specific extreme loading can be modelled. In chapter 7, pavement deterioration on three SH sites that have been subject to extreme traffic loading is investigated. Chapter 8 considers maintenance activities and costs. Chapter 9 includes discussion, conclusions are made in chapter 10 and chapter 11 presents recommendations for further work.

2 Determination of limiting structural number

2.1 Introductory comments

With reference to Paterson (1987), a major difficulty in investigating and modelling pavement behaviour is the representation of pavement strength. The pavement is a semi-infinite continuum comprising layers of materials with often greatly differing properties and behaviour under load. Furthermore, light loads have a shallower depth of influence than heavy loads. Therefore, some uniform basis is required for representing pavement strength. The most widely used pavement strength parameter in predictive pavement condition modelling is the structural number. The structural number ranks pavement performance by permanent deformation under repeated loading to material characteristics that are related to shear strength.

In New Zealand, the structural number is typically derived from FWD surveys, using a relationship originally proposed by Salt (1999).The Salt method of calculating the structural number relies entirely upon the deflections measured from the FWD survey. Displacements at eight locations around the impact area are available. However, only three are incorporated in the New Zealand equation. These three measurements are taken at 0mm, 900mm and 1500mm from the centre of the impact. The New Zealand equation has the following form:

$$SNC = 112(D_0)^{-0.5} + 47(D_0 - D_{900})^{-0.5} - 56(D_0 - D_{1500})^{-0.5} - 0.4 \quad \text{(Equation 2.1)}$$

where: SNC = the modified structural number

D_0 = displacement measured directly under a standardised 40kN FWD impact, μm

D_{900} = displacement at 900mm from loading plate, μm

D_{1500} = displacement at 1500mm from loading plate, μm

Using the modified structural number (SNC), both the strength of the pavement and its underlying subgrade are accounted for.

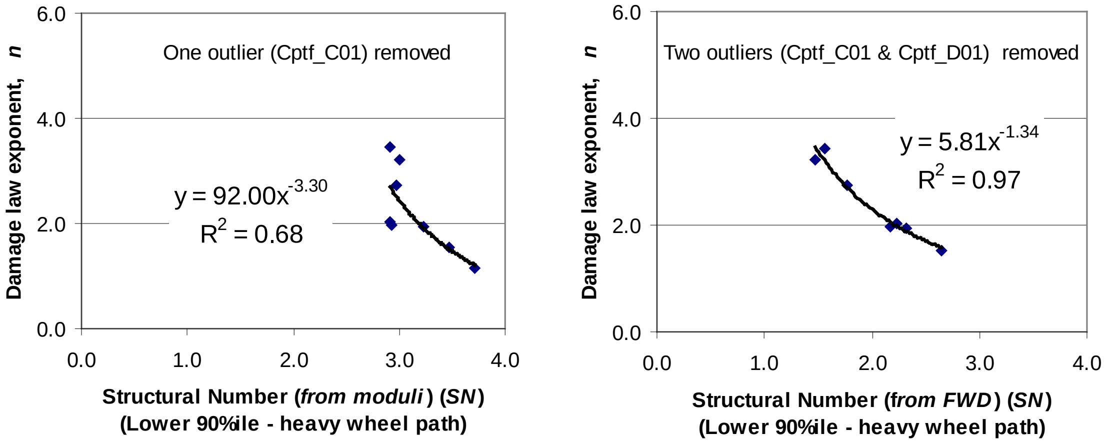
An accelerated loading test was undertaken in 2002 at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) to compare the wear generated by different levels of loading (Arnold et al 2005). The pavement consisted of five different segments that were subjected to 1,000,000 load cycles in two parallel wheelpaths. The axle load on one wheelpath was 8.2 tonnes while the load on the other was 12 tonnes. Key findings from this study can be summarised as follows:

- The relationships between SNC and pavement life are best when using the lower 10th percentile value of SNC for the road section of interest.
- There is a relationship between SNC and the damage law exponent, n, the lower the value of SNC, the higher the damage exponent as shown in figure 2.1.

Therefore, accurate determination of SNC is a key input in estimating the likely effect on pavement condition of any changes in heavy commercial vehicle (HCV) traffic characteristics.

Because FWD surveys are conducted only on selected sections of state highway and very rarely on local authority roads, this chapter investigates the possible use of a limiting value of SNC when FWD-derived SNC data is not available.

Figure 2.1 Relationship between structural number and damage law exponent (Arnold et al 2005)



2.2 Site selection

In order to obtain representative values of SNC for pavements likely to be exposed to extreme traffic loading, a selection of local authority roads were chosen that either had just recently been exposed to extreme loading or where extreme loading was predicted in the near future. This was done to minimise any differences between structural strength at the time of loading and the time of the FWD survey. Consultation occurred with regional roading authorities and representatives of industries, such as logging, mining and dairy farming, to identify suitable candidate roads. The following criteria were also used to aid the selection:

- The pavement section must be reasonably straight. This means the pavement surface will be free of any turning damage.
- The pavement section should be between 500m and 1000m in length. This ensures sufficient data will be collected without requiring excessive testing.
- Historical data for the pavement section should be available. This should include data on previous distress such as cracking or rutting, previous maintenance, roughness and traffic loading distribution.
- A mixture of pavement types should be used: those that were designed to carry heavy vehicles and those that evolved to carry heavy vehicles.

Six roads were identified that met these criteria:

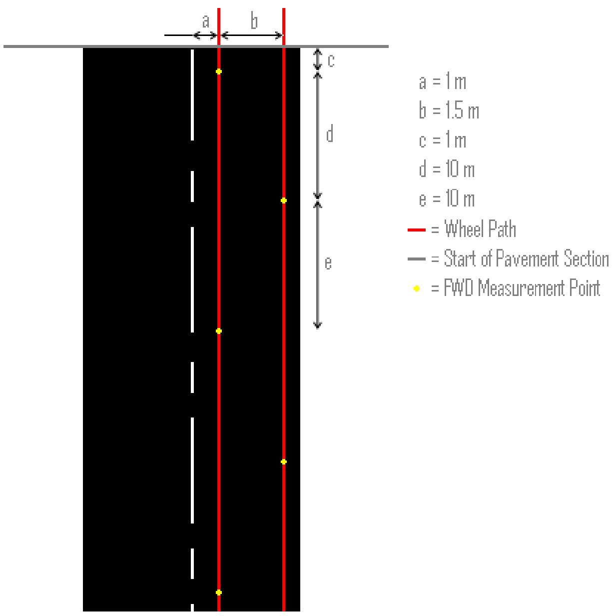
1. Harbottle Road (RAMM ID 159) in the Matamata-Piako district. Survey from 2000m to 0m. Harbottle Road was resealed in 1993, had annual average daily traffic (AADT) of approximately 200 and carried quarry traffic in years prior to 2005.
2. Barton Road (RAMM ID 28) in the Matamata-Piako district. Survey from 1700m to 0m.
3. Wairakau Road (RAMM ID 507) in the Matamata-Piako district. Survey from 2000m to 0m. Wairakau Road was subject to a significant increase in logging traffic in 2005.
4. Waikaka Road (RAMM ID 115) in the Hauraki district. Survey entire length from 800m to 0m. Waikaka Road was chosen as its mean roughness had gone from 1.79 International Roughness Index (IRI) (47 National Association of Australian State Road Authorities (NAASRA)) to 3.32 IRI (87 NAASRA) and had undergone heavy logging in late 2003.

5. Admiral Road (RAMM ID 117) in the Carterton district. Survey from 10,000m to 9000m.
6. Jervois Road (RAMM ID 79) in the Carterton district. Survey from 2000m to 0m.

2.3 Measurement of SNC distributions

Once the six sites had been identified the next step was to measure the pavement strength of each. The pavement strength data obtained for each site was calculated from data obtained using a FWD. The FWD survey obtains information on the strength of the pavement through measurement of the deflection bowl caused by dropping a 40kN standardised weight from a predefined height (HTC Infrastructure Management 2000). To achieve a higher resolution view of the pavement strength than is currently available from the RAMM database (normally, FWD readings are reported at a spacing of 100m), FWD tests were performed every 10m staggered between wheel paths. The inside and outside wheel paths were assumed to be offset from the centre line by 1m and 2.5m respectively as shown in figure 2.2. The FWD tests were performed in the lane carrying the majority of the extreme loading for each road.

Figure 2.2 Figure showing the layout of the measurement locations used for the FWD survey



2.4 Variance in SNC

Trends in distribution shape, minimum, 10th percentile and mean values were investigated so a greater understanding of the actual variance in SNC could be developed.

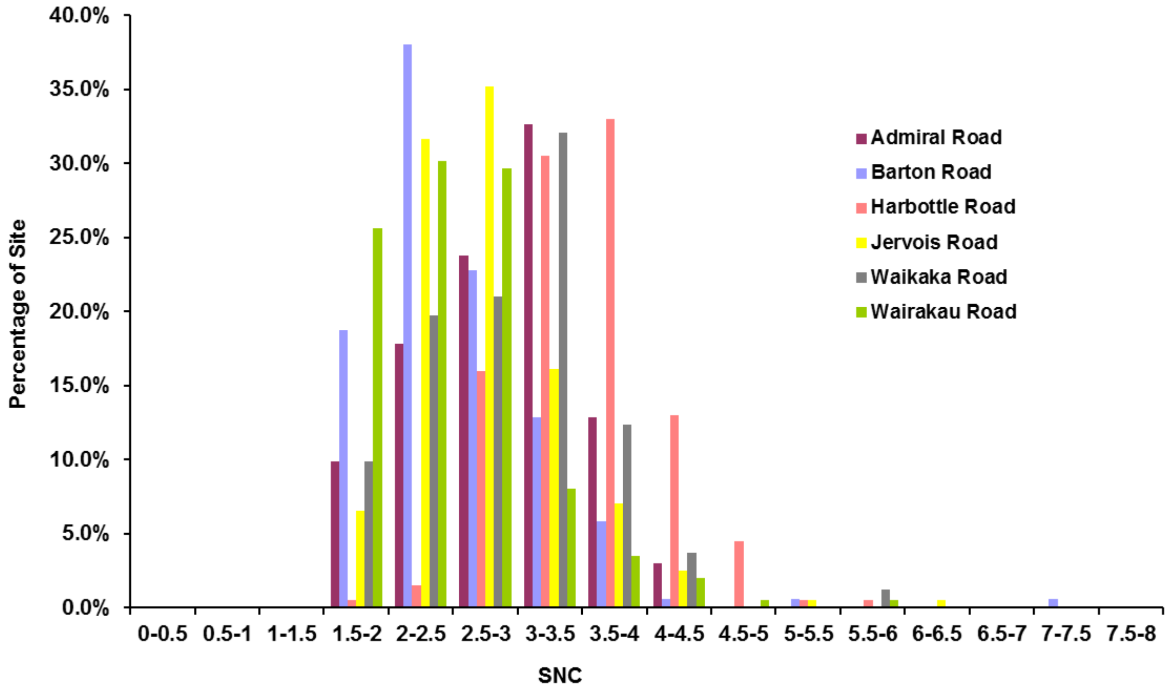
The distributions of SNC within each extreme loading site are plotted in figure 2.3 and the various statistical measures summarised in table 2.1.

From figure 2.3, it can be seen that although all the distributions have different mean values, they all possess very similar distribution shapes. It can also be seen that the SNCs are not normally distributed with an apparent lower limit causing a positively skewed distribution.

Table 2.1 SNC summary statistics for six local authority roads surveyed

Territorial local authority site	District	SNC		
		Mean	Minimum	10th percentile
Admiral Road	Carterton	2.92	2.01	2.01
Barton Road	Matamata-Piako	2.57	1.66	1.85
Harbottle Road	Matamata-Piako	3.55	1.95	2.90
Jervois Road	Carterton	2.77	1.61	2.10
Waikaka Road	Hauraki	2.48	1.62	1.80
Wairakau Road	Matamata-Piako	2.93	1.71	2.01
		Average:	1.76	2.11

Figure 2.3 SNC distributions for each of the six roads surveyed



Of particular interest to this project is the appearance of an apparent lower limit in SNC of approximately 1.8. In terms of pavement deflection, a SNC of less than 2 is associated with central deflections greater than 3mm. For New Zealand pavements on non-volcanic subgrades, deflections greater than 3mm are uncommon and thus a lower limit SNC value of 1.8 appears reasonable. As all of the pavement condition models investigated in this report are dependent on SNC, a reasonable assumption is that the point of maximum pavement deterioration will coincide with minimum pavement strength. Therefore, knowing that all pavements have a lower SNC limit of approximately 1.8 should allow for the time, location and traffic loading until pavement failure to be determined for the weakest section of road, and therefore the whole road section of interest, through use of pavement deterioration models.

2.5 Comparison structural number measures

Several other methods exist for evaluating structural strength. Two of the more widely used measures are the adjusted structural number (SNP) (HTC Infrastructure Management 2000) and the SNC as calculated in Paterson (1987). To investigate the correlation of these two methods with the FWD-derived SNC as calculated using equation 2.1, comparative plots were generated for two of the test sites – Admiral Road (figure 2.4) and Jervois Road (figure 2.5).

With reference to figures 2.4 and 2.5, the variation in structural number with displacement is very similar for all three methods, although the SNP appears to give lower values on weak (low structural number) pavements.

Figure 2.4 Comparison of three methods for calculating structural number – Admiral Road

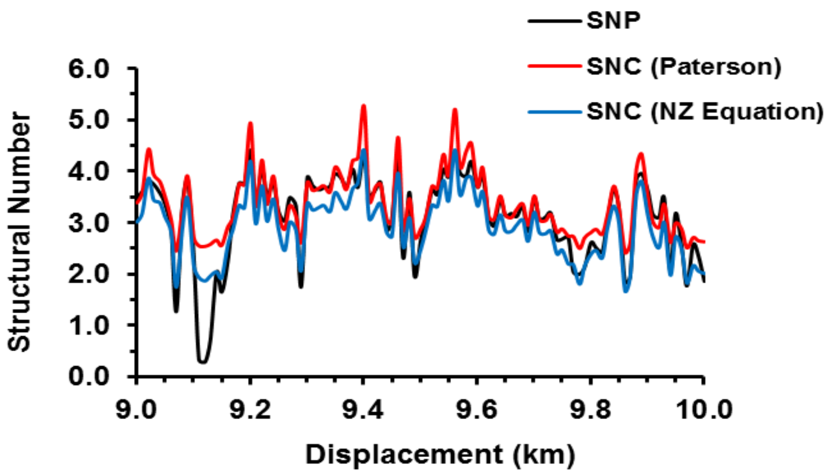
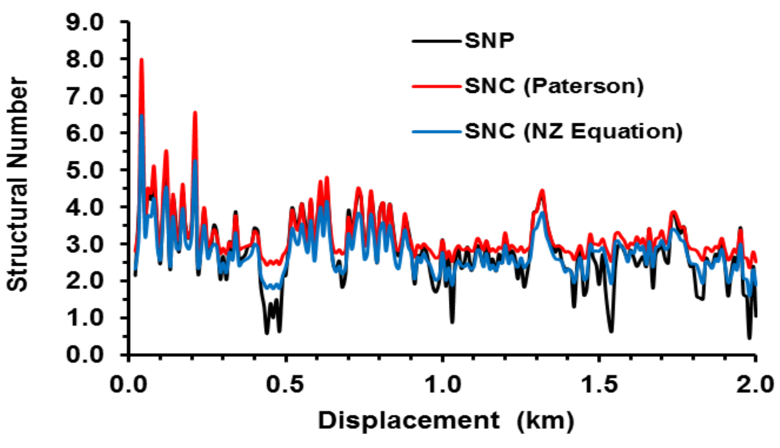


Figure 2.5 Comparison of three methods for calculating structural number – Jervois Road



2.6 SNC values for pavement deterioration modelling

In the absence of any FWD-derived SNC data, we recommend a limiting SNC value of 1.8 be used in modelling the potential effect of increased HCV traffic on pavement condition. However, for situations where recent FWD-derived SNC data is available for the road or route of interest, the 10th percentile SNC value should be used instead.

3 Pavement condition – SNC relationship

3.1 Background

Although SNC is known to vary within pavement lengths the majority of deterioration models simply take one SNC value to represent an entire treatment length (a typical treatment length is 450m).

It was hypothesised that pavement failure was likely to be instigated where SNC was at its lowest, which if shown to be true means the within pavement variance in SNC may be an important predictor for roughness, rutting and cracking in extreme loading conditions.

3.2 Investigation of Matamata-Piako district sites

To investigate the assumption that the location of maximum pavement distress coincides with the position of minimum SNC, an analysis involving the three test sites located in the Matamata-Piako district referred to in the previous chapter was undertaken. These three test sites were:

1 Barton Road (RAMM ID 28)

2 Harbottle Road (RAMM ID 159)

3 Wairakau Road (RAMM ID 507)

Being local authority roads, the only pavement condition parameter for which 100% sampling was available at the time of interest was lane roughness. Accordingly the analysis involved graphically comparing measured SNC values, averaged over 100m, with 100m roughness data taken from the RAMM database for the same period of time. Plots were produced comparing lane roughness in the left and right lanes with SNC. These plots are shown in figures 3.1 to 3.3. Also correlations between roughness and average and minimum SNC over 400m intervals were calculated. Table 3.1 summarises the results of the correlation calculations. An inverse relationship was expected if the assumption was correct, with high roughness at low SNC and vice versa.

Figure 3.1 Comparison of lane roughness and SNC for the left and right lanes of Barton Road

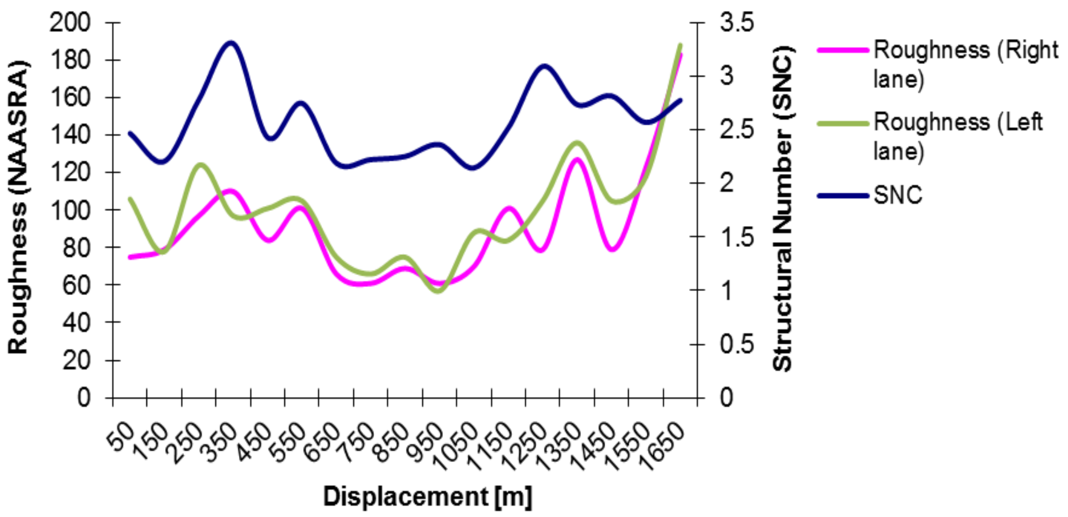


Figure 3.2 Comparison of lane roughness and SNC for the left and right lanes of Harbottle Road

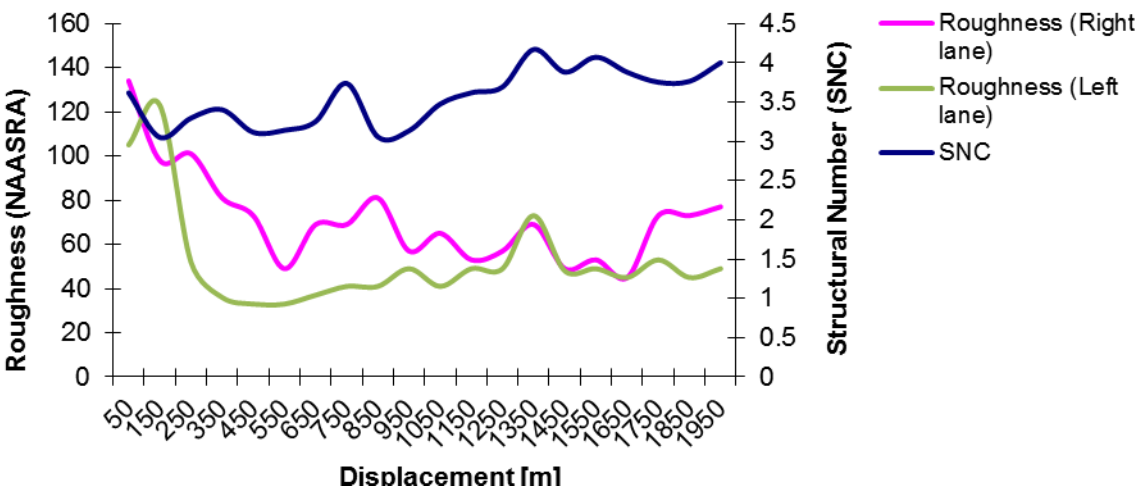


Figure 3.3 Comparison of lane roughness and SNC for left and right lanes of Wairakau Road

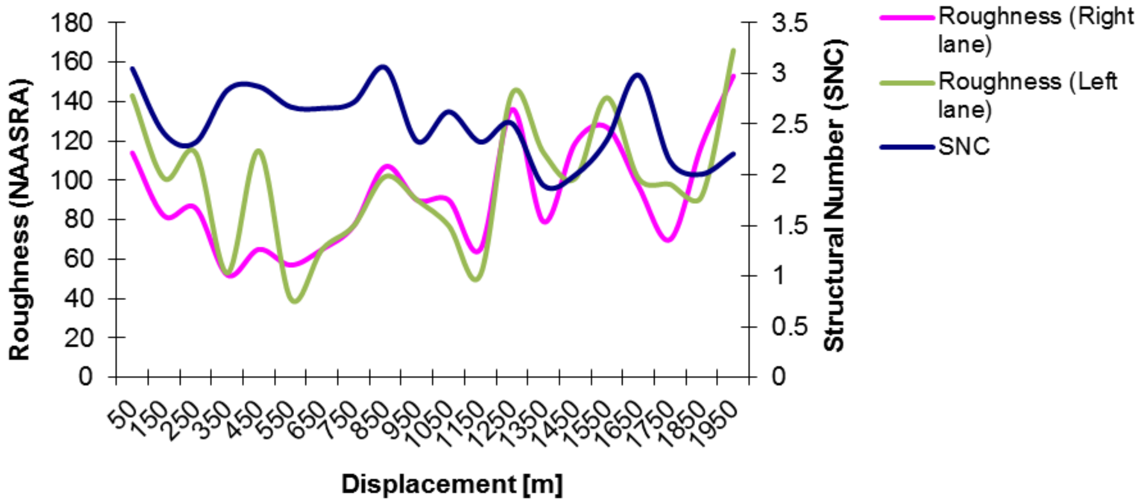


Table 3.1 Correlations between roughness and structural number over 400m averaging length

Test site	Lane roughness-SNC correlation (r)			
	Left lane		Right lane	
	Mean SNC	Min SNC	Mean SNC	Min SNC
Barton Road	0.98	0.87	0.98	0.90
Harbottle Road	-0.01	-0.12	-0.45	-0.48
Wairakau Road	-0.87	-0.92	-0.96	-0.99

3.3 Discussion of results

With reference to figures 3.1 to 3.3, no clear relationship between 100m averaged lane roughness and SNC is evident. When the averaging length was increased to 400m, two of the roads examined showed a negative correlation as expected, these being Harbottle Road and Wairakau Road (see table 3.1). For these two sites, the correlation coefficient was slightly higher if the minimum 100m SNC value over the 400m averaging length was used rather than the mean SNC value, with the right lane of Wairakau Road producing the highest correlation coefficient of -0.99.

Surprisingly, the third site, Barton Road, showed a very positive correlation between lane roughness and SNC, ie low roughness corresponded to low SNC. The reason for this result is not readily apparent.

Unfortunately, due to the small number of test sites it was not possible to reach a definitive conclusion regarding the hypothesis that maximum pavement distress occurred in the vicinity where SNC was lowest. However, the key points to arise from this analysis are as follows:

1. Both pavement condition and SNC appear to be highly variable along a road section. Therefore when applying SNC-based road deterioration models, it should be remembered that the reliability of the resulting predictions of road condition improve as the analysis length of interest increases. For this reason, modelling results should only apply to road lengths of 400m or longer.
2. There is evidence to suggest that minimum rather than average SNC is a better predictor of pavement condition, supporting the findings of Arnold et al (2005) that relationships between SNC and pavement life are best when using the lower 10th percentile value of SNC for the road section of interest.
3. There may be some merit in repeating the exercise on a larger number of test sites, using GPS coordinates to better link the location of the SNC measurements to pavement condition data extracted from the RAMM database so that any spatial relationships between areas of pavement distress due to traffic loading and pavement areas with lower than surrounding SNC can be established.

4 NZ-dTIMS models

4.1 General

According to RIMS (2001) 'NZ-dTIMS is a decision-support system, a tool to help road controlling managers to plan, in an objective and effective way, pavement maintenance management and activities over the short, medium and long term'.

The aim of the research described in this section was to examine how effectively the current models contained within NZ-dTIMS could be used to model pavement condition under extreme traffic loading. This research was originally to be performed by comparing the results of each NZ-dTIMS model with actual data taken from the extreme loading sites. However, due to a variety of reasons, the relevant information required for model validation was very difficult to obtain.

The first difficulty was that the majority of roads suffering from extreme loading are not state highways. This meant that data was not widely available for the roads of interest, and where it was available it was often insufficiently detailed for use. The second difficulty was due to the fact that distress caused by the extreme loading led to increased maintenance work being performed on the road. This in turn meant that the sections of most interest to the research, ie areas where roughness or rutting is high, are often not left for a sufficient length of time to be recorded. One un-adopted but contemplated method of avoiding this problem would be to use long-term pavement performance (LTPP) sites as they are left un-maintained specifically for the purpose of observing pavement deterioration for model calibration. However, this was ruled out as the LTPP sections are of insufficient length and none could be identified as suffering from extreme loading.

Three state highways were identified as suffering from extreme loading: SH32 due to increased logging truck traffic and SH7 and SH63 which were subjected to large traffic volumes for approximately six days when SH1 near Kaikoura was closed due to slips. Although these cases are useful for looking at how the models behave, due to the lack of data on the mechanisms of distress on SH32 and the very short nature of the loading on SH7 and SH63, the models' sensitivity to traffic loading variables was analysed. (The comparison of the models' predictions against actual pavement deterioration data is detailed in chapter 7 of this report.)

The sensitivity analyses were performed on the NZ-dTIMS roughness progression and rutting progression models with respect to the number of ESA, growth in AADT and the percentage of heavy commercial vehicles (%HCVs).

It should be noted that sensitivity analyses presented in this chapter (chapter 4) and in chapter 5 relates to extreme loading brought about by increases in HCV traffic volume only (ie the make-up of the HCV traffic remains constant). Vehicle-specific extreme loading is considered in chapter 6.

4.2 NZ-dTIMS roughness model

The roughness model currently incorporated in the NZ-dTIMS system is a function of pavement age, structural strength, annual traffic loading, rut depth, cracking and potholing. The model has the following form (Henning and Hatcher 2008):

$$IRI_i = \left(IRI_{i-1} \times (1 + 0.023 \times KGE)\right) + KGP \times \left(134e^{(KGE \times 0.023 \times AGE) \times \frac{YE4}{(1+SNP)^5}}\right)$$
$$+ 0.114\Delta RUTS + 0.0066\Delta CrackIndex + 0.42(\Delta PotholeCracking + \Delta PotholeEnlargement) \quad \text{(Equation 4.1)}$$

where: *IRI* = pavement roughness in terms of the International Roughness Index (m/km)

KGE = calibration constant

KGP = calibration constant

AGE = pavement age (years)

YE4 = annual ESALs per lane/10^6

SNP = adjusted structural number

ΔRUTS = annual change in rut standard deviation

ΔCrackIndex = annual change in cracking index

ΔPotholeCracking = annual change in potholes due to cracking

ΔPotholeEnlargement = annual change in potholes due to enlargement

For the sensitivity analyses, defaults were set as below:

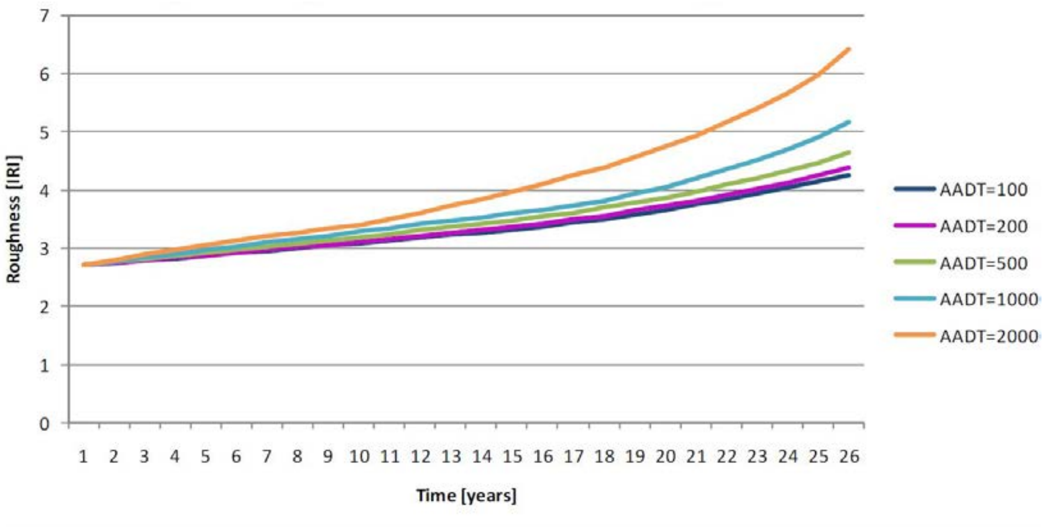
- the SNC was kept constant at 1.8
- the traffic growth was maintained at 0%
- HCV kept at 10% (5% HCVI and 5% HCVII)
- the AADT was kept at 2000
- the initial roughness was 2.7 IRI (70 NAASRA counts/km).

Naturally, the above defaults were used only if the given variable was not the subject of the sensitivity analysis.

Graphical results for three of the sensitivity studies are shown in figures 4.1 to 4.3 below.

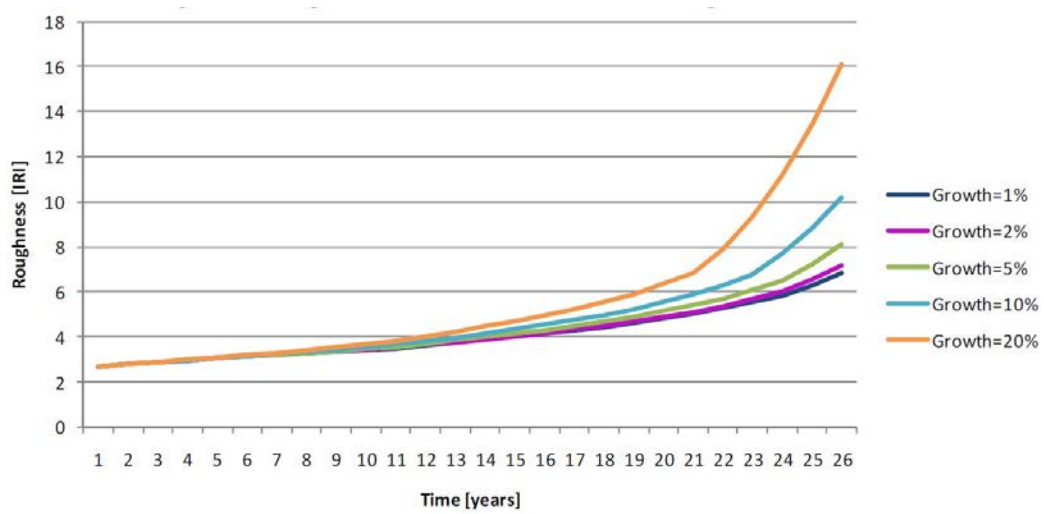
In the first of these (figure 4.1), the sensitivity of the model's predictions of IRI was evaluated at several different values of AADT. This figure shows that the model has little sensitivity to traffic, with a 20-fold increase in AADT causing little accelerated deterioration over the first decade or so. The roughness progression is seen to accelerate after around 12 years at higher loadings. This is caused after cracking has been initiated.

Figure 4.1 Predicted roughness for a range of traffic volumes as modelled using the current NZ-dTIMS roughness progression model



The next step was to examine the models sensitivity to the rate of traffic growth. The results of this analysis are of particular interest as a simplified representation of extreme loading can be achieved by using high growth rates. The results of this sensitivity analysis are shown below in figure 4.2.

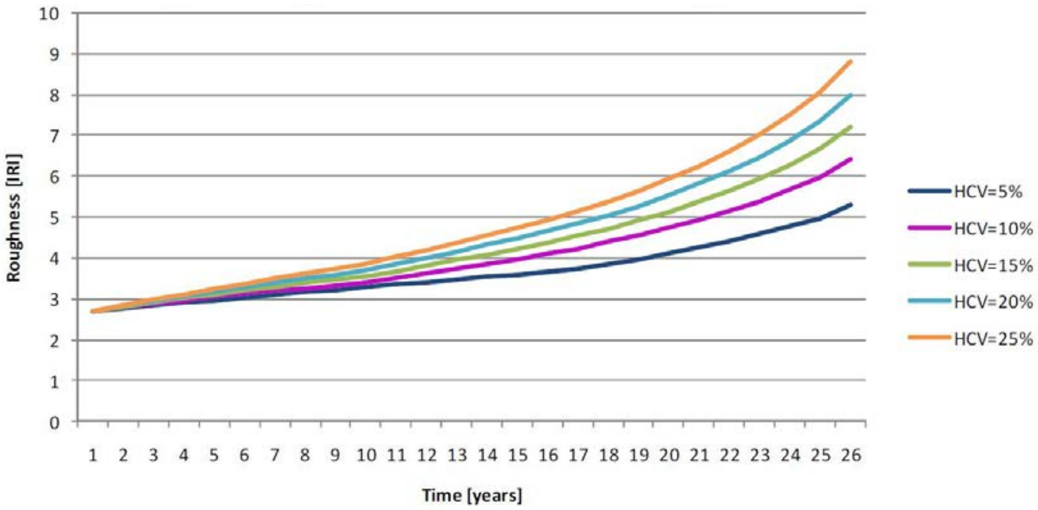
Figure 4.2 Predicted roughness for a range of traffic growth rates as modelled using the current NZ-dTIMS roughness progression model



The above figure shows that the model has very low sensitivity to traffic growth rates, with a 1% and 20% growth rate leading to almost identical pavement roughness until well after 15 years.

The final traffic loading variable used to test the model's sensitivity was %HCV. For this analysis the traffic loading was maintained constant at 2000 AADT, while the %HCV was varied. Although increases in %HCV are linked directly with increases in traffic volume, they have been kept separate in this analysis so the effects of each can be observed. The results of the analysis are shown in figure 4.3.

Figure 4.3 Predicted roughness for a range of %HCV modelled using the current NZ-dTIMS roughness progression model



The results of the above analysis show that the percentage of HCVs is the most important of the traffic loading variables investigated, with increases in %HCVs leading to dramatically accelerated roughness progression. However, again a significant effect does not occur for approximately 10 years.

4.3 NZ-dTIMS Henning rutting model

4.3.1 Model description

The Henning rutting model is the current rutting model included in the NZ-dTIMS system.

Rutting is defined as the permanent or unrecoverable traffic-associated deformation within pavement layers which, if channelised into wheel paths, accumulates over time and becomes manifested as a rut (Paterson 1987). Rutting is a commonly used measure of pavement performance and is included in most international pavement management systems as a trigger for maintenance work (Henning and Hatcher 2008).

Rutting can be split into three stages. The first stage is called ‘initial densification’ and is caused by bedding in or post-construction compaction of the subgrade. This form of rutting generally only occurs during the first year after sealing. The next stage of rutting is known as ‘constant rate deformation’ and is characterised by a constant rate of increase in deformation over time or traffic loading (ND Lea International Ltd 1995). The final stage, known as ‘accelerated deformation’, is where deformation rates increase dramatically near the end of the pavement’s life.

Several models, including HDM-III and HDM-4, exist for predicting rut progression with the majority being based on traffic loading, pavement strength and initial compaction of the pavement. The HDM-4 rut progression model has been used in New Zealand. However results found poor correlation with actual rutting data (Henning and Hatcher 2008). This, along with the fact that rutting is an important pavement performance measure led to a new rutting model being developed by Henning specifically for New Zealand conditions (Henning and Hatcher 2008). This new rutting model is described below.

The two stages of rutting that are of particular interest to this research are the initial rut densification and the following constant rate deformation, as these represent the majority of rut progression.

The initial rut densification model was created by regression of data obtained from the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). CAPTIF consists of a large circular track of pavement where traffic loading can be simulated and pavement performance measured in a controlled environment. The resulting rut densification model generated from the data has the following form:

$$RUTM = 3.5 + e^{(2.44-0.55xSNP)} \quad \text{(Equation 4.2)}$$

The subsequent constant rate deformation stage of rutting was also modelled using regression of data obtained from CAPTIF. This regression found that thick (>150mm) and thin (<150mm) pavements have different deformation rates. The rut progression model for the constant rate deformation stage of rutting has the following form:

- For thin pavements (H<150mm and SNP<2.5)

$$\Delta RUTM = MAX\{1.2\ ,\ 9.94\text{-}1.38xa_1xSNPxYE4\} \quad \text{(Equation 4.3)}$$

- For thick pavements (H>150mm)

$$\Delta RUTM = MAX\{0.8\ ,\ 14.2\text{-}3.86xa_2xSNPxYE4\} \quad \text{(Equation 4.4)}$$

4.3.2 Sensitivity study

Figures for rut depth sensitivity studies for two consecutive stages of rutting are shown in figures 4.4 to 4.6. These stages of rutting were the ‘initial rut densification’ (year 1) followed by ‘constant rate deformation’ (years 2–25). The variable settings were:

- the SNC was kept constant at 1.8
- the traffic growth was maintained at 0%
- HCV was kept at 10% (5% HCVI and 5% HCVII)
- AADT was 500
- H was 200mm.

Naturally, the above settings were used only if the given variable was not the subject of the sensitivity analysis.

It is apparent from these three figures (ie figures 4.4 to 4.6) that, when using the NZ-dTIMS (Henning) rutting models for the first two stages of rutting, the relationship between rut depth and time is approximately linear for ages from 10 years to 25 years (ie for this time period, no rapid exponential increase in rut depth with time is evident).

Figure 4.4 Predicted rut depth modelled using the NZ-dTIMS (Henning) rut depth progression model for a range of traffic volumes

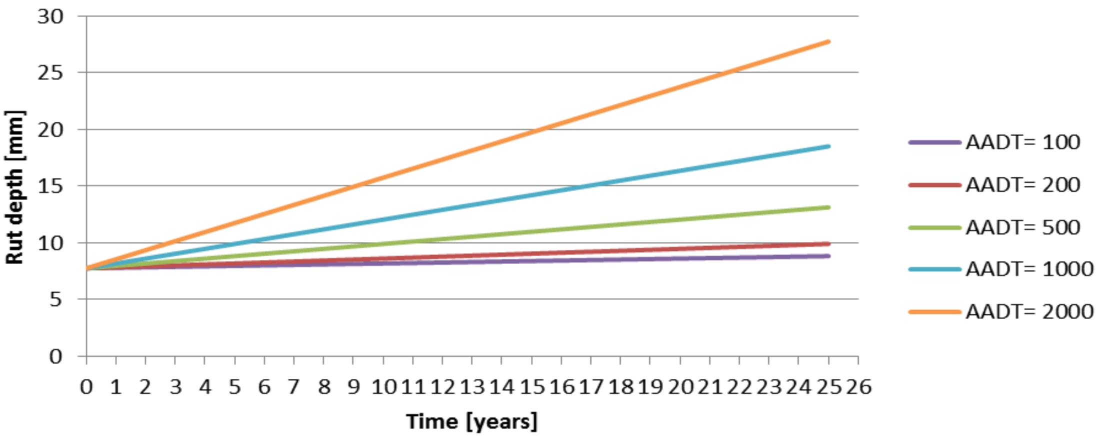


Figure 4.5 Predicted rut depth modelled using the NZ-dTIMS (Henning) rut depth progression model for a range of traffic volume growth rates

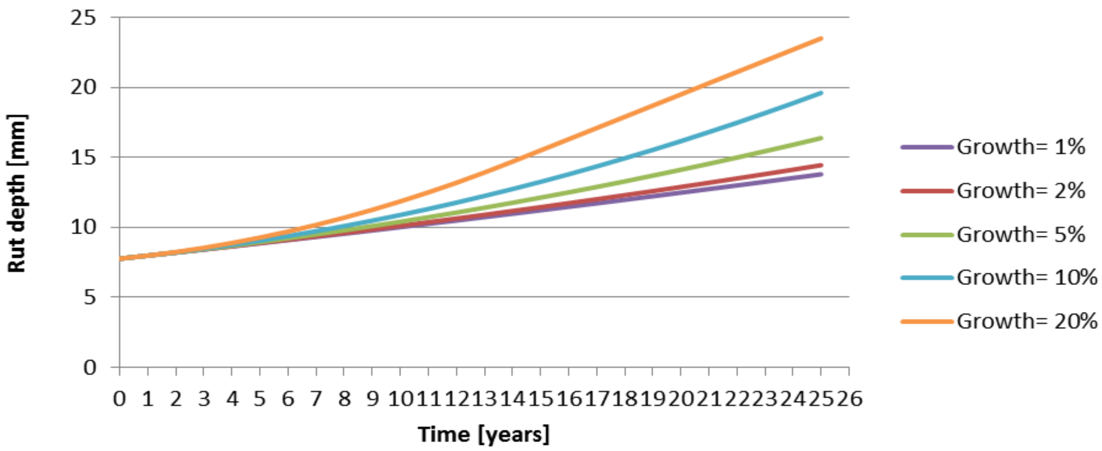
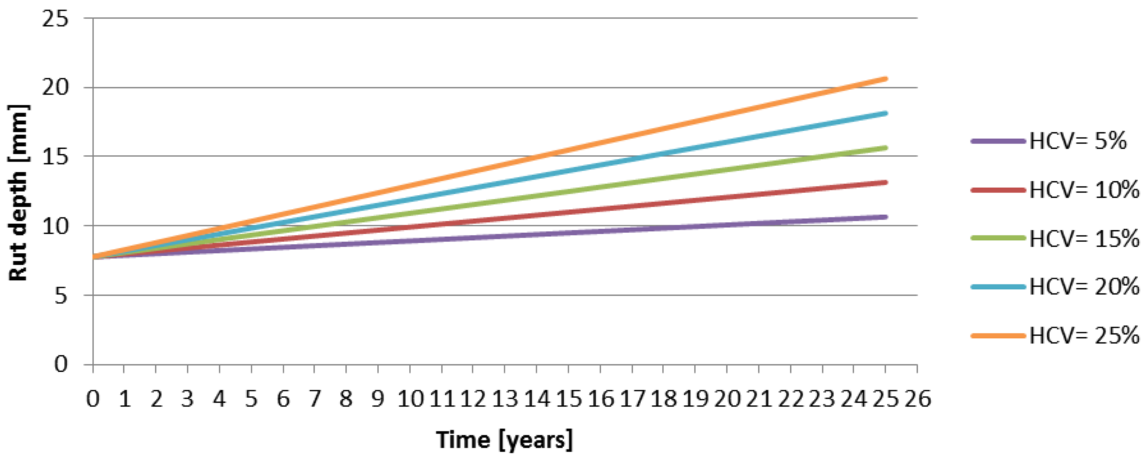


Figure 4.6 Predicted rut depth modelled using the NZ-dTIMS (Henning) rut depth progression model for a range of %HCV



5 Alternative models

5.1 HDM-4 roughness progression model

The HDM-4 roughness progression models were developed as improvements upon the World Bank's HDM-III models presented by Paterson (1987). The HDM-4 models have been widely used to predict pavement condition, with some of the models from HDM-III and HDM-4 being incorporated in NZ-dTIMS. Of particular note is that the HDM-4 roughness progression model is based on the HDM-III component incremental model, which has been validated in a large number of countries (ND Lea International Ltd 1995).

The HDM-4 roughness model accounts for the contributions to roughness from several different components including: cracking, rutting, potholing and age and environmental effects. As the calibration and implementation of this model is complex, this research took the simpler approach of analysing each component's sensitivity separately. As the HDM-4 model remained essentially unchanged from the HDM-III component incremental model, the components of the model as presented by Paterson (Paterson 1987) were investigated. Paterson's thorough analysis of the model found that the components individually contributing most to the model's accuracy were: the structural component, the age-environmental component, the cracking component and the potholing component. To better understand the sensitivity of the overall component incremental model, the sensitivity of each component (ie structural, age-environmental, cracking or potholing) to both the traffic volume and the %HCV was investigated.

5.1.1 Structural component

The structural component of the model takes into account the structural strength contribution of the pavement layers to resisting the roughness progression. The structural component has the following form:

$$\Delta RI_S = e^{mt} SNCK^{-5} \Delta NE_4 \quad \text{(Equation 5.1)}$$

where: ΔRI_S = structural contribution to roughness, in IRI

m = environmental coefficient

t = time since last seal, in years

ΔNE_4 = incremental number of equivalent axle loads in period Δt, million ESA/lane

$SNCK$ = combination of structural number and pavement thickness adjusted to account for cracking in the pavement at the time of the FWD survey.

The time step used for this investigation was one year and ΔNE_4 is the total number of ESA in one year where ESA is taken to accumulate at a rate of 0.7 per heavy vehicle and can be calculated from the following:

$$\textit{Million ESA/lane/year} = 0.7*AADT*[365*(\%HCV/100)]/10^6 \quad \text{(Equation 5.2)}$$

According to the *New Zealand supplement to the Austroads pavement design guide* (Transit NZ 2007), the presumptive number of axle groups per heavy vehicle (N_{HVAG}) for New Zealand state highways is 2.4 and the damage index is 0.6 ESA/HVAG, which corresponds to 1.4 ESA per HCV. However, if it is assumed that there is no directionality (ie equal number of loaded HCVs travelling in the increasing direction as in the decreasing direction) and that a full load is carried for only 50% of the time (ie fully loaded on the outbound journey and partially loaded on the inbound journey), one HCV will correspond to 0.7ESA, as adopted in equation 5.2. This assumption was shown to give good agreement between predicted and observed roughness progression rates on rural New Zealand state highways (Cenek and Patrick 1991).

As the structural component of the HDM-4 roughness model is directly proportional to the traffic loading variable NE_4, which in turn is directly proportional to both AADT and %HCV, there is no need to perform a sensitivity analysis. Any change in AADT or %HCV will be directly matched by an equivalent change in the structural component of the model.

5.1.2 Age-environmental component

Two main environmental factors influence the roughness progression in HDM-4. These are fluctuations in temperature and moisture.

The main effects of age on pavement roughness are through foundation subsidence which can cause volume changes or distortion (Paterson 1987).

In the HDM-4 roughness model, the age-environmental component is not a function of traffic loading and is simply dependent upon a coefficient relating to the temperature and humidity conditions.

5.1.3 Cracking component

Pavement cracking in HDM-4 can be split into four separate classes according to severity. These four classes are as follows (Paterson 1987):

- Class 1: hair line cracks, width 1mm or less
- Class 2: crack widths 1mm to 3mm
- Class 3: crack widths greater than 3mm without spalling (spalling = breaking or chipping of the surfacing adjacent to the crack edges)
- Class 4: spalled cracks, ie fragments of the surfacing adjacent to the crack are lost.

The cracking component of the HDM-4 roughness progression model is proportional to the change in area of indexed cracking over each year. The indexed area of cracking can be defined as the average of the four classes of cracking as a percentage of the pavement area. One method of calculating the area of all cracking as a percentage of the pavement area is shown below (Queiroz 1981):

$$CR_2 = a + b\, S \log NE' + c\, S\, AGE \log NE' \qquad \text{(Equation 5.3)}$$

where:

CR_2 = area of all cracking, in percentage of pavement area

NE' = number of equivalent axle passes since crack initiation

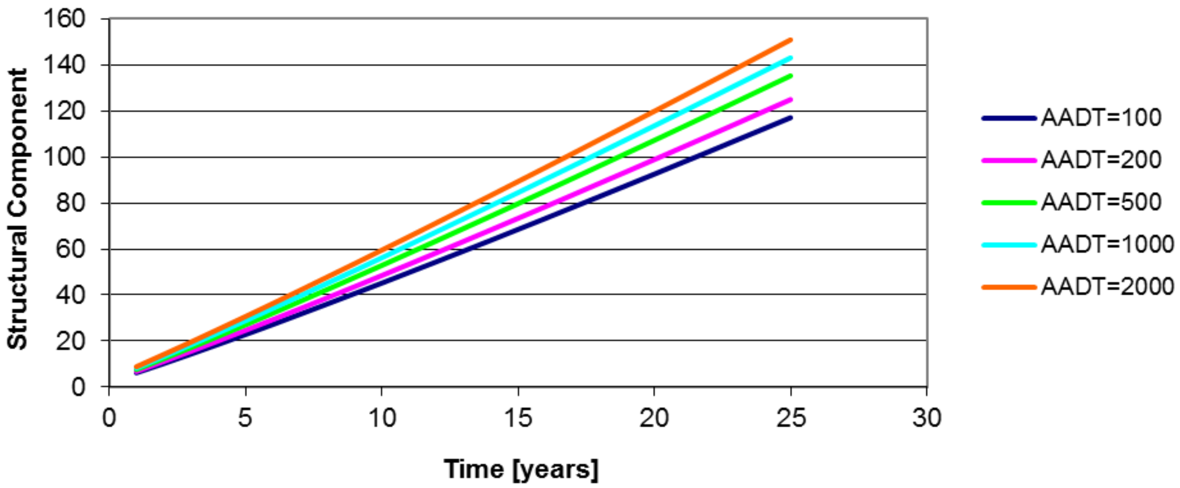
S = pavement strength index

a, b, c = estimated coefficients

AGE = age of pavement since surfacing, years.

To establish the cracking component's sensitivity to traffic loading, the above expression was evaluated over a range of traffic volumes. As NE_4 is directly proportional to both AADT and %HCV it was not necessary to examine the sensitivity of the cracking component to both. When examining the sensitivity to traffic volume the %HCV was maintained constant at 10%. The results of the sensitivity analysis are shown below in figure 5.1.

Figure 5.1 Cracking component of the HDM-4 roughness progression model for a range of traffic



The sensitivity of the cracking component to growth in traffic volume was also investigated. The results of this investigation along with the results from figure 5.1 are summarised below in table 5.1.

The values tabulated in table 5.1 are the percentage change over the 25-year analysis period in the cracking component caused by the specified changes to the model input variables. For example a 5% change in AADT produces only a 0.4% change in the cracking component of the HDM4 roughness model whereas a 50% change produces a 3.4% change. This table format is adopted for all the sensitivity analyses that follow.

With reference to table 5.1, of the three traffic characteristics investigated, the cracking component of the HDM-4 roughness progression model is most sensitive to traffic growth.

Table 5.1 Results of sensitivity analyses performed on the cracking component of the HDM-4 roughness model

Variable	Percentage change in cracking component		
	5% change in variable	20% change in variable	50% change in variable
AADT	0.4%	1.6%	3.4%
Traffic growth	3.7%	14.0%	31.0%
%HCV	0.4%	1.6%	3.4%

5.1.4 Potholing component

The potholing component of the HDM-4 roughness model was found to give the largest individual contribution to the total roughness. However as potholes are normally repaired very shortly after appearing, and as open potholes were avoided in the roughness measurements used for model development, direct statistical estimation of the effect of potholes was not possible (Paterson 1987). However, when dummy intercepts obtained from actual roads displaying potholing were compared with the incremental change in pothole volume, they were found to be proportional. Therefore, the potholing component could be obtained via the incremental change in pothole volume.

Potholing is considered to be the ultimate form of pavement distress, occurring once significant amounts of ravelling and wide cracking have first appeared. It is probable that by the stage potholing is present, the pavement will already be considered failed through cracking and ravelling. As the development of potholing is subsequent to significant distress most likely causing pavement failure, it was considered secondary for this research.

5.1.5 Concluding remark

The NZ-dTIMS roughness progression model was found to be reasonably insensitive to changes in traffic loading variables and as the HDM-4 roughness model failed to accurately simulate maximum roughness progression on the SH32 example considered in chapter 7 of this report, it was decided to investigate some additional pavement deterioration models.

5.2 Cenek-Patrick roughness model

The first alternative model investigated was the Cenek-Patrick roughness model (Cenek and Patrick 1991). This model is based upon the HDM-III 'aggregate trend' levels roughness progression model and has been modified to model roughness in NAASRA rather than IRI. However, to assist comparison with other model results reported here, the NAASRA roughness predictions of the Cenek-Patrick model have been converted to IRI using the conversion 1 NAASRA = 26.2 IRI when plotting figures 5.2 and 5.3.

The Cenek-Patrick roughness progression model has the following form:

$$RN(t) = [RN_0 + 5.7(1 + SNC)^{-4.99} EDAt]e^{0.0153t} \quad \text{(Equation 5.4)}$$

where: $RN(t)$ = roughness at time t

RN_0 = roughness at time t=0

SNC = the modified structural number

t = time, years

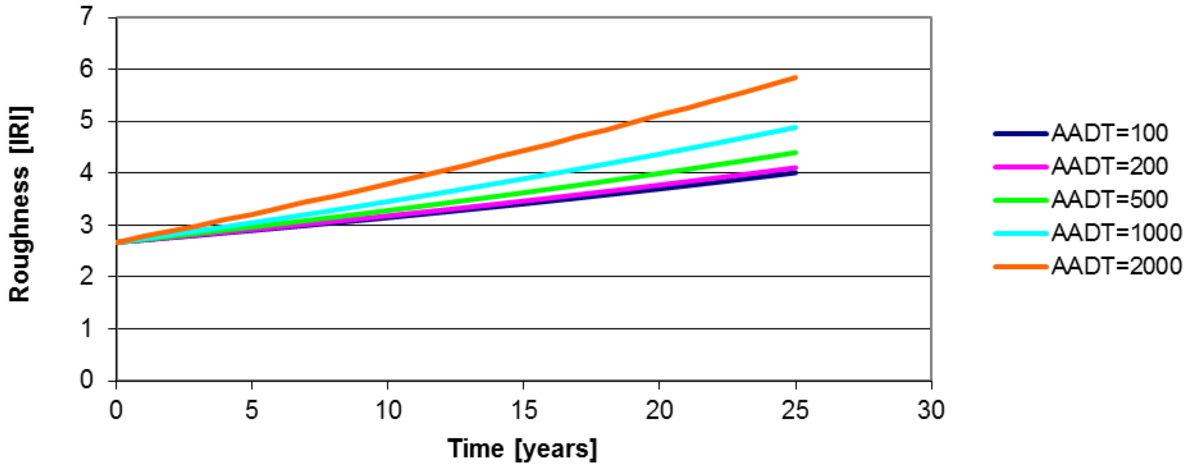
EDA = the equivalent design axle.

The equivalent design axle (EDA) is based on an 80kN axle loading, ie the same as the ESA used in previous sections. EDA is assessed to accumulate at a rate of 0.7 per heavy vehicle and can be calculated from the following relationship (Cenek and Patrick 1991):

$$EDA/lane/day = 0.7\left[\frac{AADT}{\#lanes}\right]\times\left[\frac{\%HCV}{100}\right] \quad \text{(Equation 5.5)}$$

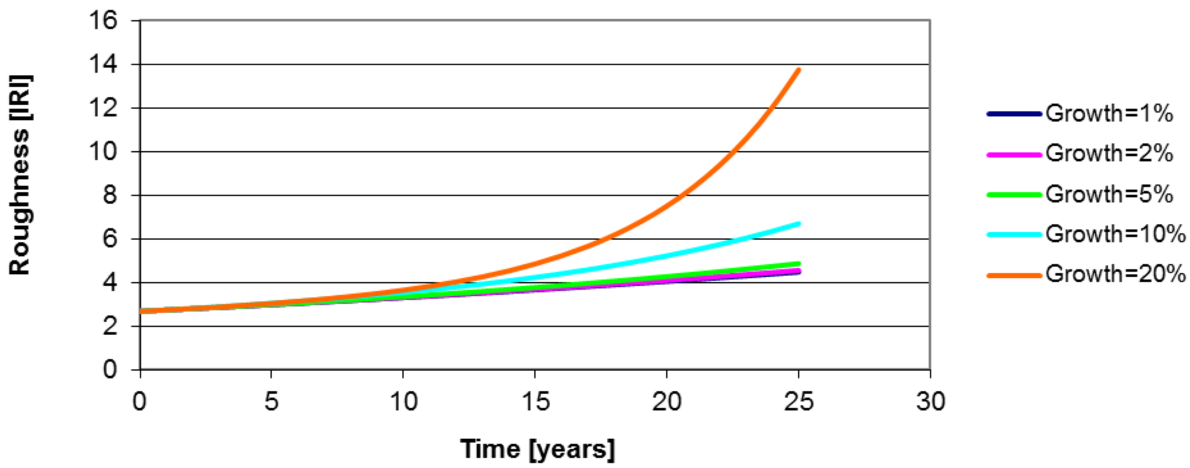
The first step performed to assess this model's suitability in modelling extreme loading was to examine the model's sensitivity to traffic loading variables. As all the traffic loading information enters the model via EDA, due to the form of the EDA equation equivalent changes in AADT and %HCV will give the same percentage change in the model. For this reason, only the model's sensitivity to AADT and traffic volume growth rate was examined. The first sensitivity analysis was performed using AADT. Results are shown in figure 5.2.

Figure 5.2 Roughness progression modelled over a range of traffic volumes (AADT) using the Cenek-Patrick roughness model



The next step was to examine the sensitivity of the model to various traffic volume growth rates. The results of this investigation are shown below in figure 5.3.

Figure 5.3 Roughness progression modelled over a range of traffic volume growth rates using the Cenek-Patrick roughness model



The results of subsequent tests into the sensitivity of the Cenek-Patrick roughness model with respect to initial roughness and SNC are summarised in table 5.2 along with the results of figures 5.2 and 5.3.

Table 5.2 demonstrates the Cenek-Patrick roughness model is most sensitive to SNC followed by traffic growth.

Table 5.2 Results of sensitivity analyses performed on the Cenek–Patrick roughness progression model

Variable	Percentage change in predicted roughness		
	5% change in variable	20% change in variable	50% change in variable
EDA	1.4%	5.4%	13.5%
SNC	4.8%	26.8%	160.5%
Traffic growth	2.2%	14.8%	111.0%
RN_0	3.6%	14.6%	36.4%

5.3 Ball alligator cracking model

Alligator cracking and surface flushing are generally accepted to be the principal causes of failure in surface dressed roads, the most popular surface for New Zealand's state highways (Ball, date unknown). Therefore, this section looks at predicting the life of a pavement section relative to alligator cracking rather than roughness as in sections 5.1 and 5.2.

The degree of alligator cracking found in a road cannot be predicted accurately from a knowledge of traffic loading or structural strength alone due to a degree of randomness in its nature (Ball, date unknown). However, it is possible to find the probability of a seal being cracked by statistical analysis of field data.

Data on the level of cracking, traffic loading and maximum pavement deflection was taken from the RAMM database. A logarithmic regression was then performed to obtain the following relationship relating the traffic loading and maximum pavement deflection to the probability of a seal containing 'C' percent alligator cracking.

$$P(C) = \frac{e^{a_0^C + a_T^C T + a_F^C F}}{1 + e^{a_0^C + a_T^C T + a_F^C F}} \qquad \text{(Equation 5.6)}$$

where: $P(C)$ = the probability that a seal will crack to C%

a_0^C, a_T^C and a_F^C = constants, dependent upon C

T = $10\log_{10}$[total heavy vehicles per lane since last sealing]

F = the FWD deflection, mm.

A pavement in New Zealand is considered to have critical levels of alligator cracking when 1% of the pavement area is cracked, and is considered to require immediate corrective action once 3% cracking occurs. For the purposes of this research, and for developing a conservative model:

- the traffic loading required to cause 1% of total pavement area cracking will be found
- a road will be considered to require maintenance once there is a 50% chance the pavement has cracked to a level of 1%.

In chapter 2 of this report it was found that pavement sections in New Zealand have a minimum cut off SNC value of approximately 1.8. However, the Ball alligator cracking model uses a single deflection as a measure of pavement strength rather than a combination of three deflections as with SNC. Therefore, it is necessary to examine the distribution of maximum displacements before a maximum limit can be found that represents the weakest section of pavement.

An analysis of the data obtained from the FWD surveys on the six roads, as detailed in chapter 2, found the average central deflection that corresponded with the bottom 5th percentile of structural numbers. The average maximum deflection for each of the roads in the weakest 5th percentile was 2.3mm.

To understand how the model copes with extreme loading, the sensitivity was investigated with respect to traffic volume and traffic volume growth rate. The results of these sensitivity analyses are shown in figures 5.4 and 5.5.

Further sensitivity analysis was performed on the model with respect to pavement deflection as determined by the FWD. The results of this analysis along with those from figures 5.4 and 5.5 are summarised in table 5.3.

As shown in table 5.3, the Ball alligator cracking model is most sensitive to traffic growth followed by pavement deflection.

Table 5.3 Results of the sensitivity analysis performed on the Ball alligator cracking model

Variable	Percentage change in predicted cracking		
	5% change in variable	20% change in variable	50% change in variable
AADT	1.7%	6.7%	15.3%
Traffic growth	8.5%	37.7%	103.2%
Deflection, F	7.2%	30.5%	82%

Figure 5.4 Probability of 1% alligator cracking against time. Cracking probability modelled over a range of traffic volumes using the Ball alligator cracking model

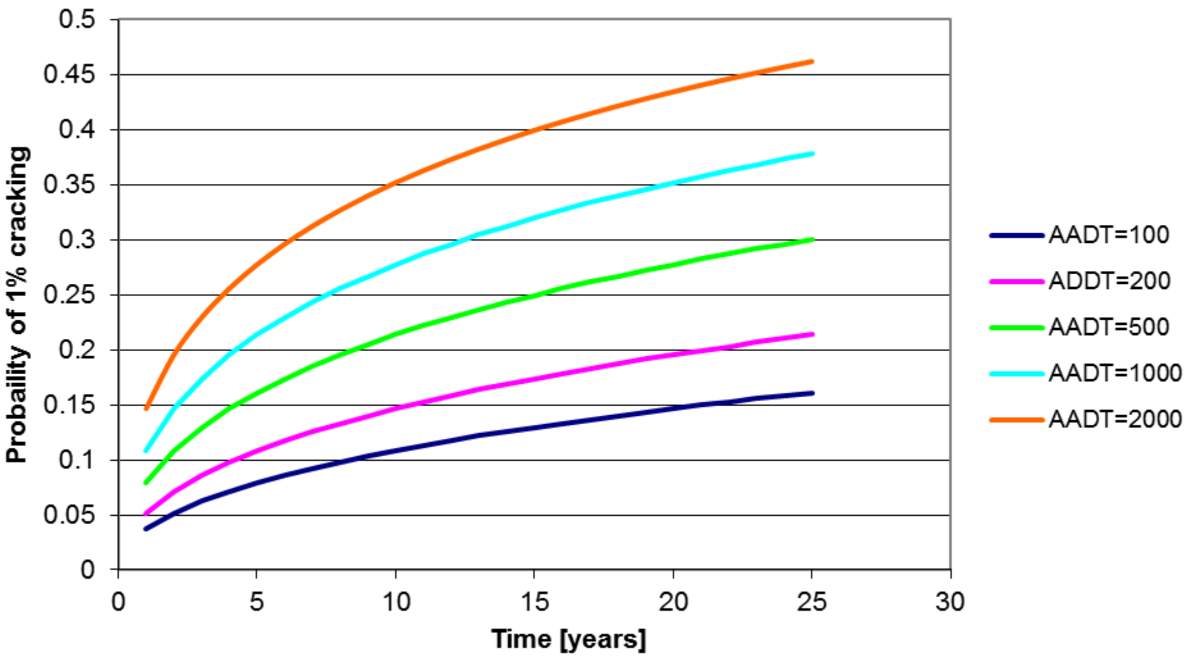
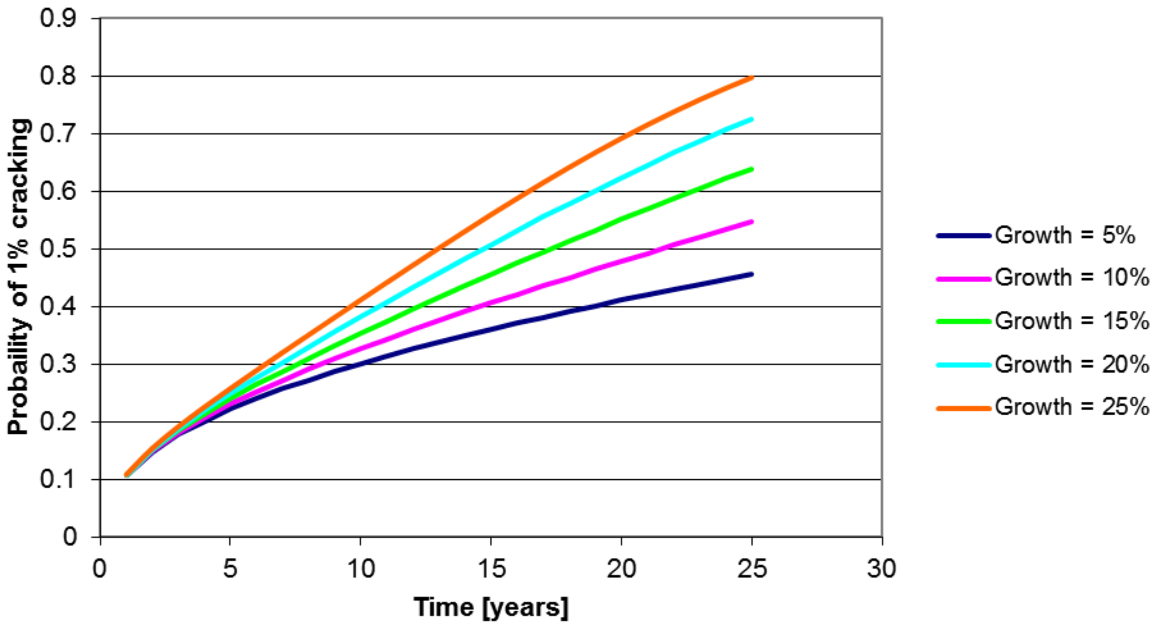


Figure 5.5 Probability of 1% alligator cracking against time. Cracking probability modelled over a range of traffic volume growth rates using the Ball alligator cracking model



6 Vehicle specific extreme loading

As mentioned in chapter 1, damage to roads resulting from extreme loading can occur through two ways:

1 diversion of traffic brought about by an emergency situation

2 planned or programmed change in road use resulting in increased traffic flows of a particular vehicle configuration, eg in areas where production forests enter the harvesting phase.

This chapter presents a methodology for assessing the impact on pavement condition of a sudden increase in HCV traffic. It utilises procedures given in chapter 7 of the *Austroads pavement design guide* (Austroads 2012) for determining traffic loading of specific axle configurations.

6.1 Calculation of ESA

With reference to Austroads (2007), the damage due to different axle groups is dependent on the axle spacing, the number of tyres per axle, the load on the group and the suspension. Axle groups are considered in terms of the following five types:

- single axle with single wheels (SAST)
- single axle with dual wheels (SADT)
- tandem axle both with single wheels (TAST)
- tandem axle both with dual wheels (TADT)
- tri-axles all with dual wheels (TRDT)
- quad-axle with dual tyres (QADT).

The damage associated with any particular axle load can be expressed relative to a reference axle group termed 'the standard axle'. The standard axle is a single axle with dual tyres (SADT) applying an axle load of 80kN to the pavement. Loads on the axle configurations given above that cause the same amount of damage as the standard axle are given in table 6.1.

Table 6.1 Axle loads which cause equal damage

Axle configuration		Reference load	
Type	**Description**	**kN**	**tonnes**
SAST	Single axle, single tyre	53	5.4
SADT	***Single axle, dual tyre***	***80***	***8.2***
TAST	Tandem axle, single tyre	90	9.2
TADT	Tandem axle, dual tyre	135	13.8
TRDT	Tri-axle, dual tyre	181	18.5
QADT	Quad-axle, dual tyre	221	22.5

For axle group loads other than those in table 6.1, the damage caused is expressed as the number of standard axles that produce the same damage and is calculated as follows:

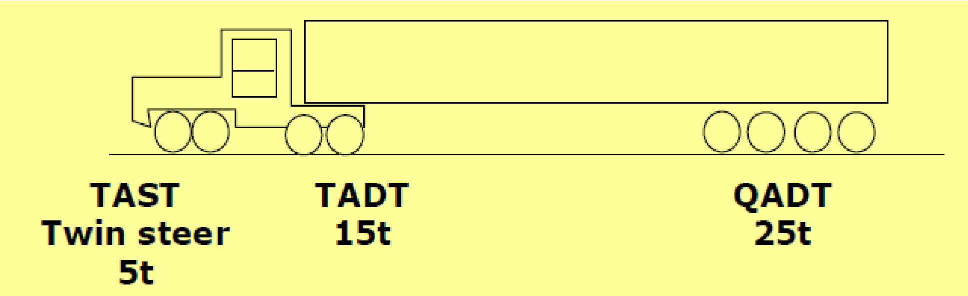
$$No.\,of\ standard\ axles\ for\ same\ damage = \left[\frac{Load\ on\ Axle\ Group}{Appropriate\ Load\ from\ Table\ 6.1}\right]^{EXP} \quad \text{(Equation 6.1)}$$

where the exponent EXP may vary depending on the type of pavement

If the ratio within the brackets of equation 6.1 (ie applied load to standard load ratio) is less than or equal to 1, a value of 4 is commonly adopted for the exponent, EXP, in which case the number of standard axles required to produce the same damage is termed the number of equivalent standard axles (ESAs). However, if this ratio is greater than 1, then a more accurate value for EXP can be calculated from figure 2.1. For example, for a SNC of 1.8 the value of EXP is calculated to be 2.64.

An illustrative example for calculating ESA for a semi-trailer configuration is provided in figure 6.1.

Figure 6.1 Example calculation of ESA



If SNC of pavement is not known:

$$\text{ESA}=\left[\frac{5}{9.2}\right]^4+\left[\frac{15}{13.8}\right]^4+\left[\frac{25}{22.5}\right]^4=3.00\ \text{ESA}$$

If SNC of pavement is known:

(example ESA calculation below assumes SNC = 1.8 giving EXP = 2.64)

$$ESA=\left[\frac{5}{9.2}\right]^4+\left[\frac{15}{13.8}\right]^{2.64}+\left[\frac{25}{22.5}\right]^{2.64}=2.65\ ESA$$

For more detailed information regarding relationships between pavement depth, subgrade California bearing ratio, aggregate quality, SNC and damage exponent (EXP), the reader is directed to Arnold et al 2005.

6.2 Log and milk transport

During the period 2006/07 to 2031, the freight task of wood, milk and dairy products is expected to increase by about 70% to 75% in terms of tonnes-km transported. Therefore, the forestry and dairy industries can be expected to be a major cause of increased demand on the use of rural roads.

The predominant vehicle configuration for these two industries at present, as can be seen in figures 6.2 and 6.3, is a 4-axle truck and 4-axle trailer with a gross vehicle weight (GVW) rating of 44 tonnes and maximum overall length of 20m. The loading on each axle group is 3.96 tonnes per TAST and 6.013 tonnes per TADT giving 2.30 ESA, calculated as follows:

$$\left[\frac{3.96+3.96}{9.2}\right]^4+\left[\frac{6.013+6.013}{13.8}\right]^4+\left[\frac{6.013+6.013}{13.8}\right]^4+\left[\frac{6.013+6.013}{13.8}\right]^4=2.30\ ESA \quad \text{(Equation 6.2)}$$

Figure 6.2 Representative log truck

Figure 6.3 Representative milk tanker

6.3 Illustrative extreme loading scenarios

By way of illustrative example, various extreme loading scenarios were considered for a weak strength (SNC = 1.8) dual lane rural carriageway. The analysis time period is 30 years from the construction/rehabilitation of the pavement, which relates to the target design life. The design traffic is 1000 AADT and 10% HCV corresponding to an ESA loading (ESAL) of 35 ESAs per lane per day. The pavement roughness immediately post construction is assumed to be 2.7 IRI m/km (ie 70 NAASRA counts/km).

The various extreme loading scenarios take place after year 12, when the pavement has reached a lane roughness of 3.8IRI m/km (ie 100 NAASRA counts/km) and are as follows:

1 Scheduled harvesting of a plantation forest results in the introduction of 44 tonne log trucks carrying a full load every half hour over a 12-hour period (6am to 6pm). This corresponds to an additional ESAL of 55 ESAs per lane per day (ie 2.3 ESA × 24).

2 The 10% HCV traffic now is directional so that the fully loaded HCVs travel only in one lane. This corresponds to an additional ESAL of 35 ESAs per lane per day.

3 There is a fourfold increase in general traffic from 1000 AADT to 4000 AADT resulting from traffic being detoured from a state highway. The %HCV remains at 10% so the additional ESAL corresponds to 105 ESAs per lane per day (ie [0.7 × (4000-1000) × 0.1] ÷ 2).

4 Detour of wharf traffic results in a fully loaded A-train (ie 2.18 ESA) passing by every eight minutes throughout the day (24-hour period). This corresponds to an additional 180 truck passes or additional ESALs of 392.4 per lane per day (ie 2.18 ESA × 180).

Two durations have been modelled with each of the extreme loading scenarios:

1 The extreme loading remains from year 13 until the end of the design life of the pavement, which is taken to be year 30.

2 The extreme loading only occurs for two years after which the traffic returns to the design values of 1000 AADT and 10% HCV at year 15.

A modified form of the Cenek–Patrick model was used to investigate the impact of the above extreme loading scenarios on pavement condition and is reproduced as equation 6.3 below for ready reference.

$$RN(t_n) = RN(t_{n-1})e^{m(\Delta t)} + [0.2175(1+SNC)^{-4.99}ESAL(\Delta t)]e^{mt_n} \qquad \text{(Equation 6.3)}$$

where: $RN(t_n)$ = IRI roughness at time t_n

$RN(t_{n-1})$ = IRI roughness at time t_{n-1}

Δt = change in time, ie $t_n - t_{n-1}$

SNC = the modified structural number

m = environmental coefficient

$EASL$ = equivalent single axle load per day.

This model was selected for the following reasons:

- It allows evaluation of the change in IRI lane roughness over any arbitrary time period without having to determine the roughness value of the pavement at time of construction or immediately after shape correction.
- It has been found to provide good agreement with observed roughness progression provided an appropriate value of environmental coefficient, m, is used (Cenek and Patrick 1991). In this case the default value of 0.0153 has been used, though if time series roughness data is available for the road section or region of interest, m can be can be simply approximated by IRI_2-IRI_1= m.IRI_1.t where IRI_1 and IRI_2 are the IRI lane roughness measurements at two different times and t is the time interval between the measurements (Cenek and Locke 2003). The roughness environmental coefficient, m, can therefore be estimated from the following relationship:

$$m = \frac{\left\{\left(\frac{IRI_2}{IRI_1}\right) - 1\right\}}{t}$$

- The model predictions can be readily validated as 100% roughness surveys are routinely performed on both state highways and local authority roads resulting in comprehensive roughness databases being available in RAMM.

Figure 6.4 graphically compares the predicted roughness progressions up to year 30 for the various extreme loading scenarios for the situation when they continue from year 13 to year 30. Figure 6.5 graphically compares the predicted roughness progressions up to year 30 for the various extreme loading scenarios for the situation when they occur only during a limited period (year 13 and year 14).

Figure 6.4 Comparison of predicted roughness progressions – duration of extreme loading 17 years

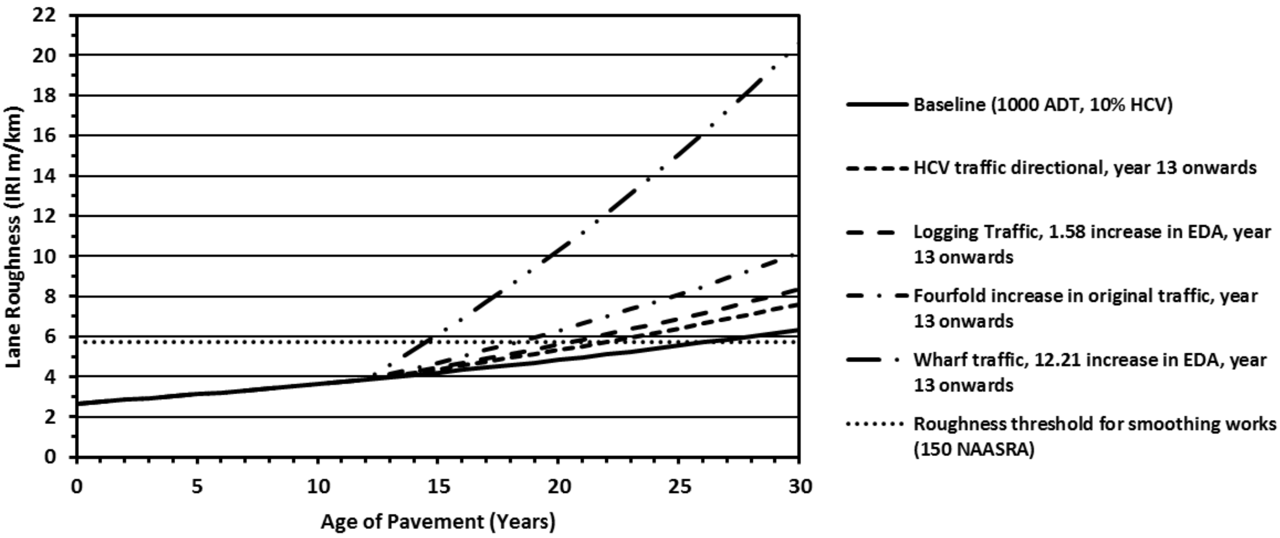


Figure 6.5 Comparison of predicted roughness progressions – duration of extreme loading 2 years (years 13 and 14)

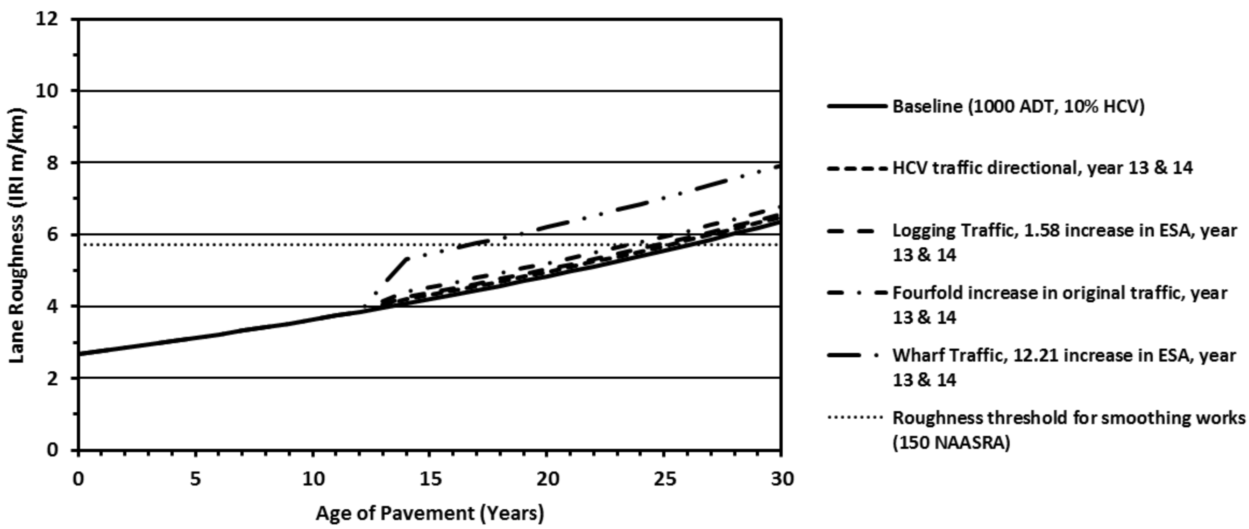


Table 6.2 summarises the key findings from the roughness progression modelling of the extreme loading scenarios.

Table 6.2 Time to reach end-of-life condition for different extreme loading scenarios

Loading scenario	ESAL per day	Time to reach end of life (150 NAASRA counts/km)	
		Duration of extreme loading	
		2 years	17 years
Baseline	35	27	27
Directional flow	70	26	22
Log truck traffic	90.2	25	21
Fourfold increase in traffic	140	24	19
Wharf traffic	427.4	17	15

With reference to figures 6.4 and 6.5 and also table 6.2, it is apparent that a typical extreme loading over a short period is predicted to have a minimal impact on the useful life of the pavement, ie a fourfold increase in the traffic over a two-year period is estimated to reduce the life of a weak pavement by three years, corresponding to a 11% reduction. However, extreme loading over a significant period clearly shows the need for detailed traffic characteristics when considering the likely impact on pavement condition. Particularly important is the direction split of fully laden HCVs and their axle configuration.

If there is a significant change in ESAL (ie tenfold or more increase), then significant deterioration can occur in a relatively short time frame. In the case of wharf traffic example, the useful life of the pavement has essentially halved from 30 years to 17 years through only two years' exposure to the higher loading level.

The typical repeatability of lane roughness measurements is ±0.2 IRI m/km. Therefore, to be confident that a measured change in yearly lane roughness due to extreme loading is real and not due to measurement inaccuracy, the difference in successive year lane roughness measurements has to be at least 0.4 IRI m/km. Applying equation 6.3, the change in ESAL required to produce a change of 0.4 IRI m/km over a year period in weak pavements (ie SNC = 1.8) is 300 ESA per day. This suggests that effects of extreme loading will only show up in RAMM condition data trending for significant changes in ESAL, ie tenfold or more increases.

7 State highway sites

As mentioned previously, actual pavement condition data for roads suffering extreme loading was very hard to come by. This was mainly due to:

- insufficient traffic loading and condition data recorded for the majority of roads suffering extreme loading
- the damage caused by extreme loading being rectified before the progression of pavement distress could be measured.

Although data was difficult to come by, three state highways were identified as having deteriorated due to extreme loading:

- SH32, RS57 and RS69 were identified as having been subjected to extreme loading caused by log trucks. This loading occurred in the right-hand lane and started in early 2008. This extreme loading was in the form of approximately 150,000 tonnes of logs per year being transported by 44 tonne log trucks of the configuration shown in figure 6.2.
- SH7, RS115 and SH63, RS46 and RS59 were exposed to extreme traffic loading resulting from a slip closing SH1S near Kaikoura on 10 September 2010. Accurate data on the daily heavy vehicle volume was available along with descriptions of the pavement distress caused.

7.1 Extreme loading of SH32

7.1.1 Traffic loading

Before the performance of the various models could be evaluated, the traffic loading needed to be calculated. It was known from logging data that approximately 150,000 tonnes of logs were transported along SH32 by log trucks, 30 tonnes at a time. This corresponds to 5000 log truck passes over the course of a year, or 13.7 per day.

The original AADT in each lane for RS57 and RS69 were 390 and 379 respectively. As the composition of the traffic was not known, the original traffic flows were split up using a standard traffic composition from appendix A2 of the *Economic evaluation manual*, volume 1 (NZTA 2010), commonly referred to as EEM1, which has been reproduced in table 7.1 for ready reference.

Table 7.1 Traffic composition for rural state highways (NZTA 2010)

Traffic type	CAR	LCV	MCV	HCV I	HCV II	BUS
Composition	81%	9%	3%	4%	3%	0%

The logging trucks used on SH32 had eight axles and were loaded to 44 tonnes (ie 14 tonne self- weight + 30 tonne payload of logs). Therefore they were added to EEM1's HCV II class. By including the logging trucks in the HCV II class the traffic compositions for RS57 and RS69 were calculated and are shown in table 7.2.

Table 7.2 Percentage composition of the base traffic flow combined with the extreme loading

Traffic type	CAR	LCV	MCV	HCV I	HCV II	BUS
RS 57 composition	78.3%	8.6%	2.9%	3.9%	6.3%	0%
RS 69 composition	78.1%	8.7%	2.9%	3.9%	6.4%	0%

The majority of the models used for this research use traffic loading data in terms of ESA. Equation 7.1 was used to calculate the ESA loading for the two sections of SH32 of interest (ie RS57 and RS69).

$$\text{million ESA/lane/year} = 365*[0.35*MCV+0.83*HCVI+1.86*HCVII+0.5*BUS]/[\text{No. of lanes} * 10^6] \quad \text{(Equation 7.1)}$$

where: MCV = the daily number of medium commercial vehicles

HCVI = the daily number of heavy commercial vehicles, type 1 (3-4 axles)

HCVII = the daily number of heavy commercial vehicles, type 2 (≥5 axles)

BUS = the daily number of buses

The traffic loadings used in the modelling were:

- RS57, 0.023725 million ESA/lane/year
- RS69, 0.022752 million ESA/lane/year.

The change in traffic loading in terms of ESA/lane/day was from 38 to 64. For the year of loading associated with the harvesting of logs, this is equivalent to approximately one year of pavement life.

7.1.2 RAMM data

High-speed survey data was extracted from the RAMM database for SH32. The latest high-speed survey completed before the start of the extreme loading was made on 23 January 2008 while the survey immediately after the onset of extreme loading was made on 21 November 2009. (Rating data was also taken from the RAMM database for both RS57 of SH32 and RS69 of SH32. This data mainly referred to rutting and cracking data, required for the HDM-4 roughness progression model discussed in section 7.1.3.)

Unfortunately the results of the cracking models cannot be compared to the actual deterioration of SH32 due to insufficient cracking data being available, leaving just roughness and rutting.

7.1.3 Roughness

When the roughness data was analysed for the period 23 January 2008 to 21 November 2009, it was found that the 300m average roughness on section RS57 had increased during the period of extreme loading, as expected, whereas the 300m average roughness on section RS69 had decreased. RS57 of SH32 had significant lengths resealed during this period. These resealed sections were removed before average roughness values were calculated leaving RP4.00 – 8.00 in the decreasing lane. When the data was analysed over greater length of time, additional chainages were removed due to resealing, eg when the analysis period was increased to 2003–09 only RP0.000–0.520 of RS69 was not resealed and was included in the analysis.

The change in roughness in terms of 100m values for the two sections is given in tables 7.3 and 7.4. It can be seen that in terms of standard deviations and averages there has been no change in the level of roughness from 2008 to 2009 and 2010.

Table 7.3 Roughness (NAASRA) in terms of 100m values for RS57 of SH32

Statistic	2008		2009		2010	
	Left lane	Right lane	Left lane	Right lane	Left lane	Right lane
Standard deviation	23	25	22	28	23	30
Average	87	91	83	92	77	88

Table 7.4 Roughness (NAASRA) in terms of 100m values for RS69 of SH32

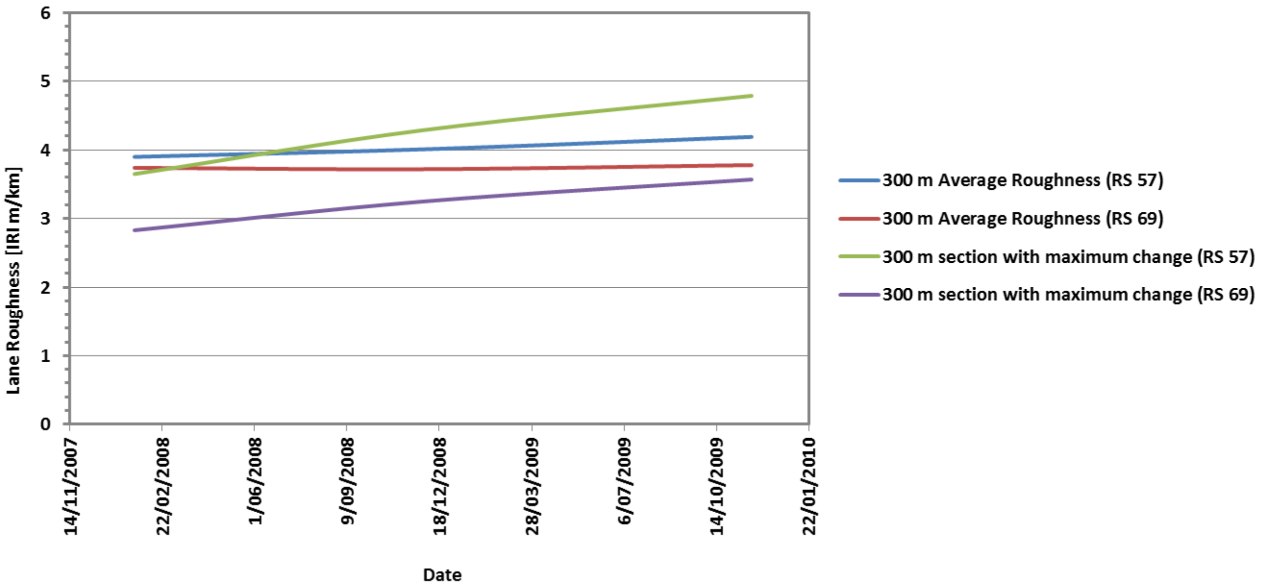
Statistic	2008		2009		2010	
	Left lane	Right lane	Left lane	Right lane	Left lane	Right lane
Standard deviation	26	25	24	25	24	26
Average	98	97	95	97	94	98
Lane average	98		96		96	

Four measures of roughness progression were used as follows:

- the average roughness values of sections of RS57
- the average roughness values of sections of RS69
- the largest change in roughness over 300m lengths of section RS57
- the largest change in roughness over 300m lengths of section RS69.

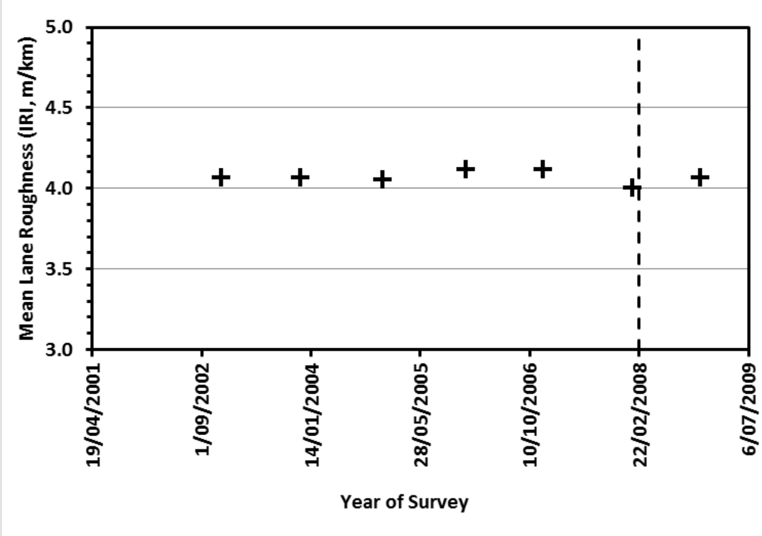
The roughness data obtained from the RAMM database for the period 23 January 2008 to 21 November 2009 for these four measures is shown below in figure 7.1.

Figure 7.1 Roughness progression for SH32 – sections RS57 and RS69 for period 23 January 2008 to 21 November 2009



The RAMM average roughness data for sections of SH32/RS57 and SH32/RS69 when considered over a five-year period is shown in figure 7.2. The vertical dashed line coincides with the approximate date when the traffic increased.

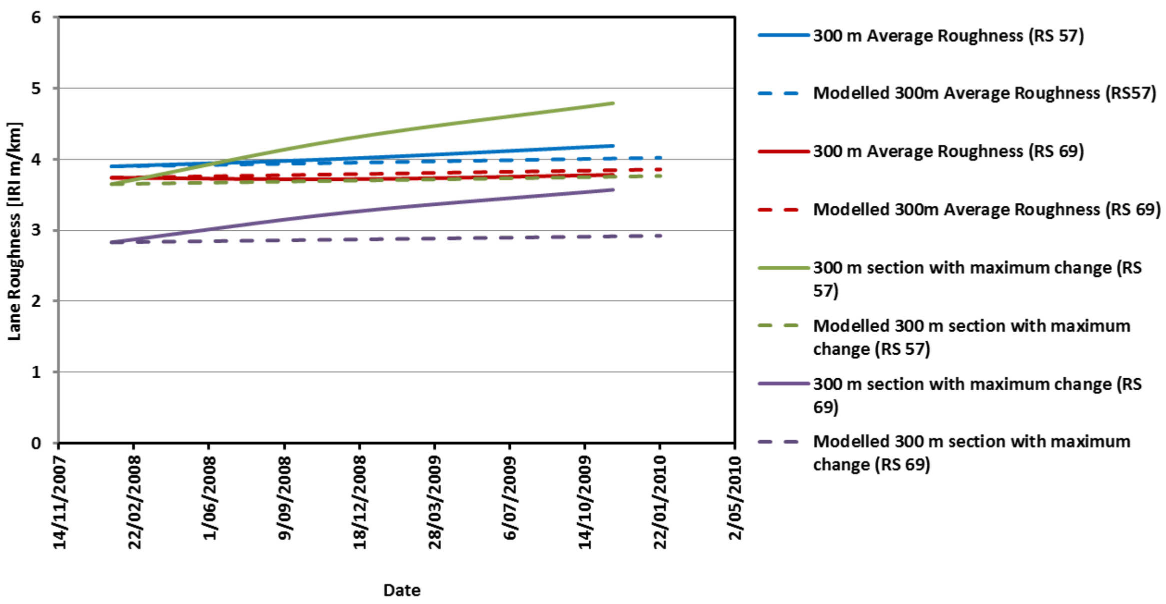
Figure 7.2 Average roughness of right-lane sections of SH32/RS57 and SH32/RS69 over a seven- year period



7.1.3.1 NZ-dTIMS roughness model

The NZ-dTIMS roughness progression model was applied to the data from SH32 to investigate how well the model showed the progression of the average and maximum RAMM roughness values. A SNC of 1.8 was used for the simulation to model the worst case roughness progression. The pavement thickness as of early 2008 was taken from the RAMM database as 200mm and the monthly rainfall was taken as 83mm[1]. The results of this dTIMS modelling are shown below in figure 7.3.

Figure 7.3 RAMM roughness progression for SH32 along with the modelled roughness using the NZ-dTIMS roughness model



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[1] www.teara.govt.nz/en/climate/2/2 (Accessed August 2011)

The modelling suggests there should have been a minimal change in roughness, which on average has occurred. The maximum change on each section is approximately 0.5 IRI which again can be regarded as minimal.

7.1.3.1 HDM-4 roughness model

Section 5.1 showed that the HDM-4 roughness model displayed a reasonable amount of sensitivity to both traffic volume and the growth in traffic volume. Therefore, it was decided to compare the predicted roughness using this model with the actual observed roughness progression on SH32.

As discussed earlier in chapter 5, the HDM-4 roughness progression model retains the basic form of the HDM-III component incremental roughness model with the addition of terms accounting for roughness due to ravelling and shoulder usage on narrow roads (ND Lea International Ltd 1995). As the HDM-III version of the component incremental roughness model is in a simpler form and excludes terms that are not vital for this research, it was used to assess the model's ability to model roughness progression due to extreme loading.

For the comparison performed in this section a wide variety of variables were required. (When available these variables were taken from the RAMM database. However, in some cases the variables had to be estimated based upon knowledge of the road surface rating.) Some of these variables were set as follows:

- SNC – a modified structural number of 1.8 was used to represent the weakest portion of pavement for each section
- H – the thickness of the pavement was taken as 200mm for both RS57 and RS69.
- CRX – the value of cracking given in the RAMM database is in terms of length of wheel path cracking. To convert this into the percentage area of all cracking, equation 7.2 below (RIMS1999) was used:

$$ACRA = 0.0004\ LWC^2 + 0.28\ LWC \qquad \text{(Equation 7.1)}$$

where: $ACRA$ = the percentage area of all cracking

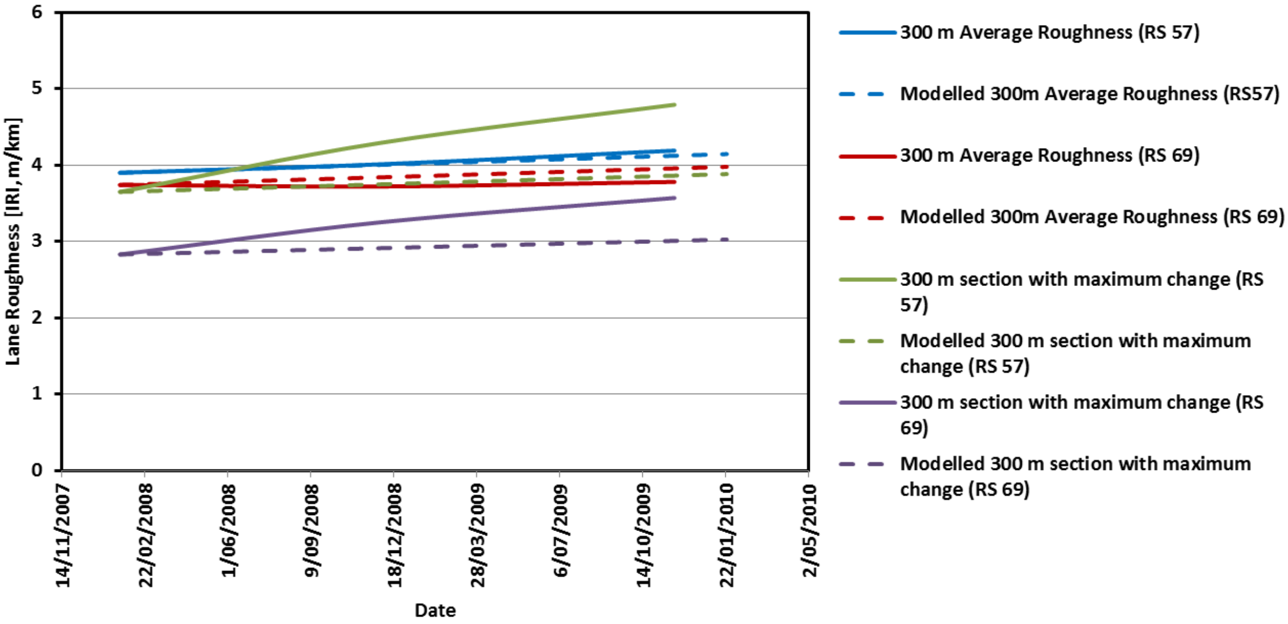
LWC = the length of wheel path cracking in m

- RS57 and RS69 showed signs of alligator cracking. The value of cracked length from the rating length showing the longest wheel path cracking was taken from RAMM for each section. Once converted using equation 7.2, the percentage area of all cracking was found to be 4.90% and 5.10% for RS57 and RS69 respectively.
- ΔRDS. The standard deviation of rut depth was taken to be zero as the rating data from the RAMM database showed no signs of rutting being present.
- ΔCRX. The change in area of indexed cracking was estimated at 0.5%. This was based on the fact that the areas of both road sections showing large amounts of cracking had not been resealed in several years. Assuming a rate of 0.5%, each pavement section had sufficient time for crack initiation before deteriorating to their current level.
- As the rating table in the RAMM database showed there was some patching occurring in the same rating length as the maximum alligator cracking, the patching term was considered initially in the modelling. However its contribution was minimal and so was eventually neglected.
- For this comparison the effects of potholes were neglected. This was due to the significant previous damage required for potholing and the little evidence of potholing found in the RAMM database.

- The traffic loading was taken as 65 ESA/lane/day and 62.3 ESA/lane/day for RS57 and RS69 respectively.
- The initial roughness values were taken as the values measured in January 2008 from figure 7.3.

The results of the modelling are shown in figure 7.4 below. The pavement roughness on average did not change significantly with extreme loading and the modelling also did not tend to suggest that a rapid change in roughness would be expected, even with a very low structural number.

Figure 7.4 Results of the comparison between HDM-4 component incremental roughness model predictions and RAMM/actual roughness data



7.1.3.1 Cenek–Patrick roughness model

Figure 7.5 compares for SH32 the roughness progression predicted by the Cenek–Patrick model with RAMM data. As can be seen, the results are comparable to those obtained with the d-TIMS and HDM4 models.

7.1.4 Rut depth

Figure 7.6 gives the RAMM-recorded rut depth over a five-year period. Again the vertical dashed line coincides with the approximate date when the traffic increased.

There is some evidence to suggest that the rut depth readings in the left wheel path may have been suspect for the 2008 survey[2] and so the dip observed is likely to be caused by measurement error rather than a true reduction in rut depth. Therefore, considering the overall trending, it is apparent that the increase in HCV traffic on SH32 has not had a significant effect on the rut depths of the sections considered.

[2] Email correspondence with Gerhard van Blerk, NZTA's National Technical Advisor (Pavements), 26 June 2012

Figure 7.5 RAMM roughness progression for SH32 along with the modelled roughness using the Cenek–Patrick roughness model

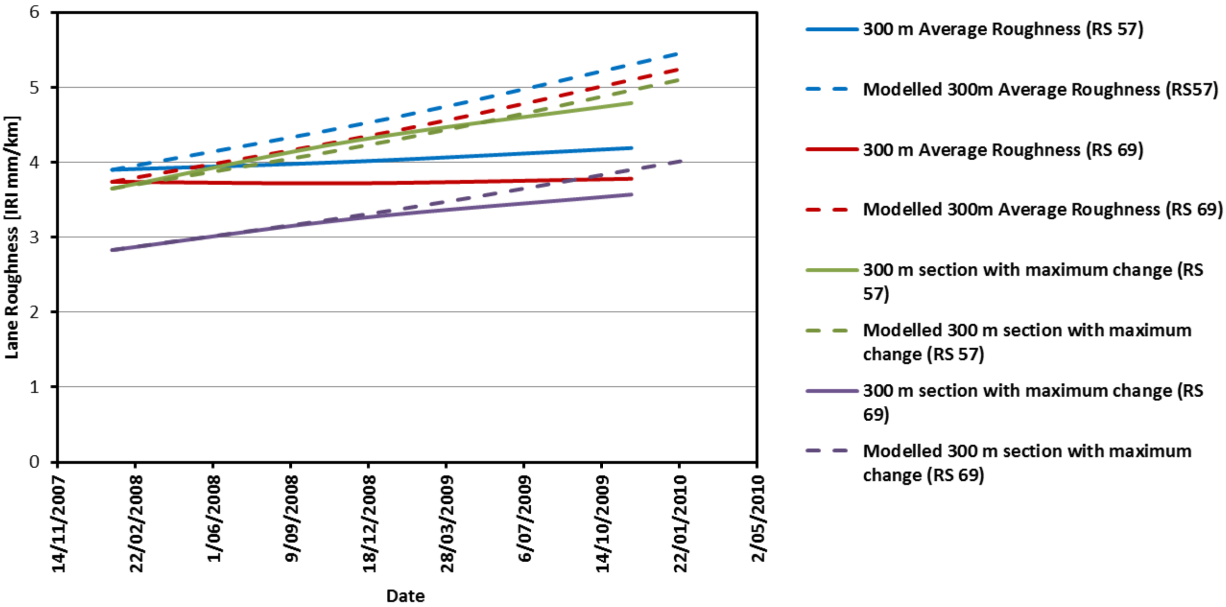
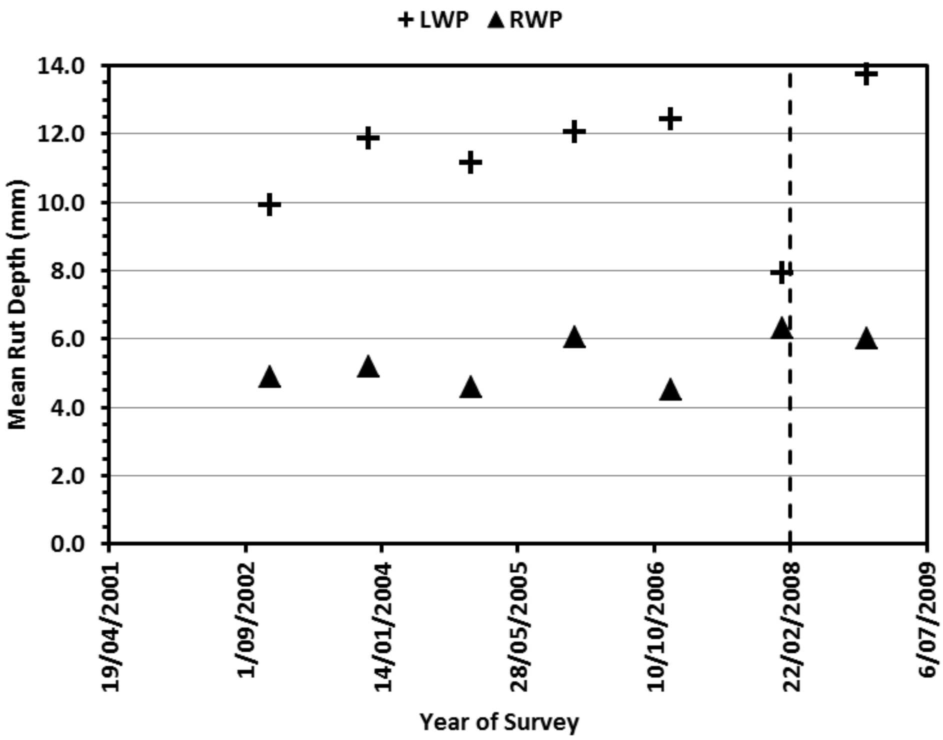


Figure 7.6 Average right-lane rut depth of SH32/RS57 and SH32/RS69 over a seven-year period



7.2 Extreme loading of SH7 and SH63

On 10 September 2010, a large slip occurred on SH1 near Kaikoura which closed the road for a total of six days. During this time a large amount of traffic was diverted onto SH7 and SH63. Accurate volumes of HCVs travelling on these highways were recorded over the six days. Figure 7.7 shows the increase in HCV volume on SH7, while figure 7.8 shows the same for SH63.

From this traffic loading data it was found that for SH7 the HCV traffic volume had increased by a factor of 3.6, while for SH63 the HCV traffic volume had increased by a factor of around 12.3. Assuming that not just HCV vehicles were diverted onto these roads but all traffic that normally would be travelling on SH1, the traffic volumes for SH7 and SH63 would have been the normal traffic volumes multiplied by a factor of 3.6 and 12.3 respectively. The original traffic volumes for these sections of road were taken from the RAMM database. As the composition of the traffic was not known, the composition given in table 7.1 was applied using equation 7.1 to obtain estimates of traffic loading in million ESA/lane/year.

For SH7 the traffic increased from averaging 156 HCV/day to 3315 HCV over a seven-day period. This extra traffic is equivalent to 474 HCV/day or an increase of 316 HCV/day. In terms of the life of the pavement this is equivalent to an approximate tripling of the HCV for 7 days or 14 days off the pavement life. For SH63 the increase in heavy traffic was more significant and was equivalent to an extra 60 days of traffic. However, for both SH7 and SH63 these durations of extra traffic do not appear to have been sufficiently long to have visibly affected pavement condition data. For example, figure 7.9 below shows RAMM survey data for rut depth prior and post the diversion of traffic, the vertical dashed line coinciding with the approximate date when the traffic increased. This is consistent with the analysis performed in chapter 6, which suggests that traffic has to be diverted for a considerable period of time (ie in the order of years) before significant deterioration becomes apparent, given New Zealand traffic volumes and inherent strength of the pavements.

Figure 7.7 Heavy vehicle traffic volume increase on SH7 caused by a slip on SH1

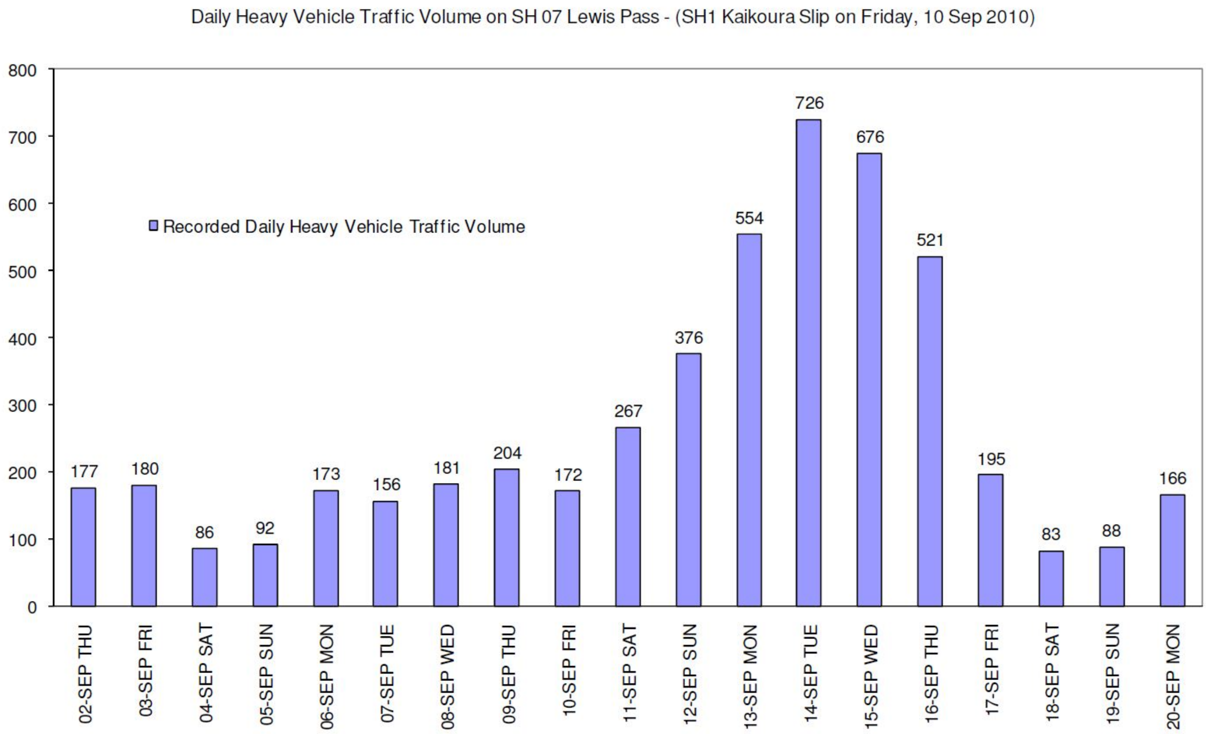


Figure 7.8 Heavy vehicle traffic volume increase on SH63 caused by a slip on SH1

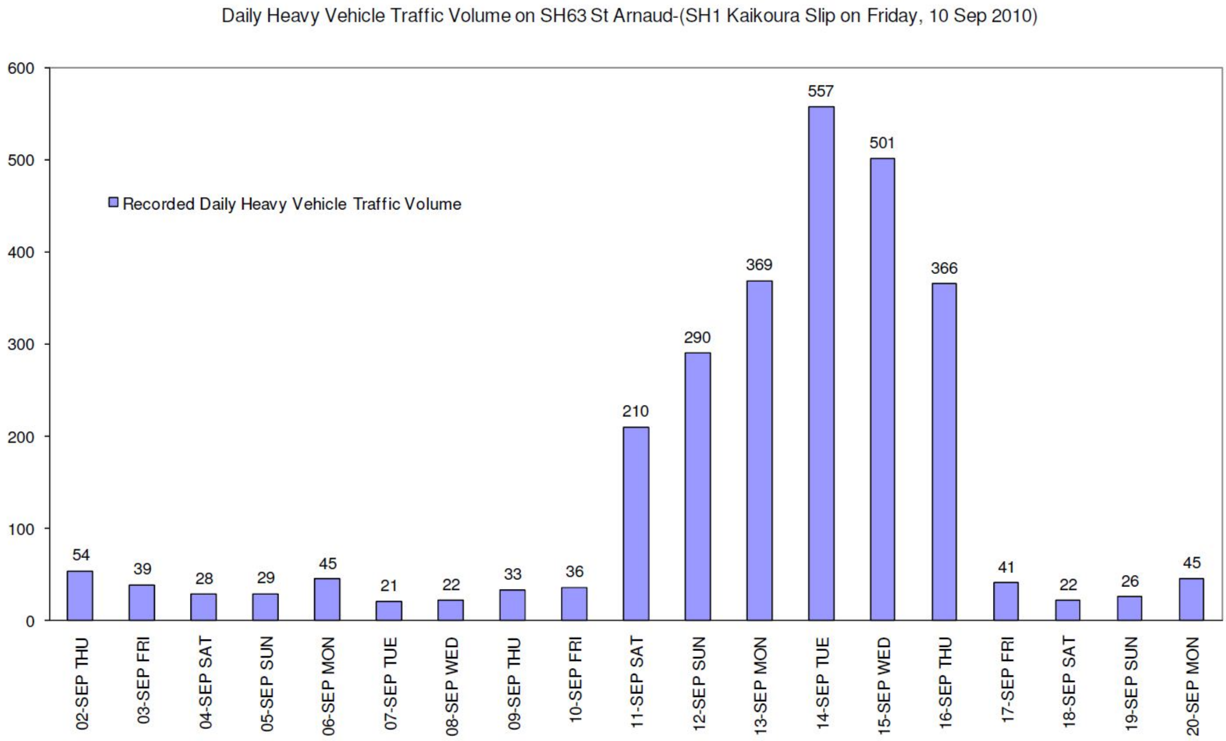
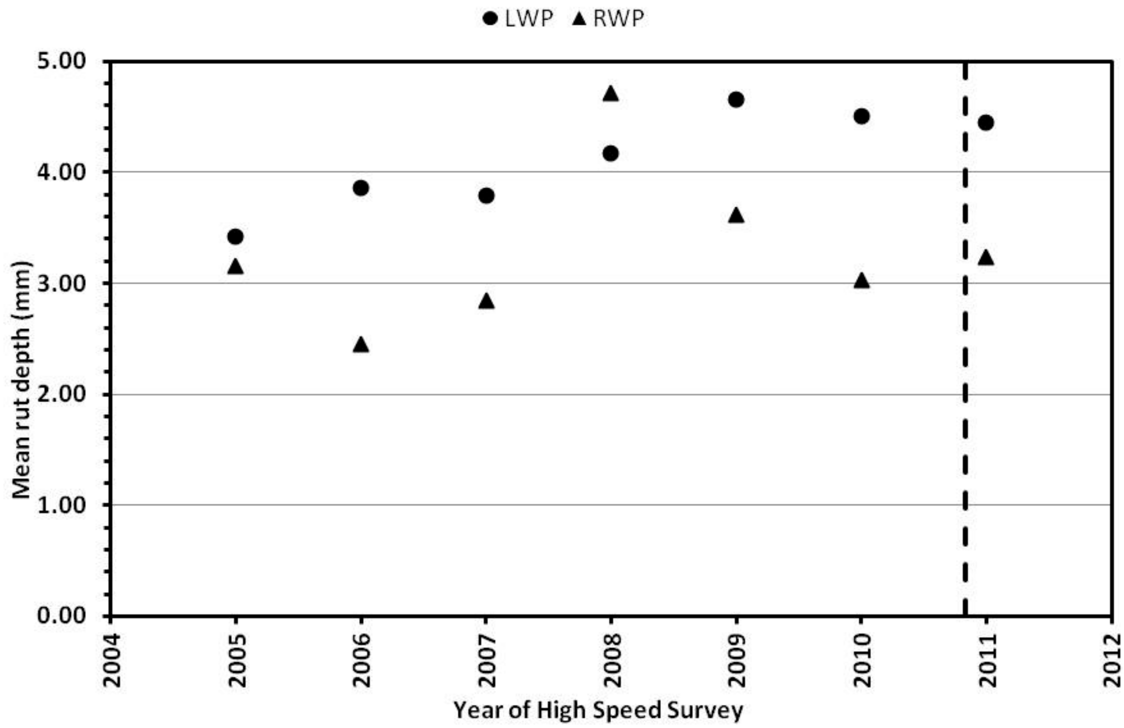


Figure 7.9 Average right-lane rut depth of sections of SH7 and SH63 over a five-year period



8 Maintenance costs

As the pavements investigated in this project did not suffer a rapid marked change in any RAMM-recorded pavement condition measures when subjected to a sudden increase in traffic loading, the effect of extreme traffic loading on RAMM-recorded maintenance activities and costs was examined. To achieve this, the maintenance activities and costs recorded in the RAMM-database for SH32, SH7 and SH64 were examined via plots. Examples are shown in figures 8.1 and 8.2 below. The vertical dashed line on these figures coincides approximately with the date at which the extreme loading commenced.

The maintenance costs plotted have been extracted from the 'Maintenance cost' table in RAMM, which has the facility to only record reactive maintenance expenditures.

From these two figures and chapter 7, it is apparent that while RAMM-recorded pavement condition measures have not changed markedly, pavement/surface maintenance costs for the studied sections of SH32 have.

To summarise, the main observations are as follows:

1. On SH32 the traffic increase was sustained for at least a year and there is an increase in maintenance costs (figure 8.1) even though there is no marked change in RAMM pavement condition data. This suggests extreme traffic loading may increase maintenance costs while RAMM-recorded pavement condition changes remain minor and undetectable (chapter 7).
2. On SH7 and SH63 the increase in traffic loading was sustained only for a very short time and does not appear to have resulted in an increase in maintenance costs (figure 8.2). This suggests extreme traffic loading has to be sustained for a long period to have a noticeable effect on RAMM maintenance cost data.

   An increase in reactive maintenance costs brought about by extreme loading may occur because of a number of factors. Speculatively, edge break on narrow roads may be the prime reason.

Figure 8.1 Pavement/surface maintenance costs of sections of SH32 over a seven-year period

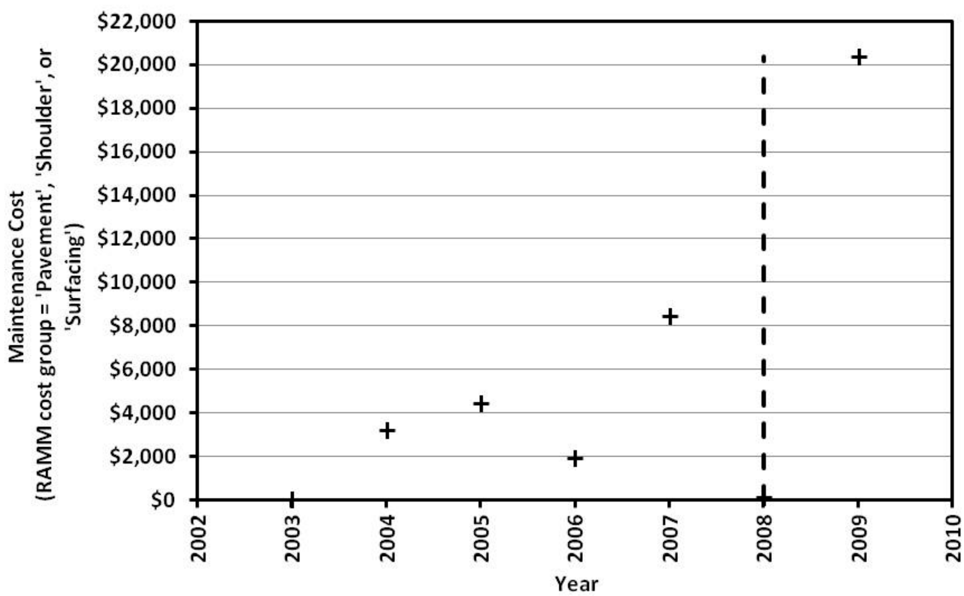
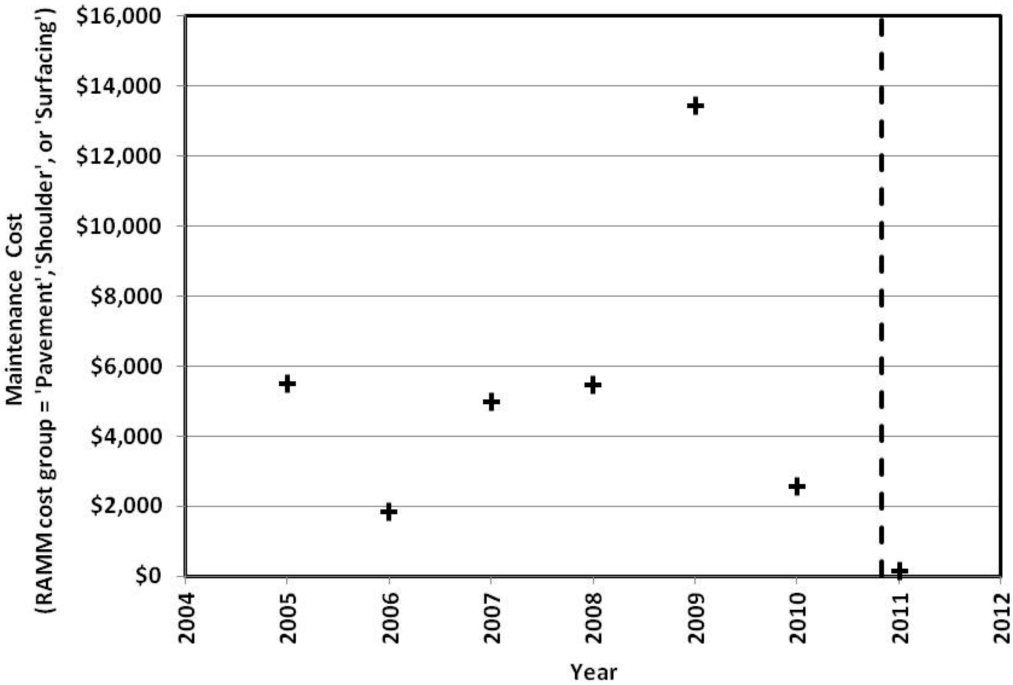


Figure 8.2 Pavement/surface maintenance costs of sections of SH7 and SH63 over a seven-year period



9 Discussion

Extreme loading issues that were not covered by the research but merit some discussion are the effect of frequent heavy passes in quick succession, the applicability of the fourth power rule when the ratio of the applied load to the standard load is less than 1 and the opportunity afforded by the closure of the Manawatu Gorge since 19 August 2011 to investigate extreme loading effects.

9.1 Frequency of heavy passes

Where there is a significant amount of moisture present in pavement aggregate or sub-grade soil, the effect of heavy axle loading in quick succession is believed to be more damaging than the same number of axle passes spread over a longer period (Frame Group 2003).

The effect is the result of the development of positive pore water pressure under rapidly applied loads. If successive loads are applied before the pore water pressure has time to dissipate and reach equilibrium, the pore water pressure continues to increase with each successive loading. Excessive pore pressure reduces effective shear strength resulting in permanent deformation of the pavement or sub-grade material.

This effect is a function of the time between successive loads and the permeability of the pavement or sub-grade material. When road pavements or sub-grades are saturated, provided heavy vehicle passes are separated by a time period of 30 minutes or more, it is unlikely that pore water pressure increase will be a factor in road deterioration. However, given the permeable nature of New Zealand chipseal pavements, it may be worthwhile to test the theory that frequent heavy passes will not cause more damage to rural roads than the same traffic over a more extended period. This could be readily researched by testing partially saturated and saturated chipseal pavements at CAPTIF.

9.2 Fourth power rule

The fourth power law has its origins in the AASHO (American Association of State Highway Officials) road test undertaken in North America during 1956–61 on sealed flexible pavements, but which did not include any thin surfaced pavements such as chipseals. Therefore, there is a need to confirm that the fourth power law is applicable to a range of 1) pavement depths, 2) California bearing ratios and 3) aggregate quality when the ratio of the applied load to the standard load is less than 1 as this is the most common HCV loading scenario. There is also a need to specifically investigate the dynamic loading characteristics of 8 axle log trucks and milk tankers as there is a suspicion that when fully loaded, the torsional rigidity of the trailer is significantly increased causing undesirable weight shifts and wheel scrub during cornering or when the vehicle encounters isolated road corrugations. A possible method for evaluating the damage potential of different loaded vehicle configurations is to monitor kerbside ground vibrations adjacent to a corrugated road section.

9.3 Manawatu Gorge closure

The on-going closure of the Manawatu Gorge since 19 August 2011 because of a series of landslides has caused SH3 traffic to detour along either Saddle Road or the Pahiatua Track. Given the high HCV traffic volumes and duration of the closure, this provides an excellent opportunity to collect detailed traffic composition data, pavement condition and reactive maintenance cost data for both Saddle Road and the Pahiatua Track so that procedures presented in this report for estimating the effect of extreme traffic loading can be better validated and refined if need be. There also appears to be a need for a data collection methodology for the industry formulated around extreme loading events to ensure that any data acquired is appropriate and in the correct format for researching the performance of local roads when exposed to frequent heavy loading.

10 Conclusions

The aim of this research was to establish whether the roughness progression models contained within the NZ-dTIMS and HDM-4 pavement management systems could be used to predict pavement condition under extreme loading. The original intended methodology was to retrospectively apply the models to data obtained from extreme loading sites. However, due to a lack of suitable data, the research also focused on the sensitivity of the models to traffic loading variables. Cracking and rutting in addition to roughness models were also investigated.

Investigations into the structural strength distribution on six local authority roads found that a lower limit of 1.8 exists for structural strength – an important discovery if the position of maximum pavement deterioration is confirmed as being related to minimum pavement strength.

The research also looked at the effects of the pavement structural strength on three of these six local authority sites. The main focus was on identifying the effects of within pavement strength variation on roughness so that more precision could be used in locating pavement deterioration. This was done by investigating the spatial relationship between roughness and structural strength.

With regard to rutting, it would seem that while, for example, the NZ-dTIMS Henning rutting model is sensitive to traffic growth, the moderate degree of this sensitivity may suggest this rutting model is not suited to modelling rut depth caused by extreme traffic loading.

Using the limited state highway validation data available, it was found that of the roughness models, the HDM-4 model gave the best correlation with maximum roughness change, while the NZ-dTIMS model gave the best correlation with average roughness change. As the research was most interested in the development of a conservative model capable of predicting pavement deterioration well in advance, the HDM-4 roughness model is recommended until further research can be performed.

Investigations of RAMM data for three state highways subjected to extreme traffic loading suggest that the effects of extreme loading may show up in RAMM maintenance cost data, but not initially in RAMM pavement condition survey data. Additionally, indications are that the extreme traffic loading must be sustained for a lengthy duration to show up in any RAMM data. These observations indicate that employing pavement deterioration models may be the preferable analysis route for detecting the effects of extreme traffic loading rather than attempting to analyse RAMM data. To enable this, available pavement condition models should be improved to better model pavement deterioration resulting from extreme traffic loading.

11 Recommendations

It is highly recommended that more research be performed into uncovering the true relationship between structural strength and roughness. This will lead to a better understanding of the location of maximum pavement distress and the development of more accurate pavement deterioration models.

As the roughness models investigated in this research were found to perform best when modelling average roughness, it would also be beneficial to understand the relationship between average roughness and maximum roughness. The same may be true of rutting.

To gain a better understanding of the true performance of the models investigated in this research, more extreme loading sites need to be identified and monitored. For a site to be useful in model validation it is important to have accurate information on:

- the commencement date of the extreme loading
- the traffic loading during the extreme loading, including frequency of HCV passes
- the rating of the road at the commencement of the extreme loading
- the roughness and rutting pavement condition survey data at the commencement of the extreme loading.

Once more data is available for model verification a more in-depth analysis can be performed looking at pavement deterioration models and their effectiveness at modelling the effects of extreme loading. This should include both examining existing models and the development of new models specifically from the extreme loading data obtained. In particular, emphasis should be placed on edge break as it is the failure mode most often associated with increased traffic loading on New Zealand rural roads. However, existing edge-break models take no account of pavement strength being based on lane width and traffic (AADT).

As well as the above research-related recommendations, it is also suggested that practitioners be aware:

- when investigating pavement deterioration resulting from extreme traffic loading that the consequences of the extreme traffic loading may show up in pavement maintenance cost data before pavement condition survey data
- that extreme loading must be sustained for some time to have a noticeable effect on any pavement deterioration data (including maintenance cost data) stored in the RAMM database. The exact duration would be a function of the magnitude of the increase in traffic relative to normal traffic levels.

Finally, in the interim, it is recommended that:

- either the HDM-4 or the Cenek–Patrick roughness model is used to predict the pavement roughness condition of extremely loaded roads. The Cenek–Patrick model is recommended due to its simple form and reasonable sensitivity to traffic loading, whereas the HDM-4 model is recommended as it provides a good prediction of maximum roughness progression. Although these models can be used as tools for helping to predict pavement condition, it should be remembered that they are not specifically designed for modelling extreme loading and may still significantly under-predict pavement deterioration.
- the NZ-dTIMS Henning model continues to be used for modelling rut progression.

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