Relationship between design and predicted performance of New Zealand pavements

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An important note for the reader

This report is the final stage of a project commissioned by Transfund New Zealand before 2004, and is published by Land Transport New Zealand.

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Abbreviations & acronyms

AADT	Average Annual Daily Traffic
AASHO	American Association of State Highway Officials, before 1973
AASHTO	American Association of State Highway and Transportation Officials, after 1974
Austroads	Australian Road Research Board, Sydney, Australia
CBR	California Bearing Ratio
dTIMS	Deighton's Total Infrastructure Management System
ESA	Equivalent Standard Axle
FWD	Falling Weight Deflectometer
HCV	Heavy commercial vehicle(s)
HDM III	Highway Design & Maintenance Standards model, Version III
LTNZ	Land Transport New Zealand
NAASRA	National Association of Australian State Roading Authorities
PSMC	Performance-specified Maintenance Contract
RAMM	Road Assessment & Maintenance Management
SN	Structural Number
SNC	Modified Structural Number
TNZ	Transit New Zealand, Wellington, New Zealand
TRL	Transport Research Laboratory, Crowthorne, UK, after 1992
TRRL	Transport & Road Research Laboratory, Crowthorne, UK, before 1991

Contents

Executive summary

Both the design of new pavements in New Zealand and their rehabilitation treatments are currently performed in accordance with the Austroads Pavement Design Guide and its New Zealand Supplement. New Zealand is also adopting pavement deterioration modelling based on the World Bank HDM models. This report demonstrates how the modelling of roughness progression can supplement this pavement design.

The research carried out in 2002-03 has shown that:

- Most shape correction and reconstruction projects on New Zealand pavements are driven by factors other than structural deterioration.
- Most of these pavements have significant remaining life when analysed using the Austroads Pavement Design Guide criteria.
- On the relatively lightly trafficked granular pavements (i.e. with granular thin surfaces) of New Zealand roads, a life in excess of 50 years is common.
- The roughness levels of these pavements are in the order of that predicted by the HDM III roughness model.
- The rut depths on all pavements measured were, in general, less than 10 mm and therefore not a reason for rehabilitation.
- For granular pavements designed to the Austroads Pavement Design Guide, the HDM III models predict that the roughness at the end of the design life will be less than 100 NAASRA counts when the design traffic is in the range of 10^5 to 10^6 ESA, but over 200 NAASRA counts for design traffic of 10^7 ESA.
- The use of HDM III roughness progression models is a powerful tool to supplement mechanistic design, especially in determining the effect of changing variables such as initial roughness.
- In terms of roughness there is a significant difference in the expected terminal condition of a pavement designed in terms of the Austroads Pavement Design Guide criteria and the condition determined from an HDM III roughness progression.

Recommendations obtained from the project are that:

- Further investigations are made of the reasons for rehabilitating pavements, and to determine if the increasing maintenance expenditure that may occur on a road section is associated with a surfacing failure or a structural failure of the pavement.
- The differences between the HDM-predicted change in shape and the terminal condition associated with the design criteria in the Austroads Pavement Design Guide should be examined to harmonise the two approaches.

- The approach used in this project should be incorporated in the New Zealand Supplement to Austroads Pavement Design Guide, to encourage pavement designers to explore the sensitivity and risk of their pavement designs to changes in pavement shape with time.
- An investigation should be carried out of the factors affecting maintenance costs and the assumptions used to generate the Net Present Value of future maintenance costs.

Abstract

Both the design of new pavements in New Zealand and their rehabilitation treatments are currently performed in accordance with the Austroads Pavement Design Guide and its New Zealand Supplement. New Zealand is also adopting pavement deterioration modelling based on the World Bank HDM models. The research carried out in 2002-03 demonstrates how the modelling of roughness progression can supplement pavement design. It also demonstrates that over the life, sometimes more than 50 years, of many pavements on New Zealand roads, it is not uncommon to find little significant roughness or rutting.

The conclusion is that a combination of deterioration modelling and mechanistic design can be a powerful tool to supplement mechanistic design, and that the rehabilitation of most of the network is associated with failure other than that caused by classic roughness and rutting.

1. Introduction

1.1 Background

Both the design of new pavements in New Zealand and their rehabilitation treatments are currently performed in accordance with the Austroads Pavement Design Guide (1992) and its New Zealand Supplement (1995). The Guide sets out a mechanistic approach to pavement design and also allows the pavement designer to select a ratio of initial to final roughness levels for the design life of the pavement (Austroads 1992). The Guide gives an estimate of the expected life of a pavement but does not give a guide to the rate of change of pavement shape. The recently updated Austroads Pavement Design Guide (Austroads 2004) has excluded the ability to modify the levels of roughness.

Recently New Zealand has introduced pavement deterioration modelling using the dTIMS system, which is based on HDM III models, for use on New Zealand roads. Pavement strength in dTIMS is defined in terms of the Structural Number (SN), which is an empirical method initially developed in the AASHO road test in 1962, and modified by TRRL in 1975 to include the contribution to strength from the subgrade. In contrast to the mechanistic design, where the methodology is concerned with 'design life', deterioration modelling is concerned with the rate of change of factors such as roughness and rutting.

The current Austroads design default analysis assumes that the initial pavement roughness is 50 NAASRA counts and the final is 150 NAASRA. This assumption has not however been proven under New Zealand conditions, where the initial roughness level is often 70 NAASRA counts and that rehabilitation is performed when the roughness is well under 150 counts.

The HDM III model suggests that the difference in initial roughness levels of 50 to 70 can be significant in determining the life of a pavement. For example, starting with a roughness level of 50 NAASRA counts will take 25 years to reach a value of 150 NAASRA counts, while the pavement that starts at 70 NAASRA takes only 20 years to reach 150 NAASRA.

The basis of the modelling and design is similar however, in that the pavement strength is described in terms of the contribution of the subgrade and each pavement layer.

The research carried out in 2002-03 aimed to relate the two methods so that the pavement designer can obtain the benefits of both approaches by obtaining two estimates of pavement life, as well as an indication of the rate of change of pavement shape.

The approach in this research should also help in the development of performance-based construction specifications such as TNZ B/3 (provisional) *Performance-based specification for structural design and construction of flexible unbound pavements* (2000). In this type of specification the need is to be able to develop rational bonus or penalty payment systems if the constructed pavement exceeds or fails the specification requirements. The ability to model the change in pavement shape with time allows rational payment systems to be developed.

This project will also assist in the development of Performance-specified Maintenance Contracts (PSMCs) because confidence in the relationship between design and performance will be better understood. This will assist in building confidence in both pavement design and deterioration modelling for setting and meeting performance levels.

One of the major issues in PSMC contracts is the concept of pavement life. The methodology currently used is based on the Austroads Pavement Design Guide and does not take into account the existing pavement condition. By relating the two systems with the Austroads and HDM models, more robust measures of remaining life will be possible.

1.2 Objective of the research

The objective is to combine the performance prediction relationships in the HDM models with the mechanistic approach to design pavements currently used for New Zealand roads.

The relationships were validated by analysing the performance of 88 road sections that have been investigated for rehabilitation. By combining the two methods, more robust predictions of pavement performance can be made which will assist:

- Pavement deterioration modelling,
- Bonus or penalty payments for performance-based specifications,
- Prediction of pavement performance in PSMCs.

2. Data collection

2.1 Site selection

A search of RAMM and Transit records was made to find road pavement sites where historical data were available and that had been programmed for reconstruction or shape correction. For a site to qualify as being of use in the research, the following information was needed:

- CBR[1] or Scala-derived CBR of the subgrade
- Description and thickness of each pavement layer
- FWD-derived modulus for each layer
- At least five years of roughness data
- Rut depth
- Pavement age
- An estimation of traffic mix and volume

2.2 Site locations

Sites identified from the search included roads in both South and North Islands that would provide a data sample that was representative of New Zealand roads.

Data suitable for the research was obtained from Southland and Hawke's Bay Transit NZ regions. Some of the data for these sites were obtained from pavements where Area-Wide Treatment (AWT) measurements had been performed. The Appendix lists all 88 sites that were originally selected. However some inconsistencies in the measured data obtained from historical records meant that the results of only 79 sites were used.

1 CBR – California bearing ratio.

3. Database construction

3.1 Data collection

Data obtained for the 88 sites were in various forms, from hard copies to electronic input to analysis programs. The RAMM database was used to extract the traffic and historic measurements of roughness and rutting for each site.

3.2 Restructuring the data

Much of the data needed to be processed and restructured in a format that could be worked with. Keeping the site name and ID number consistent throughout allowed the process to be monitored so that the correct site information could be matched to the correct site location.

For each site, basic site information such as

- site number;
- name;
- state highway;
- location;
- CBR; and
- date of construction,

were used as a summary table (see Appendix).

All relevant performance and traffic data for each site were combined in separate worksheets, containing summary information such as traffic per year and annual average NAASRA counts for the last 10 years. Structural data were stored separately in a single spreadsheet.

3.2.1 Road construction data

Structural data included:

- details of site rehabilitation dates;
- FWD[2] modulus of the pavement layers;
- FWD CBR of the subgrade;
- layer thickness;
- position of in-situ CBR test;
- in-situ subgrade CBR value.

At some sites, Scala penetrometer values were converted to the inferred CBR value using the standard relationship given in the Austroads Pavement Design Manual.

[2] FWD – Falling weight deflectometer.

Data obtained from the FWD measurements, as well as the performance data, covered a significant distance along each road section, with test pits located at various sites along that section. To relate the performance data and FWD results with the CBR measurements in one site, sites within each road section were identified by a measured CBR value.

3.2.2 Performance data

Data obtained from the RAMM database were roughness measurements for 1992 to 2001, and rutting measurements for 1996 to 2001.

Roughness and rutting data were matched to the structural data through the site name, ID, and the location of the CBR readings at each site.

From the rutting data, relatively little rutting was found to have occurred over the sites. Some sites were identified as having rutting over 10 mm (but few over 20 mm) at the beginning of data measurement in 1996. However, after this date and up to the rehabilitation date, no sites showed significant amounts of rutting greater than 10 mm. Therefore the decision was that the rutting of the pavement was neither critical nor the main reason for the pavement being rehabilitated. Thus the research concentrated on analysing the roughness measurements to pavement performance.

For the roughness data, a separate worksheet was used for each site. The roughness measurements taken from 1992 to 2001 were obtained for each site and the average NAASRA counts for the pavement location for each year were obtained.

3.2.3 Traffic data analysis

The traffic using the site for each year (ESA[3]/lane/day) was also included in each of the performance worksheets for each site.

As the traffic counts in 2001 were known, the ESA/lane/day for each previous year was calculated using the arithmetic growth model. The total traffic from the date of construction until the rehabilitation was calculated, together with the total traffic between the date of construction and the start of the roughness readings, i.e. 1992. Traffic counts in ESA/lane/day from the date of construction to the rehabilitation date were also calculated.

Annual heavy traffic loading for 2001 was calculated from the product of AADT, the percentage of heavy vehicles (HCVs), and ESA/HCV, divided by the number of lanes, as found from the RAMM database. Loadings for previous years were estimated from the arithmetic traffic growth model, provided in Transfund's Project Evaluation Manual (1997), p. A2-6. Care was taken to use the appropriate rate for each individual road in the dataset.

Obviously it is difficult to determine the exact amount of traffic that the pavement has experienced in its entire lifetime. However this should not significantly affect the results, as most traffic has occurred in the most recent past where the traffic readings are fairly well known.

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3 ESA – Equivalent standard axles.

4. Calculation of the Modified Structural Number

4.1 Introduction

Two modified structural numbers (SNCs) were calculated using two different sets of measurements. The two calculations were performed using either the FWD data only or the in-situ data only. The differences that can occur between the two methods have been described in Patrick (2001).

4.2 SNC equation

The SNC is defined by Equation 4.1 below, and can be described by the sum of the layer coefficient multiplied by the thickness of each layer plus the subgrade contribution to strength.

Modified Structural Number (SNC): Equation 4.1

$$SNC = \Sigma_i^n a_i h_i + SN_{sg}$$

where:

a_i = layer coefficient of layer i, obtained from the standard material coefficient a_g or the resilient modulus constant E_g in Table 4.1, and the material E_i the measured layer modulus:

$$a_i = a_g(E_i/E_g)^{1/3}$$

h_i = thickness of layer i

n = number of layers above the subgrade

SN_{sg} = structural number contribution from the subgrade, derived from Hodges et al. (1975) as follows:

$$SN_{sg} = -\ 0.85(\log CBR)^2 + 3.51(\log CBR) - 1.43$$

Table 4.1 Layer coefficients and resilient moduli of standard materials used to calculate structural number.

Layer Type	Layer Coefficient a_g	Resilient Modulus E_g (MPa)
Asphalt concrete surface course	0.44	3100
Untreated & stabilised basecourse	0.14	207
Granular sub-base course*	0.12	104

* updated from Cenek & Patrick (1991)

If all the required data are not available the SNC can be estimated using FWD data by a method developed by Tonkin & Taylor (1998).

For this research three methods were used to calculate the SNC.

4.2.1 Method 1: Calculation using FWD data

The calculation of SNC using FWD data involves using the back-analysed FWD resilient modulus of the pavement layers (Tonkin & Taylor 1998) and inferring the subgrade CBR from the FWD resilient modulus of the subgrade. These values were used in Equation 4.1 together with the thickness of each layer.

For some sites the back-analyses of the FWD testing produced very high resilient modulus values (>1000 MPa). In order to avoid unrealistically high values of basecourse modulus, a maximum value of 750 MPa was used, i.e. if the calculated modulus was greater than 750 MPa, then 750 MPa was used in subsequent calculations.

4.2.2 Method 2: Calculation using in-situ data

The calculation of SNC from in-situ data used the layer thickness, the in-situ CBR of the subgrade, and the layer coefficients a_g from Table 4.1. This would be the method that practitioners would use if they did not have access to layer modulus values.

4.2.3 Method 3: Calculation using in-situ SNC and FWD SNC data

As the subgrade contribution to the SNC is based on the subgrade CBR, the third method used the measured in-situ subgrade CBR with the FWD granular modulus values. This method was advocated by Patrick (2001) as it is not based on assumptions of the relationship between modulus and CBR, and is appropriate for use on subgrades of volcanic origin.

4.3 Results

The comparison between the first two SNC calculation methods for the selected pavement sections are shown in Figure 4.1. (Figures 4.1, 4.2 and 4.3 each show a line of equality that has been drawn to help visualise the correlations. Plots falling on the line would indicate an exact relationship between the readings.)

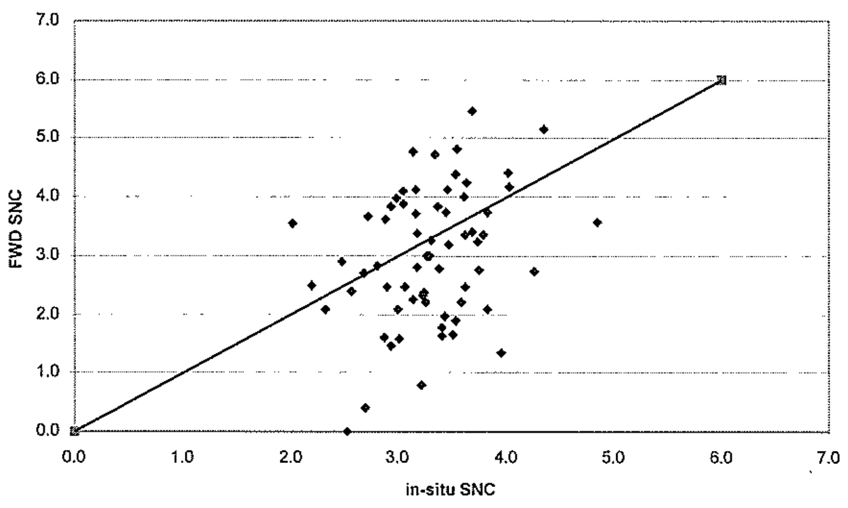

Figure 4.1 Comparison of SNCs calculated using FWD SNC (Method 1) and in-situ data (Method 2).

The correlation of SNCs obtained from Method 3 and the other two methods can be seen in Figures 4.2 and 4.3.

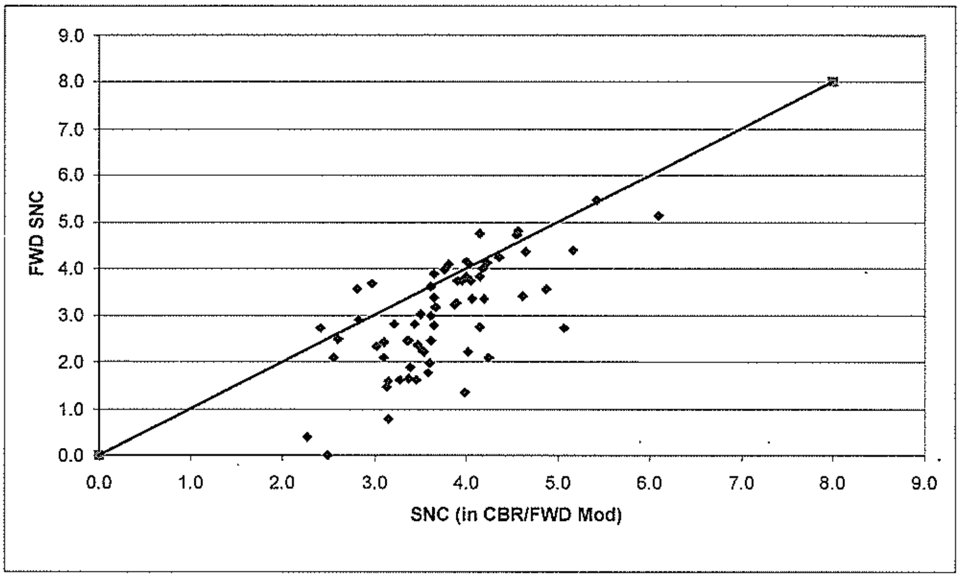

Figure 4.2 Comparison of SNCs calculated using FWD data (Method 1) and CBR/FWD moduli (Method 3).

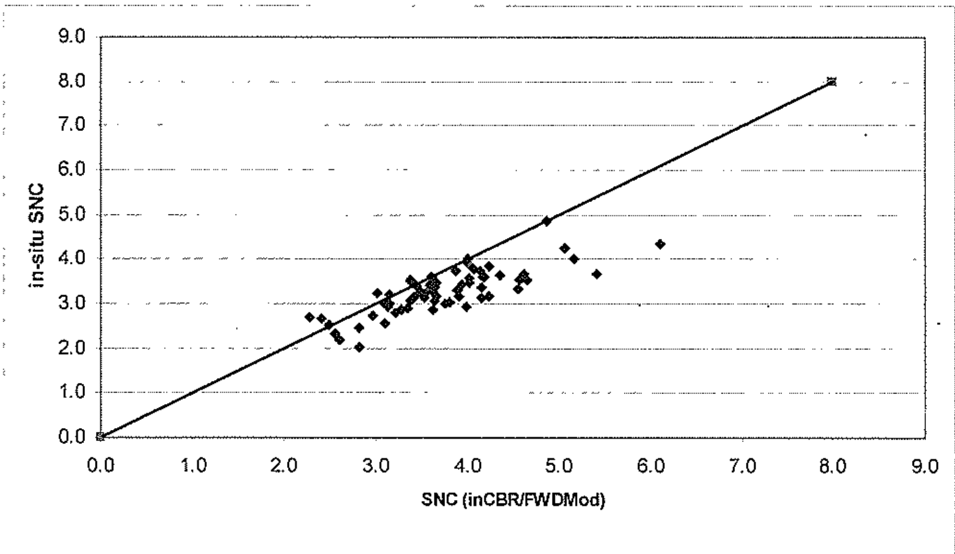

Figure 4.3 Comparison of SNCs calculated using in-situ SNC (Method 2) and CBR/FWD moduli (Method 3).

The main difference between the three SNC calculation methods is in the subgrade contribution. The very low SNC values derived from FWD analysis (Method 1) are associated with inferred subgrade CBR values in the 1 to 2 range, and therefore many of the data points in Figure 4.2 lie below the line of equality. The lowest measured in-situ CBR was 4. The difference in CBR from 1 to 4 changes the SNC by approximately 1.7.

As Method 3 is considered the most appropriate method for use in New Zealand, it was used for the calculations given in the following chapters.

5. Mechanistic analysis of sites

5.1 Introduction

The mechanistic analysis was carried out using the computer program CIRCLY (Wardle 1977). CIRCLY is an elastic layer analysis program that can calculate the stresses and strains within the pavement caused by a specified loading (in ESAs). The program can also be used to calculate the expected loading based on the subgrade fatigue criteria given in the Austroads Pavement Design Guide (1992, and in the NZ Supplement 1995).

The analysis enabled the expected life of each site to be determined using the Austroads procedure, which was then compared to the actual life, in ESAs (calculated using the procedure outlined in Section 3.2.3).

5.2 Data input

Data relevant to the analysis was extracted from the collated database to form a worksheet with all the data required to complete the CIRCLY analysis. This comprised:

- information on the pavement layers;
- their thickness and properties;
- the subgrade strength; and
- the total traffic that the pavement had experienced since construction.

For the analysis to be comparable with the HDM III analysis, the in-situ CBR and the moduli of the pavement layers obtained from the FWD testing (see Section 4.2.1) were used in the analysis.

CIRCLY determines the critical layer within the pavement based on Austroads failure criteria for the asphalt and/or the subgrade layers. As all the selected pavement surfacings were chipseals, the pavements were modelled without a surfacing layer and therefore the critical strain was always obtained from the subgrade strain criterion.

5.3 Data analysis

For each site, a CIRCLY analysis was run and the results were added to the worksheet along with the pavement data. The expected life in terms of ESAs was derived.

5.4 Results

The CIRCLY-predicted pavement life in ESAs was compared to the actual ESAs which the pavement had experienced before rehabilitation. The results are plotted on Figure 5.1.

Figure 5.1 shows a wide scatter of data points indicating that the predicted life calculated from CIRCLY and the actual pavement life differ significantly for most sites.

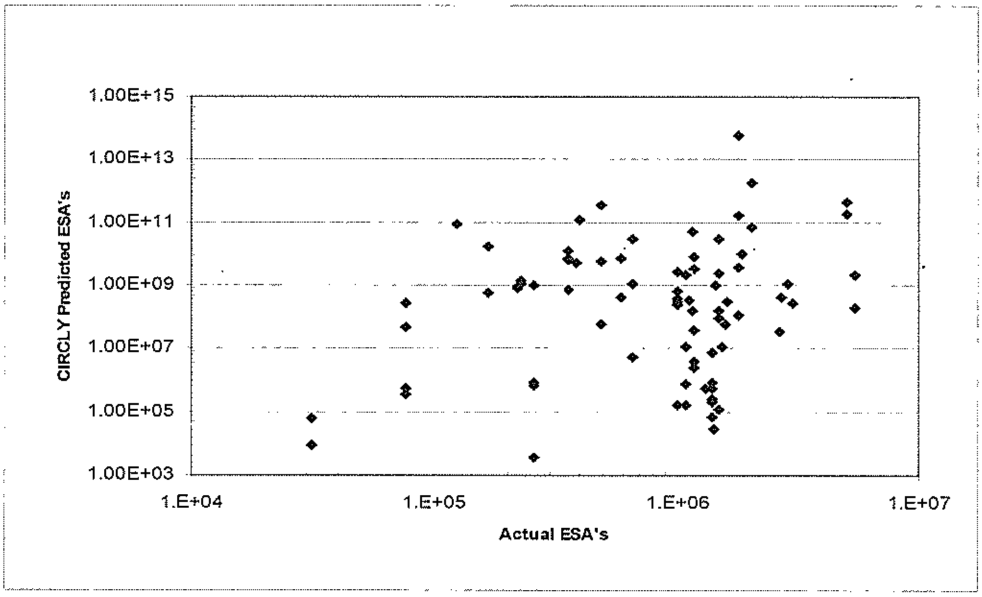

Figure 5.1 Comparison of CIRCLY-calculated traffic versus actual design traffic (in ESAs).

6. HDM III analysis of sites

6.1 Introduction

The selected pavement sites were also analysed using the HDM III pavement deterioration model. The calculations were performed using the approach used to validate the HDM III roughness progression model that is detailed in Cenek & Patrick (1991). This method does not rely on knowledge of the roughness level at the time of construction.

The SNCs determined in Chapter 4 were used in the analysis so that the predicted roughness at the rehabilitation date could be determined. This predicted roughness (in NAASRA counts) obtained from the HDM model was then compared to the actual roughness measured before rehabilitation.

6.2 Data input

As with the mechanistic analysis, the data used for the HDM III modelling was extracted from the collated database to form a single worksheet and included all the relevant data for the analysis. Information was obtained that enabled calculation of SNC: for example the HDM model incorporates the age of the pavement into the prediction, and therefore the total age from construction to rehabilitation was included together with the age from the first roughness reading to the final reading at rehabilitation.

The HDM analysis used the cumulative traffic in ESAs/lane/day[4] between the dates of the first roughness reading and roughness at the first available reading (i.e. 1992) to predict the roughness at end-of-life of each pavement section.

6.3 Data analysis

First, the SNC for each site was calculated using Equation 4.1 (Section 4.2). The HDM III model was then used to calculate the roughness at the rehabilitation date. Equation 6.1 which is used (from Cenek & Patrick 1991) is based on the HDM III model as defined here:

$$RN(Y2) = (RN(Y1)e^{(0.0153(t2-t1))}) + (5.7\ (1+SNC)^{(-4.99)}\ ESA\ (t_2-t_1))\ e^{(0.0153t2)} \qquad \text{Equation 6.1}$$

where:

RN(Y2)	= roughness data at period 2 (rehabilitation date in this case)
RN(Y1)	= earliest available roughness data (1992 in this case)
t_1	= Y1-Y0
t_2	= Y2-Y0
(t_2-t_1)	= Y2-Y1

[4] The traffic is in ESA/lane/day because the HDM model includes the days of the year within the equation.

Y2 = year of predicted roughness (rehabilitation date in this case)
Y1 = year of earliest available roughness data (1992 in this case)
Y0 = year of construction or overlay

The results were added to the spreadsheet to enable easy comparison and analysis.

6.4 Results

The predicted roughness at the rehabilitation date was calculated for all sites and then compared to the actual measured roughness.

The results obtained from the analysis are shown in Figure 6.1. The R^2 for the predicted versus actual roughness was calculated as 0.58. Figure 6.1 shows that the HDM-predicted roughness compares reasonably well to the actual roughness. As for the SNC graphs (Figures 4.1-4.3), a line of equality has been drawn to enable the comparison to be visualised more easily. From observation an approximately equal number of data points appear below and above the equality line.

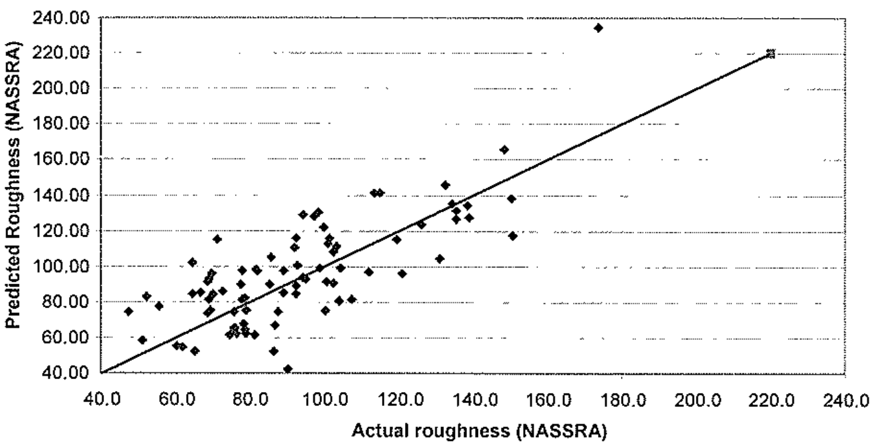

Figure 6.1 Comparison of actual measured roughness versus calculated HDM model roughness (in NAASRA counts).

7. Comparison of HDM III and mechanistic analyses

7.1 Site conditions

The predictions made by the HDM III analysis and the mechanistic analysis results were then compared and analysed.

According to Transit New Zealand guidelines, a pavement has reached the end of its life when the NAASRA counts recorded are between 100 and 150, depending on the classification of the road, e.g. 100 for motorways/expressways (>1000 vpd) or 150 for urban roads and others with <1,000 vpd. Many of the pavement sites used in the analysis were found to have a roughness lower than the Transit limit values.

The distribution of roughness before treatment is shown in Figure 7.1.

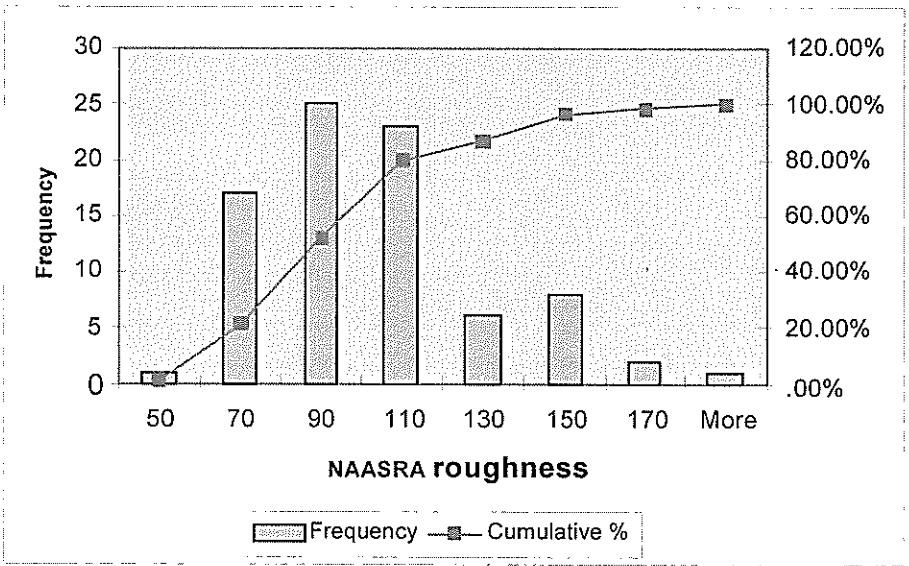

Figure 7.1 Distribution of final roughness values, by frequency (left axis) and cumulative % (right axis), for the study sites.

7.1.1 Effects on roughness

The frequency distribution of the roughness level before treatment of over 1000 sites on the New Zealand State Highway system is given in Figure 7.2. This figure illustrates that most treatments are being programmed where the roughness level is under 120 NAASRA counts, and a significant proportion of them are less than 90 NAASRA.

Figure 7.2 appears similar to Figure 7.1 which is based on the 88 sample pavements that we selected for the project. Therefore the sample sites can be assumed to be representative of the total State Highway network.

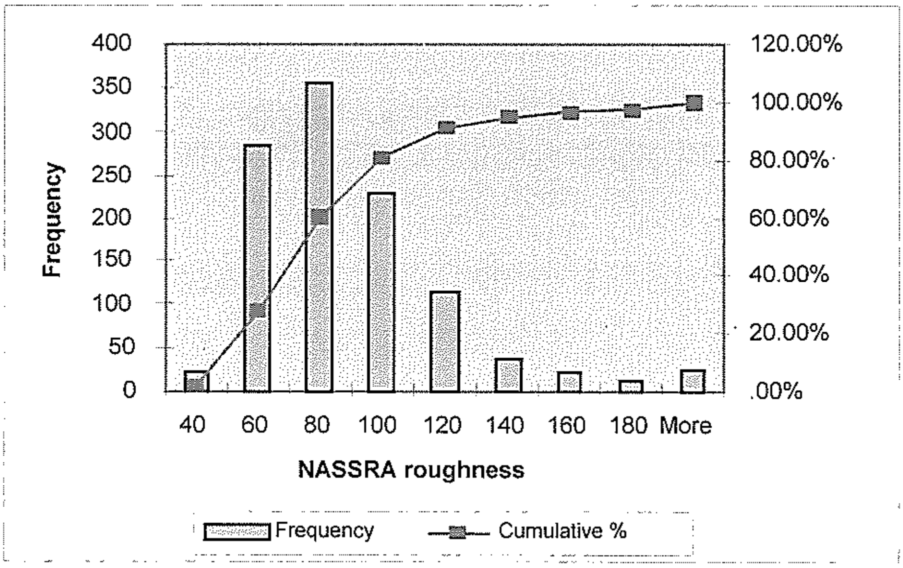

Figure 7.2 Distribution of roughness, by frequency (left axis) and cumulative % (right axis), before treatment of over 1000 sites on State Highways throughout New Zealand.

7.1.2 Effects on rehabilitation

Figure 7.1 shows that over 50% of the sites are below 100 NAASRA. This means the pavement sections used in this study had been rehabilitated before the end of their predicted life, according to Austroads and HDM deterioration modelling.

In the mechanistic analysis, CIRCLY calculates that many of the sites were rehabilitated before their full life had been achieved. Figure 7.3 compares the SNC with the CIRCLY-calculated expected life (in ESAs). The figure also presents the actual ESAs which the pavement has experienced over the same time period.

Figure 7.3 shows too that a correlation exists between the CIRCLY-predicted life and the SNC, which is to be expected as the CIRCLY analysis is based on the same soil parameters as the SNC, i.e. subgrade strength and pavement layers. However, it also shows that the actual ESAs tend to reach a limit of approximately 10^7 ESAs (or 1.0E+07 on Figure 7.3).

It appears from Figure 7.3 that, according to the Austroads-predicted ESAs, the pavement sites and sections used in the study were rehabilitated before the end of their predicted life and that they still had life remaining.

A reason for the limit to pavement life may be that, for a chipseal surface, the maximum number of ESAs is 10^7, after which a more structural type of surfacing needs to be considered.

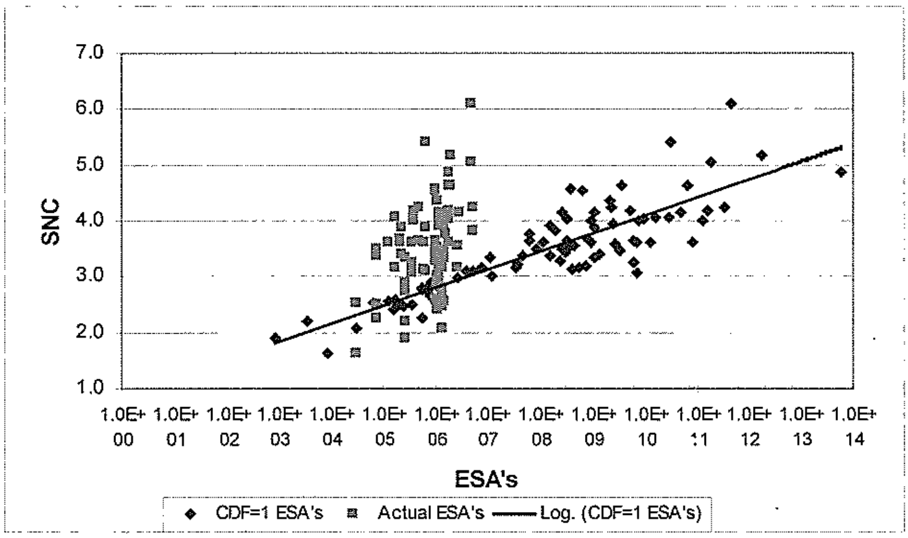

Figure 7.3 Comparison of actual and predicted ESAs with SNCs using CIRCLY-derived figures.

The main reason given for the sites being treated was the cost of maintenance. The economic analysis performed by the consultant had demonstrated that the net present value of the future maintenance costs was greater than the cost of rehabilitation and this was used as justification for early treatment.

Much of the information obtained on the pavement sites was found from Area-wide Treatments (AWTs), where the cost of maintenance is the driver for rehabilitating a site.

A number of sites on Southland roads had problems of potholes and heaving, and therefore had the potential to incur increasing maintenance costs if not rehabilitated. The pavements in the selected sites seemed to be failing even though the subgrade strain was less than that predicted at failure by Austroads criteria and the roughness was less than the Transit limits. The pavements had however an average age of close to 50 years, with very few under 30 years, as shown in Figure 7.4.

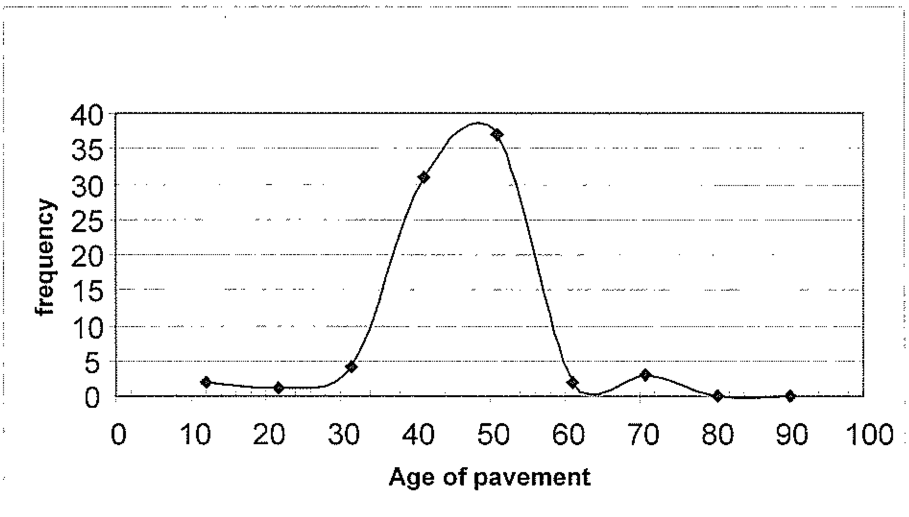

Figure 7.4 Distribution of ages of rehabilitated pavement sites.

7.2 Predictions of pavement performance using HDM III

The HDM III roughness equation can be modified to include a traffic growth component (which was used in this analysis) and may be used to construct a roughness time graph. Figure 7.5 illustrates the predicted roughness trends for two pavements, one with an SNC of 2.5 (i.e. higher strength), the other with 1.5, both having a traffic loading of 10^5 ESA over 50 years with a traffic growth of 2% per annum. It can be seen that the weaker pavement with the SNC of 1.5 has a predicted roughness of 105 NAASRA counts, while the stronger is only 69 NAASRA counts, at the end of 50 years. Figure 7.3 shows that most of the sites analysed had a SNC of 2.5 which is consistent with the strength that Austroads Pavement Design Guide would recommend for this level of traffic loading. It is thus understandable that the roughness counts obtained on the pavement sections were relatively low.

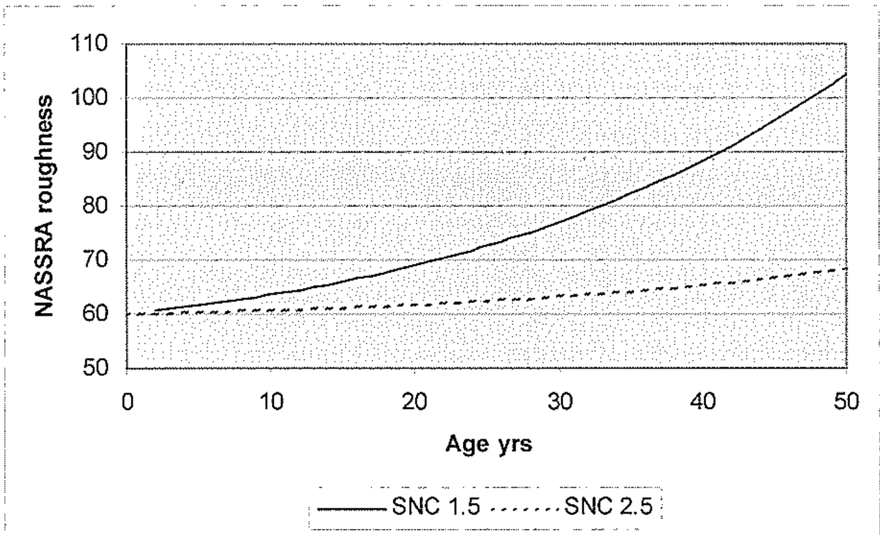

Figure 7.5 HDM-predicted roughness trend for two pavements subjected to a loading of 10^5 ESA over 50 years.

To more directly compare the predictions of the HDM III calculation with that of the Austroads Pavement Design Guide, the predicted performances of a range of pseudo-pavements were calculated.

Figure 8.4 of the Austroads Guide presents a range of granular layer thicknesses required under a range of traffic and subgrade CBR conditions. The SNC for a range of pavements was calculated based on an assumed basecourse modulus of 500 MPa and a sub-base modulus of 250 MPa.

The HDM relationship was then used to calculate the expected roughness after a 25-year design life based on an initial roughness of 60 NAASRA counts. The results are given in Table 7.1.

Table 7.1 SNC and terminal roughness for a range of granular pavements with a 25-year design life and initial roughness of 60 NNASRA counts.

Design ESA	Subgrade CBR	Total Granular Thickness (mm)	SNC	Terminal Roughness (NAASRA)
10^5	10	200	2.6	97
	5	300	2.6	92
10^6	10	260	3.0	116
	5	390	3.1	111
10^7	10	320	3.3	272
	5	490	3.7	210

The above analysis is consistent with the field data in that, over the range of 10^5 to 10^6 ESA, the change in roughness is relatively small over the life of the pavement. However for a loading of 10^7 ESA the HDM model predicts a roughness of over 250 NAASRA counts, even with the greater thickness of pavement.

Therefore HDM III would predict that, at low traffic volumes (10^5 to 10^6 vpd), pavements are over-designed and, at high traffic volumes (>10^7 vpd), they are under-designed in terms of total layer thickness.

8. Summary and conclusions

This research has shown that:

- Most shape correction and reconstruction projects on New Zealand pavements are driven by factors other than structural deterioration.
- Most of these pavements have significant remaining life when analysed using the Austroads Pavement Design Guide criteria.
- On the relatively lightly trafficked granular pavements (i.e. with granular thin surfaces) of New Zealand roads, a life in excess of 50 years is common.
- The roughness levels of these pavements are in the order of that predicted by the HDM III roughness model.
- The rut depths on all pavements measured were, in general, less than 10 mm and therefore not a reason for rehabilitation.
- For granular pavements designed to the Austroads Pavement Design Guide, the HDM III models predict that the roughness at the end of the design life will be less than 100 NAASRA counts when the design traffic is in the range of 10^5 to 10^6 ESA, but over 200 NAASRA counts for design traffic of 10^7 ESA.
- The use of HDM III roughness progression models is a powerful tool to supplement mechanistic design, especially in determining the effect of changing variables such as initial roughness.
- In terms of roughness there is a significant difference in the expected terminal condition of a pavement designed in terms of the Austroads Pavement Design Guide criteria and the condition determined from an HDM III roughness progression.

9. Recommendations

Recommendations obtained from the project are that:

- Further investigations are made of the reasons for rehabilitating pavements, and to determine if the increasing maintenance expenditure that may occur on a road section is associated with a surfacing failure or a structural failure of the pavement.
- The differences between the HDM-predicted change in shape and the terminal condition associated with the design criteria in the Austroads Pavement Design Guide should be examined to harmonise the two approaches.
- The approach used in this project should be incorporated in the New Zealand Supplement to Austroads Pavement Design Guide, to encourage pavement designers to explore the sensitivity and risk of their pavement designs to changes in pavement shape with time.
- An investigation should be carried out of the factors affecting maintenance costs and the assumptions used to generate the Net Present Value of future maintenance costs.

10. References

Austroads. 1992. *Pavement Design: A guide to the structural design of road pavements.* Austroads, Sydney. Updated in 2004.

Barksdale, R.D. 1971. Compressive stress pulse times in flexible pavements for use in dynamic testing. *Highway Research Record 345:* 32-44.

Cenek, P., Patrick, J. 1991. Prediction of road roughness progression. *Central Laboratories Report 91-29301.*

Hodges, J.W., Rolt, J., Jones, T.E. 1975. The Kenya Road Transport Cost Study: research on road deterioration. *TRRL Laboratory Report 673.* Transport and Road Research Laboratory, Crowthorne, England.

Patrick. J. 2001. Methods for determining structural number of New Zealand pavements. *Transfund New Zealand Research Report No. 199.* 37pp.

Tonkin & Taylor Ltd. 1998. Pavement deflection measurement and interpretation for the design and rehabilitation treatments. *Transfund New Zealand Research Report No. 117.* 70pp.

Transfund New Zealand. 1997. *Project evaluation manual* (PEM). Transfund NZ, Wellington, New Zealand (latest amendment October 2004).

Transit New Zealand. 1995. Supplement to *Pavement Design: A guide to the structural design of road pavements*. Austroads, Sydney.

Transit New Zealand. 2000. Performance-based specification for structural design and construction of flexible unbound pavements. *TNZ B/3 (provisional) specification.*

Appendix Sites selected for comparing HDM III and SNC calculations, from Southland and Hawke's Bay Transit NZ regions

Site No.	Name	Position	In-situ CBR	Date of Construction
1	SH2 RP691/14.38-14.70 NTH OTANE 7/99	14.44	28	1955
2	SH2 RP691/14.38-14.70 NTH OTANE 7/99	14.551	28	1955
3	SH2 RP544/0.07-0.20 7/99	0.091	8	1959
4	SH2 RP544/0.07-0.20 7/99	0.141	18	1959
5	SH2 RP544/0.07-0.20 7/99	0.181	4	1959
6	SH2 RP608/9.34-10.84 7/99	9.525	18	1951
7	SH2 RP608/9.34-10.84 7/99	9.85	50	1951
8	SH2 RP608/9.34-10.84 7/99	10	44	1951
9	SH2 RP608/9.34-10.84 7/99	10.84	18	1951
10	AWAHOHONU FOREST 2.480-2.600 7/99	2.585	61	1960
11	SH2 RP562/12.670-14.000 7/99	12.725	28	1953
12	SH2 RP562/12.670-14.000 7/99	12.9	18	1953
13	SH2 RP562/12.670-14.000 7/99	13.225	28	1953
14	SH2 RP562/12.670-14.000 7/99	13.65	8	1953
15	SH2 RP562/12.670-14.000 7/99	13.9	6	1953
16	SH50 RP49/1.56-1.84 MANGANONUKU 7/99	1.751	4	1937
17	OHINEPAKA 10.180-10.500 7/99	10.24	13	1957
18	OHINEPAKA 10.180-10.500 7/99	10.311	8	1957
19	OHINEPAKA 10.180-10.500 7/99	10.431	13	1957
20	OTANE STRAIGHT 1.500-1.990 7/99	1.56	13	1955
21	OTANE STRAIGHT 1.500-1.990 7/99	1.71	15	1955
22	SH35 RP180/7.350-7.520 7/99	7.39	23	1967
23	SH35 RP180/7.350-7.520 7/99	7.5	18	1967
24	SH35 RP225/4.56-5.12 PUKETITI 7/99	2.971	6	1956
25	SH2 RP721/7.490-7.850 7/99	7.805	11	1983
26	NORTH SNEE RD 4.520-4.750 7/99	4.581	18	1955
27	SH2 RP497/10.990-12.710 7/99	11.125	28	1960
28	SH2 RP497/10.990-12.710 7/99	11.425	13	1960
29	SH2 RP497/10.990-12.710 7/99	11.725	18	1960
30	SH2 RP497/10.990-12.710 7/99	12	23	1960
31	SH2 RP497/10.990-12.710 7/99	12.35	20	1960
32	SH2 RP562/1.540-2.300 TE KAHU 7/99	1.625	23	1958
33	SH2 RP562/1.540-2.300 TE KAHU 7/99	1.875	13	1958
34	SH2 RP562/1.540-2.300 TE KAHU 7/99	2.25	18	1958
35	SH35 RP159/7.2-7.47 WHAKAANGIANG 6/00	7.218	31	1957
36	SH35 RP159/7.2-7.47 WHAKAANGIANG 6/00	7.397	28	1957
37	SH2 RP497/7.34-7.4 BLUCKS 6/00	7.364	10	1960
38	SH2 RP443/16.3-16.5 CALCOTTS 6/00	16.389	18	1938
39	SH2 RP443/13.63-13.9 CALCOTTS BR 6/00	13.837	21	1938

Site No.	Name	Position	In-situ CBR	Date of Construction
40	SH38 RP189/1.95-2.15 CLARKES 6/00	2.041	18	Unknown
41	SH38 RP189/1.95-2.15 CLARKES 6/00	2.106	33	Unknown
42	SH2 RP608/8.215-8.7 DEVILS ELBOW 6/00	8.557	28	1951
43	SH50 RP17/0.4-0.82 FERNHILL 6/00	0.484	28	1948
44	SH35 RP172/6.09-6.28 FORDS 6/00	6.194	50	1969
45	SH2 RP516/2.21-2.57 GUTHRIES 6/00	2.45	11	1960
46	SH2 RP675/11.66-11.82 HORONUI 6/00	11.82	8	1955
47	SH2RP RP443/9.82-10.24 KARAUA 6/00	10.109	34	Unknown
48	SH35 RP213/1.92-3.93 KOPUAROA 6/00	2.075	44	1901
49	SH35 RP213/1.92-3.93 KOPUAROA 6/00	2.75	13	1901
50	SH35 RP213/1.92-3.93 KOPUAROA 6/00	3.9	28	1901
51	SH2 RP608/15.83-17.71 KAREAARA 6/00	16.575	39	1947
52	SH35 RP274/5.25-5.7 LOISELS 6/00	5.37	39	1966
53	SH35 RP274/5.25-5.7 LOISELS 6/00	5.58	44	1966
54	SH2 RP497/5.03-5.12 MAHIA 6/00	5.078	4	1960
55	SH35 RP200/4.49-4.68 NGARIMU S 6/00	4.605	23	1959
56	SH35 RP200/11.5-11.9 PAHI 6/00	11.77	109	1959
57	SH35 RP180/7.35-7.54 PEPERES H 6/00	7.389	23	1967
58	SH35 RP180/7.35-7.54 PEPERES H 6/00	7.506	28	1967
59	SH35 RP225/3.13-3.23 PUKETITI 6/00	3.186	39	1956
60	SH35 RP225/4.04-4.39 PUKETITI 6/00	4.086	109	1956
61	SH35 RP225/4.04-4.39 PUKETITI 6/00	4.264	13	1956
62	SH2 RP729/8.45-8.56 STH FRASER 6/00	8.507	13	1955
63	SH35 RP132/7.03-7.23 TRAFFORDS 6/00	7.102	13	1972
64	SH35 RP132/7.03-7.23 TRAFFORDS 6/00	7.167	13	1972
65	SH2 RP544/9.1-9.85 WAIHUIA 6/00	9.35	13	1957
66	SH2 RP544/9.1-9.85 WAIHUIA 6/00	9.75	39	1957
67	SH35 RP132/3.1-3.37 WHAAKI 6/00	3.199	10	1972
68	SH35 RP132/3.1-3.37 WHAAKI 6/00	3.343	20	1972
69	SH2 RP533/6.6-7.18 AWAMATE 6/00	7.047	13	1957
70	SH2 RP361/12.06-12.30 BROWNS 11/00	12.22	8	Unknown
71	SH96 RP0/11.450-12.150 East Waitane	11.49	5	1960
72	SH96 RP0/11.450-12.150 East Waitane	11.801	31	1960
73	SH96 RP0/11.450-12.150 East Waitane	11.95	7	1960
74	SH96 RP0/11.450-12.150 East Waitane	11.99	6	1960
75	SH96 RP0/11.450-12.150 East Waitane	12.121	5	1960
76	SH6 RP1145/9.310-10.740 McKenzie 1	9.499	8	1961
77	SH6 RP1145/9.310-10.740 McKenzie 1	9.599	6	1961
78	SH6 RP1145/9.310-10.740 McKenzie 1	9.75	12	1961
79	SH6 RP1145/9.310-10.740 McKenzie 1	9.95	6	1961
80	SH6 RP1145/9.310-10.740 McKenzie 1	10.199	4	1961
81	SH6 RP1145/9.310-10.740 McKenzie 1	10.33	5	1961
82	SH6 RP1145/9.310-10.740 McKenzie 1	10.699	8	1961
83	SH93 RP0/14.440-14.690 Range Rd 2	14.44	6	1989

Site No.	Name	Position	In-situ CBR	Date of Construction
84	SH93 RP0/14.440-14.690 Range Rd 2	14.568	10	1989
85	SH96 RP66/8.100-8.700 West Aparima	8.131	50	1958
86	SH96 RP66/8.100-8.700 West Aparima	8.34	50	1958
87	SH96 RP66/8.100-8.700 West Aparima	8.54	50	1958
88	SH96 RP66/8.100-8.700 West Aparima	8.66	50	1958