A review of the HDM/dTIMS pavement models based on calibration site data

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An important note for the reader

Acknowledgment

This project has been co-funded by Land Transport New Zealand and Transit New Zealand (Transit). In addition to direct financial contribution Transit has also provided the data for model calibration and model development work.

Contents

Executive summary

The aim of this research, carried out in 2005, was primarily to test the current pavement deterioration models adopted in the New Zealand system. Most of these models were adopted from HDM-III and HDM-4, but some locally developed models were also tested. The model calibrations performed could be divided into the following categories:

- Calibration Level 2 – adjustment of the calibration coefficient. For this calibration level, all other model coefficients were kept at the default level.
- Calibration Level 3 (model adjustment) – regression to obtain the best model coefficients for New Zealand conditions. The original model format is kept and the values of all the model coefficients are adjusted in order to minimise the error between the predicted and the observed data points.
- Calibration Level 3 (new model format) – data-driven models are developed from first principles based on the Long Term Pavement Performance (LTPP) data.

In all cases, care was taken to ensure that sufficient and appropriate data were used during the analysis. For example, given the relatively young age of the LTPP programme, more data had to be sourced in order to perform the calibration of the cracking model. It should also be noted that only the priority performance prediction model was calibrated during this study. There are also other models, such as the potholing model, which have not been addressed in this report, but which have been included in the overall model developments for New Zealand. A summary of the model calibration results is provided in the following sections.

Crack Initiation

In agreement with engineering judgement, this report again emphasised the importance of crack initiation. It signals a significant turning point in the behaviour of the pavement, and is often the starting point of accelerated deterioration. It has also been confirmed that a cracked pavement will have shorter expected life even if it is resurfaced. The crack initiation calibration included all the calibration steps and the results are presented in Table ES1.

Table ES1 Summary of the crack initiation calibration results.

<table>
<tr><th>Calibration Level / Method</th><th>Results</th><th>Error / Accuracy</th></tr>
<tr><td>Level 2 – Adjusting Calibration Coefficients</td><td>
<table>
<tr><th></th><th colspan="2">Regional Classification</th></tr>
<tr><th></th><th>High and Moderate</th><th>Low and Limited</th></tr>
<tr><td>Kci</td><td>0.49 (0.52*)</td><td>0.59 (0.64*)</td></tr>
<tr><td>Error – Default</td><td>194</td><td>477</td></tr>
<tr><td>Error – Calibrated</td><td>27</td><td>160</td></tr>
</table>
Note: Calibration was performed on LTPP data. Values in brackets resulted from calibration performed on regional data
</td><td></td></tr>
<tr><td>Level 3 – Adjusting Model Coefficients</td><td>
Default HDM-4 Model Coefficients
<table>
<tr><th>a0</th><th>a1</th><th>a2</th><th>a3</th><th>a4</th></tr>
<tr><td>13.2</td><td>0</td><td>20.7</td><td>20</td><td>0.22</td></tr>
</table>
Adjusted HDM-4 Model Coefficients for Chip Seals
<table>
<tr><th>a0</th><th>a1</th><th>a2</th><th>a3</th><th>a4</th></tr>
<tr><td>8.3</td><td>0</td><td>18.54</td><td>0.01</td><td>0.34</td></tr>
</table>
</td><td>17597.7

6697.9</td></tr>
<tr><td>Level 3 – New Model Format (Linear Model)</td><td>
For PCA = 0 (sections not cracked before resurface)

$$ICA = Kci * \exp\begin{bmatrix} 5.7 - 1.25\log(HTOT) - 0.3\log(AADT) + \\ 0.08\log(HTOT) * \log(AADT) \end{bmatrix}$$

For PCA > 0 (sections were cracked before resurface)

$$ICA = Kci * \exp\begin{bmatrix} 4.6 - 0.68\log(HTOT) - 0.47\log(AADT) \\ + 0.08 * \log(HTOT) * \log(AADT) \end{bmatrix}$$

where:
ICA is the crack initiation time in years after the surface is constructed
HTOT is the total surface thickness (in mm) of all the layers
AADT is the annual average daily traffic
</td><td>5157</td></tr>
<tr><td>Level 3 – Generalised Model</td><td>

$$p(stat.aca) = 1 \Big/ \left[1 + \exp\begin{pmatrix} -0.141AGE2 + \{(5.062, 3.440)\ \mathit{for}\ stat.pca = (0,1)\} \\ -0.455Log(AADT) - 0.275Log(HTOT) + 0.655SNP \end{pmatrix} \right]$$

where:
p(stat.aca) is the probability of a section being cracked
AGE2 is the surface age in years, since construction
stat..PCA is the cracked status before resurfacing (0 or 1 for not cracked or cracked)
HTOT is the total surface thickness (in mm) of all the layers
AADT annual number of equivalent standard axles (millions/lane)
SNP is the modified structural number
</td><td></td></tr>
</table>

Both the linear model and the generalised model deviate substantially from the original HDM model. The main reason for the significant difference is that the latter two models are purely data-driven. They suffer one big disadvantage in the sense that they can only be adopted within the environment for which they were developed. It is thus recommended that all the model formats are tested on any network where they are applied. However, this would not be an onerous task and it could be easily performed for all state highways.

Given the advantages of the logistic model as presented in the report, it is recommended that this model format is adopted for New Zealand roads. Part of this adoption will include testing the model for other regions and formulating the practical inclusion of the model in the decision process of the pavement management system.

Texture

In a previous study by Transit in 2003 a simplified texture model was developed as an alternative to the HDM texture model. That report recommended a different approach for determining model coefficients depending on the availability of data. This study has confirmed the model format to be appropriate, but has recommended that different coefficients should be developed for each chip size, but not for individual regions. The recommended model coefficients are presented in Table ES2.

Table ES2 Recommended model coefficients

Type	a_0	Std. Error	a_1	Std. Error
AC	1.024727	0.164577	−0.01889	0.009675
CHIP.G2	4.314283	0.101516	−0.12662	0.006757
CHIP.G3	4.158183	0.071979	−0.13461	0.004663
CHIP.G4	3.407922	0.177842	−0.10949	0.011158
CHIP.G5	3.479998	0.120092	−0.12425	0.007992
CHIP.G6	2.319671	0.242437	−0.07236	0.015257
OG	1.182618	0.142981	0.005555	0.00809

The model format and coefficients were tested on the calibration SLP data and a reasonably good fit was established.

The base texture model seems to be yielding satisfactory results. However, more work needs to be completed in the area of the rapid deterioration phase of texture loss (flushing model). With this model, more understanding needs to be developed on the interaction between variables such as traffic, and the surface characteristics such as total thickness of the surface.

Rutting

The rut depth progression has been calibrated according to both the HDM-III and HDM-4 model. The resulting coefficient from these analyses are presented in Table ES3.

Table ES3 Rut depth progression calibration results.

Sensitivity Risk Area[1]	HDM-III		HDM-4	
	Calibration Coefficient (Krp)	Error Function (RMSE)[2]	Calibration Coefficient (Krp)	Error Function (RMSE)[2]
Low and Limited	0.87	23,333 (30,746)	1.03	2,719 (2,729)
Medium and High	0.81	729 (1,346)	0.98	931 (933)
All Data	0.84	22,583 (32,092)	1.01	3,658 (3662)

Note: [1] The data was not sufficient (i.e. nor enough data points) to perform successful calibration on individual sensitivity risk areas

[2] The values in brackets indicate the error function result using the default calibration coefficient (Krp=1). RMSE = root mean square error

An attempt was made to improve the rutting model format, but the model could not be improved. However, it was observed that for most of the pavement life a fairly constant rate of rutting progression occurs (this equates to approximately 0.3 mm per year based on the LTPP data). The recommended work for the rut progression model includes:

- Investigate rut progression model forms on the CAPTIF data. Two major trends must be investigated:
  - Determine the factors contributing towards stable rut progression observed during most of the pavement life. In particular see if the CAPTIF data conforms to the constant 0.3 mm rut progression of pavements.
  - Determine the factors contributing towards a pavement that starts accelerated deterioration that includes a rapid rut progression.
- Further analysis into tracking rutting on individual sections over time. As the LTPP data become available for longer time periods, this analysis would become more useful. By combining trends from the LTPP data and the CAPTIF data, it may also be possible to develop model formats from basic principles.

Roughness

The roughness model calibration could not be performed based on the current available data. The data did, however, confirm that the HDM-4 model format does not reflect the actual behaviour of most pavements in New Zealand. The model suggests a relatively long period of the pavement life during which the deterioration is slow. Then it reaches a stage of rapid deterioration. The aim of the future model development would be to predict the timing when the rapid deterioration commences.

Insufficient LTPP data existed to perform the desired model development. Only a limited number of sections have reached the rapid deterioration stage. It is recommended that some CAPTIF data be applied to develop the model format. In addition to this, extended LTPP data can then be used to confirm the model behaviour on the state highways as it becomes available. The recommended work includes:

- Investigate the roughness reduction trend after construction. The aim is to determine how much of this reduction is actual smoothing of the pavement versus the perceived roughness reduction caused by measurement technique.
- Confirm the roughness trend during the gradual deterioration phase with more LTPP and CAPTIF data.
- Determine the factors contributing towards the initiation of the rapid deterioration phase of roughness progression.

A new relationship for the rut depth standard deviation is proposed:

$$\mathrm{rds} = \exp\left[0.8804\log(\mathrm{RDM}) - 0.9369\right]$$

However, this model will suffer the same limitation as the previous model since it is a function of the mean rut depth (RDM). The proposal is to develop a new model format from first principles which will not be reliant on the mean rutting.

Recommended further work

The future of the New Zealand LTPP sections

This report has demonstrated that the level of data collection accuracy is appropriate for calibration and pavement model development. Some intuitive trends with some models (such as roughness) have been confirmed for the first time since the appropriate level of data accuracy existed. However, this study has also highlighted the need for further model development based on individual section data. It has been suggested that some of the models such as roughness and rutting could be developed utilising some CAPTIF data, but ultimately could only be confirmed with the LTPP data.

Furthermore, the report has highlighted the significance of the rapid failure stages of pavement deterioration modelling. In order to understand the behaviour during rapid deterioration better, more data during this stage is needed.

It is therefore recommended that the LTPP surveys continue on both the Transit and local authority networks. It is difficult to predict the time required, but it is estimated that the surveys should continue for at least another five years.

In terms of the current data collection precision and accuracy requirements, it is recommended that the current standard be maintained.

Investigate the rapid failure stage for roughness and rutting progression

As a next stage to this research, a proposal was accepted for the 2005/06 Land Transport New Zealand Research Programme. This study is aimed at linking the LTPP and the CAPTIF programmes. The objectives of this research are explained in the extract from the research proposal:

> *Linking the outputs from CAPTIF with the LTPP study is the next logical step towards building on the understanding of pavement performance/ deterioration under New Zealand conditions. Comparing field performance*

(LTPP) with the accelerated load performance (CAPTIF) will significantly increase the confidence in outputs from both these programs. The specific objectives of the study include:

1. *Calibration coefficients and new model formats have been developed based on the Transit LTPP data (Transfund Research Programme 04/05). The first objective would be to confirm and improve the model format and results based on existing CAPTIF data.*
2. *To develop relative performance factors for different treatments and material types, similar to the work completed in Australia (Martin 2004). It should be appreciated that the data from both programmes will greatly extend the range of applicability of both programmes.*
3. *To gain a better understanding of the environmental impact on pavements. The LTPP sections are subjected to normal climatic influences whereas the CAPTIF testing was conducted under controlled conditions. It is therefore possible to investigate the specific environmental impacts on pavement performance, something which is relatively complex to do based on LTPP work alone.*
4. *Confirm CAPTIF life cycle and mass limit study results with the LTPP performance data.*[1]

An obvious emphasis of this study would be to increase the understanding of the deterioration of the pavement during the rapid failure stages.

Introducing uncertainty and statistical distribution in some models

According to some network experience combined with engineering observations, some of the pavement behaviour cannot be explained according to deterministic model formats. For example, this report has suggested a generalised linear model format for predicting cracking. Likewise, defects such as potholing and failures (shoving) would be easy to fit according to some statistical distribution such as a Proportional Intensity Model. This change in modelling approach would not be transferable like the HDM approach. However, it would produce modelling outcomes that more closely resemble actual behaviour in New Zealand. It is recommended that this development is undertaken as part of the dTIMS CT development consortium tasks, since it will optimise the input from all practitioners in New Zealand.

[1] Research Proposal: *Benchmarking pavement performance between Transit's LTPP & CAPTIF programmes* 05/06 Research Round. MWH New Zealand Ltd.

Abstract

New Zealand started a Long-Term Pavement Performance (LTPP) programme on the State Highway network during 2000. This report presents the first concrete outcomes from the calibration analysis, undertaken in 2005.

The cracking model, particularly the crack initiation model, is one of the most crucial in the simulation of pavement deterioration. It contributes to many other pavement models such as roughness and rutting. A comprehensive process of data analysis was carried out including a traditional calibration coefficient adjustment of the HDM-4 model, adjustment of all HDM model coefficients based on maximum likelihood estimation, linear model regression, and logistic model development. The same process was followed for the texture and rut progression model. The simplified model format of the texture model has been calibrated. Reviewing the model format of the rut progression has been less successful due to data shortages but a path for the next stage of development is proposed.

This research from 2005 highlights the merits of the various calibration and model-development techniques as well as providing a comparison of the model outcomes. This is done both in terms of their accuracy in predicting crack occurrence on a network and their applicability to networks outside of the development area.

1. Introduction

1.1 Background to the research

The intent of this research was to review the major pavement deterioration models used in the New Zealand Deighton's Total Infrastructure Management System (dTIMS).The New Zealand dTIMS system was adopted during 1999 and consists of the following components:

- a software analysis platform, dTIMS, which was superseded by a later version, dTIMS CT in 2004,
- an analysis framework developed to simulate New Zealand best practice in maintenance decision making,
- pavement deterioration models, which are based on the World Bank's Highway Design and Maintenance models(HDM-III), HDM-4 and some locally developed models,
- all supporting software required to perform data preparation and reporting.

This research was commissioned through the Land Transport New Zealand Research Programme and was carried out in 2005. It is the first calibration study aimed at reviewing the HDM models for New Zealand conditions. It will also set the basis for further research in this area.

1.2 Scope of the report

The purpose of this report is not only to report the findings from the calibration analysis and model development, but also to provide a strategy for future calibration needs. This strategy includes a recommendation regarding the recommended analysis approach.

The report starts with a brief summary of calibration levels and the analysis approach that has been considered during the analysis work. The existing HDM approach was adhered to wherever possible. However, where appropriate, alternative methods were recommended based on New Zealand and other international experience.

The report then documents the findings from a calibration performed on the major performance models. As a first attempt only the calibration coefficients are adjusted. This section highlights all the issues related to a poor match between the prediction model and the actual behaviour of the pavement. Outcomes from this section include the provision of regional calibration coefficients, further model development areas and proposed calibration processes.

Subsequently, the model formats are reviewed from basic principles. Significant variables are identified according to their influence on the dependent variables. Inter-relationships of variables are investigated leading towards the definition of new model formats. Where possible new model formats are compared with adjusted HDM models.

The last section makes recommendations and puts into place a 'road map' for future modelling calibration needs.

2. Background to the study

2.1 Long-term pavement performance studies in New Zealand

During the implementation of the asset management system in New Zealand, the HDM models were adopted with the knowledge that they would require calibration once the appropriate data became available. The need for calibration has also been highlighted in a number of modelling reports completed for both Transit New Zealand (Transit) regions and local authorities. As a result two Long-term Pavement Performance (LTPP) programmes were initiated:

- Transit established 63 LTPP sections on the state highways. An annual condition survey is performed on these sections and during April 2005 these sections were surveyed for the fourth time.
- Land Transport New Zealand, in association with 21 local authorities, established 82 sections on typical local authority roads in both urban and rural networks.

This report documents the calibration results based on the 2004/05 analysis round, formulates recommended calibration procedures, and reviews the appropriateness of the local authority data based on the past two years of survey data.

2.2 Research objectives

According to the HDM modelling philosophy, pavement deterioration models are provided with a set of default calibration coefficients. The intention of these coefficients is to be able to adjust the models for different climatic conditions. For example, most of the technical development work on the original HDM models was undertaken in Brazil and the climate and road building materials of New Zealand differ significantly from those in Brazil. In theory, the calibration coefficients are provided to cater for these differences. In a wetter climate with more sensitive soils, as exists in New Zealand, the expectation is that the calibration coefficients would need to be altered to reflect these conditions.

Some initial calibration analysis suggested that, for some defects, changing the calibration coefficients alone may not give satisfactory results. Henning & Tapper (2004) indicated that some models such as the roughness progression do not necessarily follow the model format as described by HDM. Figure 2.1 illustrates an example of a pavement deterioration model that required a model form change in order to reflect the actual pavement deterioration.

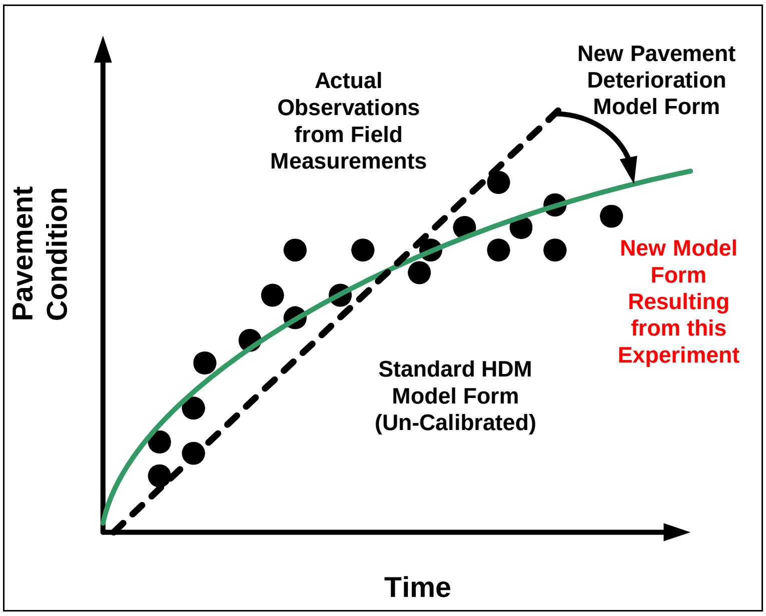


Figure 2.1 Example of changing the model format to fit actual data.

The main purpose of this research is to improve the fundamental understanding of pavement performance/deterioration including the regional variation. This was achieved through addressing the following objectives:

- To complete a full review of the pavement deterioration models used in dTIMS. This objective included the provision of calibration coefficients for the different climatic areas in New Zealand (Level 2 calibration).
- Should the analysis suggest a need for calibration beyond adjusting the calibration coefficients, to develop new model forms.
- To test the appropriateness of the local authority sections and data in terms of the defined experimental design.
- To develop a medium-term research strategy for model calibration in New Zealand.

2.3 LTPP data

The LTPP sections consist of 300 m-long sections selected according to a design matrix that ensures representative samples from different climatic areas, traffic, pavement, and network types. These sections are established on existing networks across the state highway and local authority network. On some of these sections (sterilised sections), no maintenance is allowed other than safety-related maintenance (e.g. pothole patching). The remaining sections are subject to normal maintenance practices for that particular network.

The LTPP data consist of inventory, as-built, traffic, strength, maintenance, and condition data. A summary of the data sources is provided in Table 2.1.

Table 2.1 LTPP data sources.

Data Item	Description	Data Source
Inventory	Pavement layer and surface details	Originally from RAMM* and other records but further validated by test pit information
Rainfall	Rainfall data	Purchased from NIWA
Pavement Strength	Analysed Falling Weight Deflectometer Data (FWD)	Annual FWD Surveys spaced at 50 m intervals
Traffic	Collected traffic data (AADT**) and the estimated percentage of vehicle type distribution	RAMM (for local authorities) and the Transit Traffic Management System (TMS) for state highways
Maintenance Records	Detail on any maintenance recorded on the LTPP sections	Submitted through software provided
Condition Data	Manually measured condition items such as roughness, rutting, texture and visual defects	Sourced through a condition survey contract

* averaged annual daily traffic
** road asset management and maintenance system

The condition surveys for both the programmes (Transit LTPP and Land Transport New Zealand) are secured through performance-specified survey contracts. According to these contracts, the accepted tolerances on the data collection precision are specified and the contractor nominates the instruments to be used. Henning et al. (2004a) and Transit (2001) documented the survey process along with the contracted technical specifications. The contractor opted to use the following instruments for the condition surveys.

Roughness The roughness is measured using an ARRB Walking Profilometer. Three measurements are conducted in each wheel path to achieve the required repeatability.

Rutting The transverse profile is measured with a self-driven Transverse Profile Beam (Figure 2.2). The transverse profile is measured at 10 m intervals and two measurements are conducted to achieve the required repeatability. The rut depth is subsequently determined using the HDM 2-m straight-edge method. The data are stored for 50-m subsections.

Figure 2.1 Transverse Profile Beam.

Texture The surface texture depth is measured on only 24 state highway LTPP sites. The Transit Stationary Laser Profilometer is used to measure a 1.6-m continuous length of the macro texture. These measurements are repeated at 10-m intervals in both wheel paths, thus resulting in a 16% sample size over the length of the LTPP site. This report also included some results taken on the network using the high speed laser measurements.

Visual rating The visual assessment of defects is conducted according to the HDM definition. This requires a detailed description of the defects, their location and extent. For example, all the cracks are recorded according to the type and the length/area of the crack.

Referencing Great emphasis has been placed on performing the measurement in the same position each year. The start and end positions of the sections were fixed according to GPS (global positioning system) co-ordinates, and metal pegs were also driven into the pavement at these positions.

Data used in this research The data used in this research included only the Transit data, since the local authority data consisted of only two years of measurements.

3. Calibration analysis approach

3.1 An important note on HDM models

This study has attempted either to improve or to suggest alternatives for the HDM pavement deterioration models. It is, however, paramount that the reader understands the difference between the philosophy behind the HDM models and data-driven models.

3.1.1 The philosophy behind HDM models

During the development of the HDM models, the World Bank aimed to provide generic type models which could be adopted internationally (provided that it is possible to calibrate them according to local climatic conditions and construction practices).

This approach necessitated the definition of the base models according to fundamental pavement behaviour principles. This means that the basic model format and all contributing variables were derived according to engineering principles. Model coefficients were then determined according to many international studies, e.g. in Brazil, Kenya, and Malaysia.

The main characteristics of these models are that they are transferable, and are able to model inter-effects of input variables. For example, these models could investigate what will happen if the rainfall in a particular area doubles while the traffic also increases by, say, 30%. The latter feature is possible since all variables affecting a particular model are included in the outcome, regardless of their apparent significance.

3.1.2 Data-driven models

Data-driven models are based on exclusive data for the study area which means the resulting model can only be adopted for that network. Data-driven models could be much more accurate than say, the HDM type approach, but also suffer some limitations including:

- They are only applicable for the given study area and are not transferable to other networks.
- They are valid only for the tested conditions, e.g. a model may have been developed based on well-designed pavements and may not be applicable for 'under'-designed pavements.
- As a consequence of the above, some of the models may not be able to test certain changes to the network. For example, a step-wise regression may reveal that rainfall is not significantly influencing pavement deterioration. However, this may be applicable only to the range of rainfall tested during the analysis. Should the current rainfall double, we would expect to see increased deterioration.

3.2 Calibration levels according to HDM

It is possible to obtain different levels of empirical calibration depending on the availability of data, funding for research, and time available before outcomes are defined. A more detailed and robust outcome can be expected if the calibration is based on high precision data collected and analysed according to sound statistical principles. The reality of road condition information is that it can be collected with differing levels of precision, accuracy, and sophistication. For lower order calibration, normal network level condition data may be appropriate. However, with any increase in accuracy required from the calibration process, a corresponding increase in data quality will also be required. This may even require a full scale performance trial to yield the required data.

The HDM modelling approach allows for different levels of calibration depending on the intended outcome of the study. Figure 3.1 illustrates the relationship between the resources and the time required to complete the different levels of calibration studies.

- At a lower level, only desk studies are performed and are mostly limited to network level data. This level corresponds to Level 1 - Application according to the HDM terminology.
- The next level involves some field surveys and an increase in resources committed to the calibration analysis. This is Level 2 - Calibration. Typically, one would expect the model coefficients to be adjusted based on this level of study.
- At the highest level, experimental surveys and full scale research are undertaken. Level 3 - Adaptation corresponds to the research levels used during the initial development of the deterioration models. It is therefore possible to develop new models or adjust the model formats at this level of calibration.

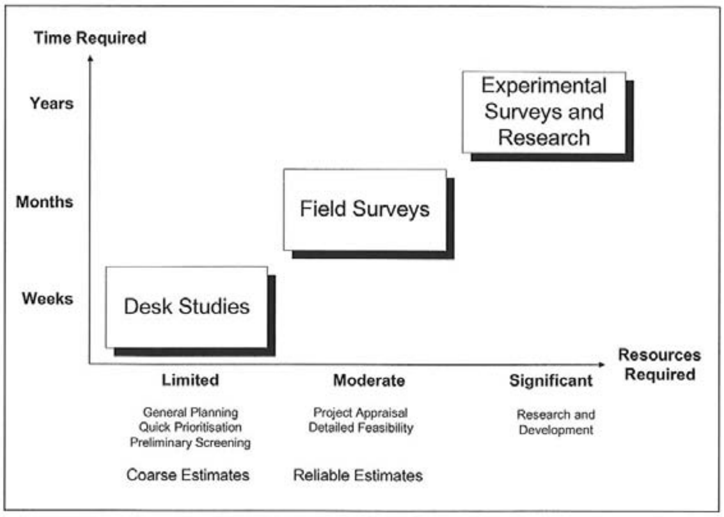


Figure 3.1 Conceptual presentation of resources and time required for calibration (Bennett & Paterson 2000).

In New Zealand, a Level 1 - Application analysis was completed during the initial implementation of dTIMS (HTC 1999). During this implementation it was recognised that a higher order calibration level would be required. The dTIMS system uses the incremental model format of HDM. This means that the condition change from one year to another is forecast. Calibrating the incremental model therefore requires robust condition data, as the expected condition change from one year to another is relatively small. The objective for the New Zealand calibration programmes was to collect the condition data to satisfy Level 3 – Adaptation. The intention was to be able to perform a Level 2 calibration within the shortest time-frame. Later, with sufficient data becoming available, a Level 3 calibration becomes possible, as explained in this study.

While the layout of the experiment and the development of the condition survey specifications are an integral part of the study, this will not be documented in detail as part of this report. For further information on the data collection, a copy of Henning et al. (2004b) is provided in Appendix A.

3.3 Analysis approach for HDM calibration Level 2

The main objective for Level 2 calibration is to adjust the calibration coefficients. The following sections provide experience and best practice regarding the calibration of the different pavement deterioration models.

3.3.1 Crack initiation

3.3.1.1 Existing model format

The HDM-4 crack initiation model forms are separated firstly for stabilised and granular bases and secondly for original surfacing and resurfacing of existing surfaces. Most New Zealand roads fall within the granular base category because most pavements are only lightly stabilised compared to world practice. The crack initiation for these types of pavements is (NDLI 1995):

Original Surfaces:

$$ICA = K_{cia}\left\{CDS^2 a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\} \qquad \text{(Equation 3.1)}$$

Resurfaced Surfaces:

$$ICA = K_{cia}\left\{CDS^2\left[MAX\begin{pmatrix} a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] * \\ MAX\left(1 - \frac{PCRW}{a_3}, o\right) a_4 HSNEW \end{pmatrix}\right] + CRT\right\} \qquad \text{(Equation 3.2)}$$

where:

ICA	time to initiation of ALL structural cracks (years)
CDS	construction defects indicator for bituminous surfaces
YE4	annual number of equivalent standard axles (millions/lane)
SNP	average annual adjusted structural number of the pavement
HSNEW	thickness of the most recent surface (mm)
PCRW	area of all cracking before latest reseal or overlay (% of total cracking area)

K_{cia} calibration factor for initiation of all structural cracking
CRT crack retardation time because of maintenance (years)
a_i model coefficients

The expressions basically consist of a structural crack component which is dependent on the SNP and the $YE4/SNP^2$. This value is then multiplied by the previous cracking and thickness of a new surface, for resurfaced sections.

3.3.1.2 Crack initiation calibration methodology

The HDM proposed method to calibrate crack initiation models is (Bennett & Paterson 2000):

$$K_{ci} = \frac{\text{mean OTCI}}{\text{mean PTCI}} \qquad \text{(Equation 3.3)}$$

where the $RMSE = SQRT\{mean[(OTCI_j - PTCI_j)^2]_{j=1,n}\}$

RMSE root means square error is the error function to minimise
OTCI observed time to crack initiation
PTCI predicted time to crack initiation

The disadvantage of the HDM approach is that it takes account only of sections that are cracked, thus ignoring sections which outlast expected performance. This method will therefore be biased towards early cracked sections. In order to take account of sections that are un-cracked beyond the point of predicted cracking, Jooste proposed an alternative method as documented in Rohde et al. (1998).

According to this method, K_{ci} is determined according to an iterative process which minimises the error (Err) between the predicted and the actual crack initiation process. The error is calculated according to Rohde et al. (1998):

$$Err = \sum w_i (TYCR - SAGE_2)^2 \qquad \text{(Equation 3.4)}$$

where:

Err is the error function to be minimised over the number of sections
$SAGE_2$ is the actual seal age at the time when crack initiation took place (first observation of cracking) or the current age when the section is still uncracked;
TYCR is the predicted time to crack initiation
w_i is the weighting factors:
0.0 if $TYCR > SAGE_2$ and the pavement is uncracked
1.0 if $TYCR < SAGE_2$ and the pavement is uncracked
1.5 if $TYCR < SAGE_2$ and the pavement is cracked
1.0 if $TYCR > SAGE_2$ and the pavement is cracked

The above weightings were subjectively derived and tested according to the model prediction outcome. According to outputs presented later in this report, the weightings are seen to be working well for New Zealand conditions.

Note that the differences between the RMSE and the Err error functions are:

- RMSE is expressed in terms of predicted and actual crack initiation. The Err function also incorporates surface age for pavements that have not cracked yet.
- The RMSE is calculated by taking the mean and square root of the difference between the predicted and actual crack initiation. The Err only takes the power of the difference, but it includes a weighting factor which is not included in the RMSE.

3.3.2 Crack progression models

The HDM crack progression model has a sigmoidal format. It is normally problematic to calibrate these since it takes a long time to collect these data in an experiment, and crack progression is seldom found on a network given early maintenance intervention. For this reason the HDM guidelines (Bennett & Paterson 2000) recommend a simple approach of:

$$K_{cp} = \frac{1}{K_{ci}} \qquad \text{(Equation 3.5)}$$

where: K_{cp} is the calibration coefficient for crack progression
K_{ci} is the calibration coefficient for crack initiation

For instances where crack progression data are available, the following method could be used:

$$K_{cp} = \frac{\text{mean PT30}}{\text{mean ET30}} \qquad \text{(Equation 3.6)}$$

where:

PT30 is the predicted age at 30% cracking
ET30 is the actual age at 30% cracking

According to previous attempts, the calibration of the crack progression model is difficult. Firstly, no historical data are kept after cracks have been sealed. Secondly, cracked sealed quantities are often not kept. However, the crack progression model has a low priority since engineering experience has indicated that the actual crack quantity is not very significant compared to the simple information of when a pavement cracks (i.e. crack initiation). This fact has been confirmed in some of the results in this report (see Section 4.2.2). For this reason, the crack progression model will not be investigated in this report.

3.3.3 Rut progression

3.3.3.1 Model description

The HDM-4 rutting model consists of the following components:

- initial densification,
- structural deformation,
- plastic deformation,
- wear from studded tyres.

Only the first three components of the rut progression are relevant to New Zealand conditions as studded tyres are not used in New Zealand. The following paragraphs discuss the model formats in more detail.

3.3.3.2 Initial densification

The initial densification is given by (NDLI 1995):

$$RDO = K_{rid}\left[a_0\left(YE410^6\right)^{(a_1+a_2DEF)}SNP^{a_3}COMP^{a_4}\right] \quad \text{(Equation 3.7)}$$

where:

RDO	is the rutting caused by initial densification (mm)
K_{rid}	calibration coefficient for initial densification
YE4	annual number of equivalent standard axles (millions/lane)
DEF	average annual Benkelman Beam deflection (mm)
SNP	adjusted structural number of the pavement
COMP	relative compaction (%)
a_i	model coefficients

The initial densification phase for rutting is valid for New Zealand conditions. Of specific interest is the relationship between the initial rut progression and the standard deviation. Currently, HDM-4 provides a positive linear relationship between rutting and the resulting rut depth standard deviation. It is believed that this trend may be negative during the initial stages of densification. The rut depth standard deviation is further discussed in Section 6.4.

3.3.3.3 Structural deformation

It is recognised by engineers that rutting is a very good indicator of the structural health of a pavement. For example, rutting is one of the key performance indices used on performance-specified maintenance contracts. Koniditsiotis & Kumar (2004) have demonstrated how to utilise the shape of the transverse profile in order to predict pavement structural capacity. The authors have establish a remarkable correlation between the rut patterns and pavement strength/capacity. It is anticipated that the rutting performance on networks will become more important in future as the understanding of this condition indicator increases.

HDM-4 provides two forms of rutting progression for cracked and un-cracked sections (NDLI 1995):

- Structural deformation for un-cracked sections

$$\Delta RDST_{UC} = K_{rst}\left(a_0SNP^{a1}YE4^{a2}COMP^{a3}\right) \quad \text{(Equation 3.8)}$$

- Structural deformation after cracking

$$\Delta RDST_{crk} = K_{rst}\left(a_0 SNP^{a1} YE4^{a2} MMP^{a3} ACX^{a4}\right) \qquad \text{(Equation 3.9)}$$

where

ΔRDST	is the incremental increase in structural deformation in the analysis year (mm)
K_{rst}	calibration coefficient for structural deformation
YE4	annual number of equivalent standard axles (millions/lane)
COMP	relative compaction (%)
MMP	mean monthly precipitation (mm/month)
SNP	adjusted structural number of the pavement
ACX	area of indexed cracking (% of total carriageway area)
a_i	model coefficients

3.3.3.4 Plastic deformation

The HDM-4 plastic deformation is presented as (NDLI 1995):

$$\Delta RDPD = K_{rpd} CDS^3 a_0 YE4 Sh^{a1} HS^{a2} \qquad \text{(Equation 3.10)}$$

where:

ΔRDPD	is the incremental increase in plastic deformation in the analysis year (mm)
K_{rpd}	calibration coefficient for plastic deformation
CDS	construction defects indicator
YE4	annual number of equivalent standard axles (millions/lane)
Sh	speed of heavy vehicles (km/h)
HS	total thickness of the bitumen surface
a_i	model coefficients

Default model coefficients are provided for both asphalt and chipseal pavements. The model format and the coefficients have to be validated for New Zealand roads, which often consist of multiple-surfaced layers.

3.3.4 Rutting calibration methodology

The calibration of the rut progression is a simple process of comparing the predicted with the actual rut depths:

Calculate the adjustment factor for mean rut depth progression, by geometric means or from log values (logORDMj and logPRDMj) as follows (Bennett & Paterson 2000):

$$K_{rp} = \text{Geometric Mean [ORDMj] / Geometric Mean [PRDMj] or}$$
$$K_{rp} = \text{[Sum (log ORDMj)] / [Sum(log PRDMj)]} \qquad \text{(Equations 3.11, 3.12)}$$

where

Krp	is the rut depth progression calibration coefficient
PRDMj	is the predicted rut depth for section

ORDM is the observed rut depth

The rut depth standard deviation is calibrated according to the same principle.

3.3.5 Surface texture

The classical HDM format of the texture depth model incorporated a complex process of combining two stages in the loss of texture depth (Figure 3.2). The initial stage sees a rapid loss in texture depth caused by the re-orientation and embedment of the seal chips. This stage is followed by a very slow loss in texture depth caused by the interaction between the surface and the vehicle tyres. The application of the texture depth model is difficult, given the sensitivity of the model towards the intercept point between the rapid texture loss phase and the gradual loss of texture later during the surface life. The main disadvantage of the model was the sensitivity to the reset (works effect) setting following construction. For example, if the wrong texture depth was assigned following construction, the error was compounded by the steep gradient for the initial texture loss according to HDM.

For this reason Transit (2003) conducted a study to determine a simplified method of calibrating the texture depth model. This was achieved by assuming a simplified model format as illustrated in Figure 3.2. Therefore, only the texture loss slope is derived and applied to the current texture value. For new work, a default texture depth at year one or two is assumed.

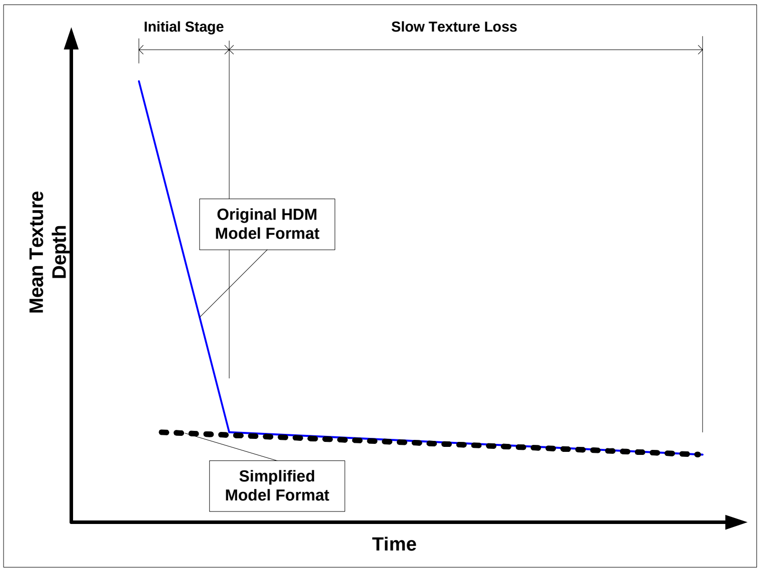


Figure 3.2 Changes to texture depth model format.

The proposed model calibration methodology included an option of three different methods Transit (2003):

- *Method 1 - Section Calibration of Texture Coefficients using the current HDM incremental texture model;*
- *Method 2 - National Calibration of Texture Coefficients using the current HDM incremental texture model; and*

- *Method 3 – Regional Calibration of Texture performance using developed Surface Age texture model.*

Method 1 involves analysing section specific data as illustrated in Figure 3.3.

The details for the calculation of the calibration coefficients are as follows (Transit 2003):

$$k_1k_2 = \frac{(t_n t_{n+1})}{Log_{10}(Y_n + \frac{1}{Y_n})} \qquad \text{(Equation 3.13)}$$

where:

t_n, t_{n+1}	is texture depth at year Y_n and Y_{n+1} and $t_n > t_{n+1}$
$Y_{(n,\ n+1)}$	is the surface age at date of texture reading ((reading date – surface date)/365)
kt_1kt_2	is texture deterioration slope

$$kt_1 = t_n + kt_1kt_2 log_{10}(NELV) \qquad \text{(Equation 3.14)}$$

where:

NELV	=	$\{365.Y_n[(1\text{-}f)AADT+10fAADT]\}$
f	=	fraction of HCVs
kt_2	=	kt_1kt_2/kt_1

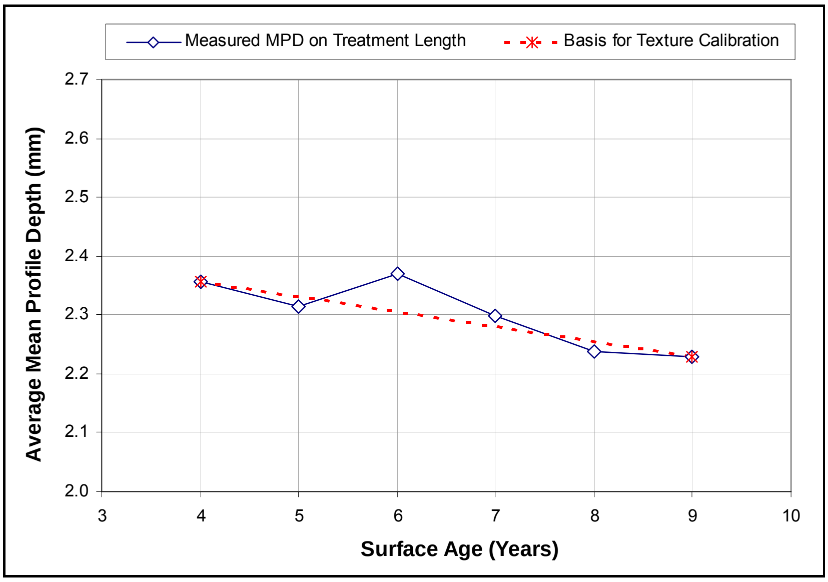


Figure 3.3 Method 1 section calibration principle (treatment length) (Transit 2003).

Method 2 involves the analysis of a full network or regional analysis to yield the calibration coefficients for the HDM texture depth model. It follows a similar process to Method 1, with the exception that kt_1kt_2 (regression slope) is determined by performing a regression analysis on the texture depth and equivalent light vehicles (Figure 3.4). Once kt_1kt_2 is determined, the equations for Method 1 are used to determine kt_1 and kt_2.

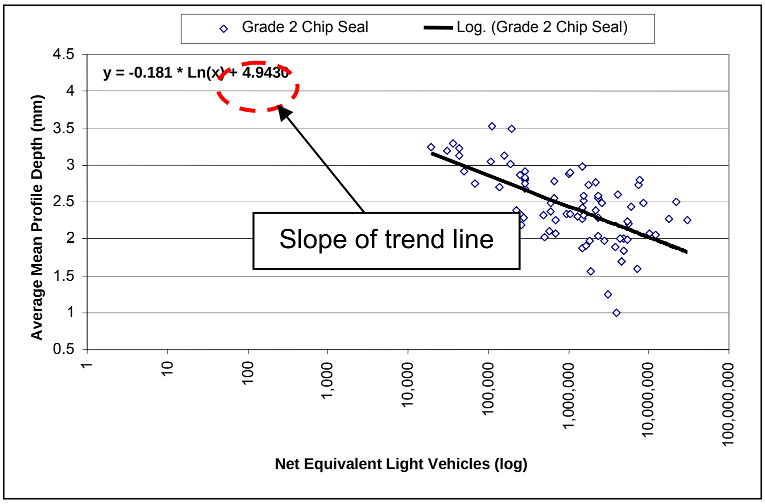


Figure 3.4 Obtaining the slope of the trend line for texture calibration.

Method 3 involves finding the slope of the trend line to determine kt_1kt_2 as illustrated in Figure 3.4. This is then incorporated into a simpler model to determine the incremental change in texture depth (Transit 2003):

$$\Delta TD = kt_1 kt_2 \times Log_{10}\left\{ {(Age2+1)}/{Age2} \right\} \qquad \text{(Equation 3.15)}$$

where: ΔTD = change in texture depth
kt_1kt_2 = the derived slope for the network
Age2 = the surface age in years

3.3.6 Roughness progression

3.3.6.1 Roughness model format

The roughness model is given by (NDLI 1995):

$$\Delta RI = K_{gp}\left[\Delta RI_S + \Delta RI_C + \Delta RI_R + \Delta RI_t\right] + \Delta RI_e \qquad \text{(Equation 3.16)}$$

where:

ΔRI is the total incremental change in roughness during the analysis year (IRI m/km)
ΔRI_S structural component of roughness change
ΔRI_C incremental roughness change caused by cracking
ΔRI_R change in roughness caused by the variation of rutting depth
ΔRI_t potholing effect on roughness
ΔRI_e environmental component of the roughness
K_{gp} calibration coefficient

Note: the components of the roughness model are discussed in the following paragraphs but the expressions are not repeated.

In the New Zealand context the potholing and cracking components of the roughness model can be ignored. New Zealand engineers follow an intensive routine maintenance regime so that these two parameters do not affect the roughness significantly.

3.3.7 Roughness model calibration

The roughness model consists of two calibration factors, namely K_{ge} (an environmental calibration coefficient) and K_{gp} (the calibration coefficient that caters for roughness increase caused by traffic loading).

The recommended calibration procedure for the roughness model is summarised below.

3.3.7.1 Adjustment of the environmental coefficient

According to Bennett & Paterson (2000), Kge and Kgp seldom need adjustment once the environmental coefficient *m* is established. The guides recommend a slice-in-time[2] analysis for adjustment of the environmental coefficient, inverting the absolute models of Paterson & Attoh-Okine (1992):

$$m = \frac{\ln\left[1.02\ RI_t - 0.143\ RDS_t - 0.0068\ ACRX_t - 0.056\ APAT_t\right] - \ln\left[RI_0 + \frac{135\ NE4_t}{(1+SNP)^5}\right]}{AGE3} \quad \text{(Equation 3.17)}$$

$$m = \frac{\ln\left[RI_t\right] - \ln\left[RI_0 + \frac{263\ NE4_t}{(1+SNP)^5}\right]}{AGE3} \quad \text{(Equation 3.18)}$$

where:

RI_t	=	roughness at AGE3 years after construction
RI_0	=	roughness when new
$NE4_t$	=	cumulative axle loading since construction
RDS_t	=	standard deviation of rut depth at AGE3
$ACRX_t$	=	area of indexed cracking at AGE3
$APAT_t$	=	area of patching at AGE3

This method is not always effective since it strongly relies on the correct estimation of RI_0. If it is based on actual data, it is often found that the scatter in RI_0 ranges more than the actual increase of roughness over time. Experience in New Zealand suggested that this method does not yield satisfactory results (Hallett & Tapper 2000).

3.3.7.2 Adjustment of all model coefficients – Riley's method

Riley proposed an alternative calibration analysis process for the roughness model in Henning & Riley (2000). An extract from this method is provided below:

[2] Slice-in time method takes the performance of pavements at a current date. The predicted value (for the given age) is compared with the performance of the pavement since its construction to the current age.

The following steps should be carried out in a spreadsheet.

Step 1

Obtain the best estimate of the environmental variable (m).

Step 2

Calculate the mean incremental values of IRI and RDS. In Excel the SLOPE function is useful for this purpose.

Step 3

Calculate the mean absolute values of IRI.

Step 4

Calculate the predicted values of the structural term using SNC, axle loading, construction age and the initial estimate of m from Step 1:

$$\text{Structural term} = \frac{\exp(m\ AGE3)\ YE4}{(1+SNC)^5}$$

Step 5

Make a multiple linear regression of observed mean incremental IRI against the following terms:

- *predicted structural component of roughness increment from Step 4*
- *observed mean increment in RDS from Step 2*
- *observed mean absolute value of IRI from Step 3*

The intercept should be set to zero when carrying out the regression.

The regression coefficients will then give a modified version of the component incremental model:

$$IRI = a1\frac{\exp(m\ AGE3)\ YE4}{(1+SNC)^5} + a2 \quad RDS + a3\ IRI_a$$

If the derived value of m (regression coefficient a3) differs significantly from the value obtained in step 1, repeat steps 4 and 5 using the value a3. Repeat the process until a stable value of m is obtained.

It is thought that the above method will be appropriate for rural roads where a time series of high speed data is available. However, many RCAs may have a time series of roughness data but lack data on RDS, having only RAMM data (length with rutting > 20 mm or 30 mm). It is thought that RDS is a major influence on roughness.

In urban areas, the area of patching may be a significant roughness component. If a historical time series of patched areas (whether pavement repairs or utility cuts) is available the mean incremental patching should be included as an additional term in the regression described above.

3.3.8 Pothole initiation and progression

Pothole initiation is fixed based on the cracking and ravelling initiation and progression. Only the progression can be calibrated according to Bennett & Paterson (2000):

For each calibration section, estimate the time for initiation of cracking (PTCI), the time for initiation of potholing (PTPI), and the time for progression of potholing to X units (PTPX) up to 500 potholing units. Compute the observed and predicted potholing times as follows:

$$\mathrm{OTPXj} = \mathrm{AGES} - \mathrm{PTPIj}$$

$$\mathrm{PTPXj} = \mathrm{PTPXj} - \mathrm{PTPIj}$$

Determine the potholing adjustment factor, either by linear regression of OTPXj against PTPXj, or as follows:

$$K_{ph} = \frac{\mathrm{mean(OTPXj)}}{\mathrm{mean(PTPXj)}}$$

Calibrating potholing in New Zealand would not be possible. Even on the calibration sections, potholes are not allowed because of safety concerns to the user. For that reason, the default model settings are accepted for pothole prediction.

3.4 Analysis approach for HDM calibration level 3 – Adaptation

Level 3 – Adaptation analysis could include either of the following approaches:

Adjusting current base HDM model format. The HDM base models are provided with a number of coefficients in addition to the calibration coefficients. These coefficients are provided to make provision for different material types, construction methods and soil conditions. They provide significant flexibility to the model calibration process. In Transit (2004) it was demonstrated that the correlation of the crack initiation model is greatly improved by adjusting these coefficients.

Developing new models based on first principles. Should it be established that a particular model format is inappropriate for the area in which it is applied, a new model format needs to be established. It is recommended that a similar approach be followed to the original HDM models as follows (adapted from Bennett & Paterson 2000):

- **Step 1 Define the base model format**

  A combination of existing knowledge on the mechanistic behaviour of a pavement and some empirical research is used to define the basic format of the model. For example, rutting for granular pavements might have a typical sigmodial format as illustrated in Figure 3.5. Other model formats include multiplicative, additives or power functions.

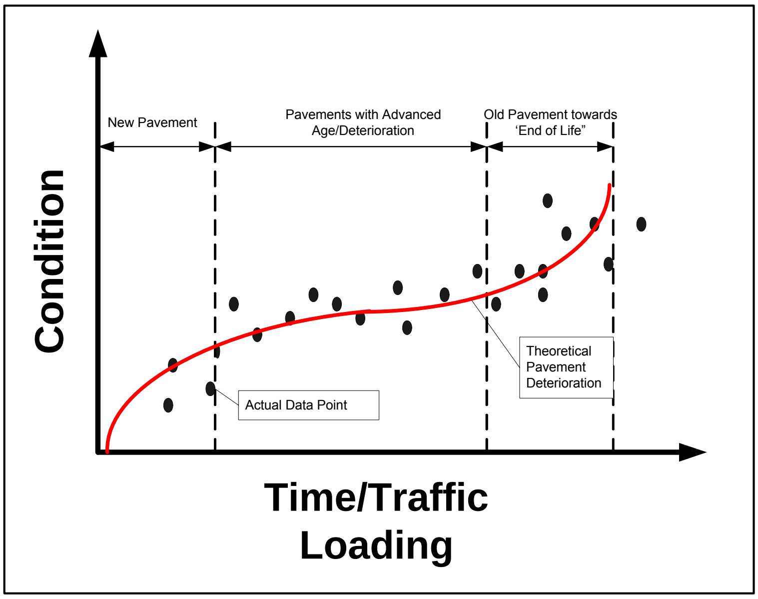


Figure 3.5 Example of a basic model format for pavement deterioration.

- **Step 2 Selecting cluster parameters/factors that form the basis of the model**

  Define the factors or parameters that affect the distress type under consideration. A critical component of this step is also to consider the interrelationship that defects have on each other.

- **Step 3 Adding in other parameters in expected format**

  It may not necessarily be possible to include some parameters according to Step 2. For these parameters, an assumption is used regarding the expected impact they may have on the defect according to an expected format.

- **Step 4 Finalise parameter coefficients through advanced statistical methods**

  Advanced statistical methods are available that can be used to define the final model coefficient settings. It is further recommended that an analysis of the residual errors be used to test the correlation of the intended model.

4. Crack initiation

4.1 HDM level 2 calibration

4.1.1 Cracking data used for the level 2 calibration

Given that the LTPP data have only been collected for the past three years, the data were not statistically robust for crack initiation calibration. An opportunity existed, however, to utilise the network survey data collected on the LTPP sections for this purpose. Most of the Transit LTPP sections are contained within the Transit benchmark sections. These benchmark sections are 1 km-long sections that undergo repetitive High Speed Data (HSD) surveys on an annual basis, and the visual rating is undertaken on these sites according to the RAMM survey methods. The repetitive HSD data are then used to benchmark the network HSD surveys in order to identify any bias in the equipment during the surveys. Since comprehensive inventory and condition data (RAMM rating) were available on these sections dating back as far as 1999, it was possible to perform the crack calibration using the benchmark section data.

Historical crack records from the RAMM rating data were interpreted according to Figure 4.1. Note that the inspection length of the rating was not always consistent since the start/end may have shifted over the years. However, it was aimed at using only the data points within the boundaries of the benchmark section.[3] Also note that the survey of the entire benchmark section length started in 2001.

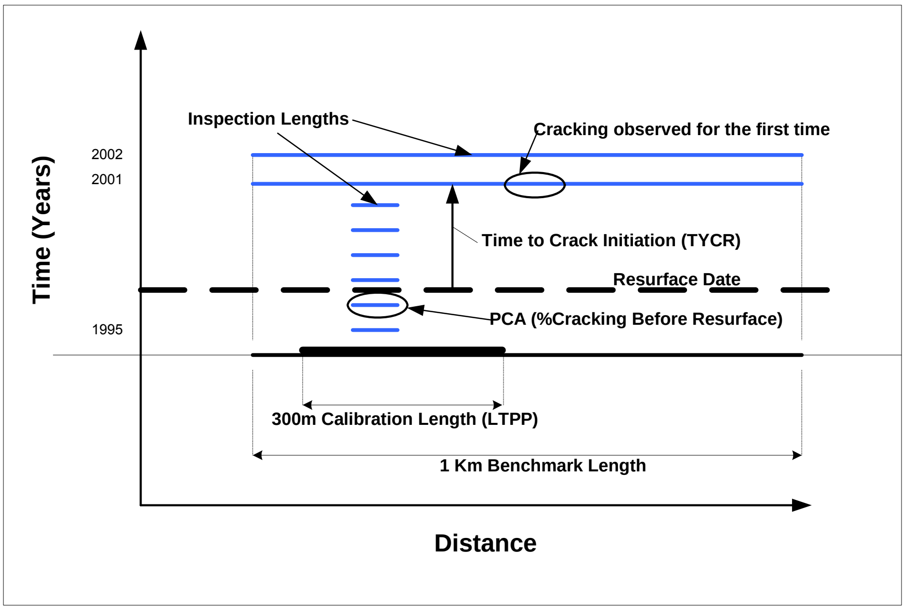


Figure 4.1 Interpretation of cracking data (Transit 2004).

3 The time to crack initiation was defined as the first observation in time when the crack percentage exceeded 0.5% of the total benchmark pavement area or the area of the rated section if it was shorter than the benchmark section.

4.1.2 Results

The resulting regional calibration factors are presented in Table 4.1 and Figure 4.2. The Transit LTPP sections are located in four climatic regions as described in Henning et al. (2004b), according to a climatic classification method proposed by Cenek (2001). According to this method, the climatic regions are classified according to the ratio of rainfall:wet strength properties of the soil. High and moderate risk areas include wetter areas combined with more sensitive soil areas (e.g. Northland), whereas low and limited risk areas are the drier and more stable soil types such as Canterbury. Statistically, more than 15 sections in each sub-category are required in order to have sufficient data for meaningful results. Sufficient data were available to group the results into only two climatic regions in order to obtain statistically significant results. The table indicates a smaller crack initiation factor (K_{ci}) for the high and moderate risk areas, thus suggesting an earlier crack initiation period. This observation is consistent with expectations, and also confirms the validity of the climatic regions as adopted for this study. The New Zealand outcome (K_{ci} equals approximately 0.5) compares well with other international calibration results (Rohde et al. 2002). Furthermore, New Zealand heavy rainfall and clay-type materials are expected to give a calibration coefficient which is less than 1.

Table 4.1 Summary of calibration result for different climatic regions (Transit 2004).

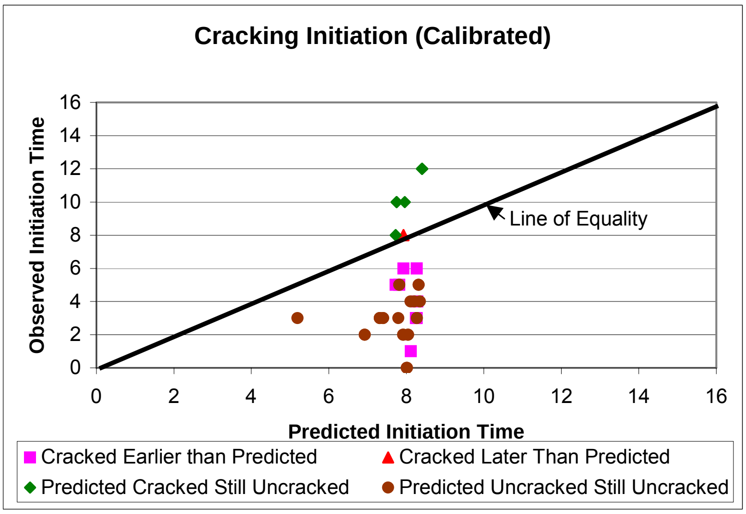
Factor	Regional Classification	
	High and Moderate	Low and Limited
Kci	0.49 (0.52)	0.59 (0.64)
Error(Err) – Default	194	477
Error(Err) – Calibrated	27	160

Notes: The error is calculated according to Section 3.3.1.
The values in brackets are the calibration coefficients obtained from data explained in Section 4.2.1.

Figure 4.2 illustrates the comparison between observed and predicted crack initiation. It further classifies the data according to four categories that indicate the relationship between predicted and actual crack initiation time. An observation from this figure is that the range of predicted crack initiation is much narrower compared with the range in actual crack initiation time. This observation corresponds well with calibration results obtained elsewhere (Henning et al. 1998). The wider spread in actual crack initiation time compared to the model can be explained as follows:

- A wider range will always be recorded in observed than predicted values because of the natural spread of the actual data and influences from external factors which are not incorporated into the model.
- The model calibration outcome as presented in this section involved only the adjustment of the climatic calibration coefficient. A closer fit between the actual and the predicted crack initiation can be obtained by adjusting all the model coefficients.
- different crack mechanisms may possibly exist, and aggregating them into one single analysis produces a poorer fit between the actual and predicted observations.

The latter two points relate to model form definitions and will be discussed further in this chapter.



Notes: Data included for all Benchmark Sections that correspond with the LTPP sections (i.e. 40 sections across New Zealand).
When calculating the predicted initiation time, the SNP was based on the back analysis of the average FWD reading for each section.
Results represent a default HDM-III crack initiation model.

Figure 4.2 Comparing actual cracking with predicted cracking (Transit 2004).

4.2 A review of the cracking model format

4.2.1 Crack initiation data for the model review

As illustrated in Section 4.1.1, the crack initiation data are limited for the Transit LTPP sections and benchmark sections. In order to expand the crack initiation data, network RAMM survey data were considered and found to be appropriate. The RAMM rating consists of assessing the length of cracked wheel path. This length of cracking is subsequently converted to percentage cracking, according to conversion factors documented in HTC (1999):

$$\text{Percentage Cracking} = 0.0004\left(\text{Alligator x}\frac{50}{\text{insp\_length}}\right)^2 + 0.28\ \text{Alligator}\frac{50}{\text{insp\_length}} \qquad \text{(Equation 4.1)}$$

where:

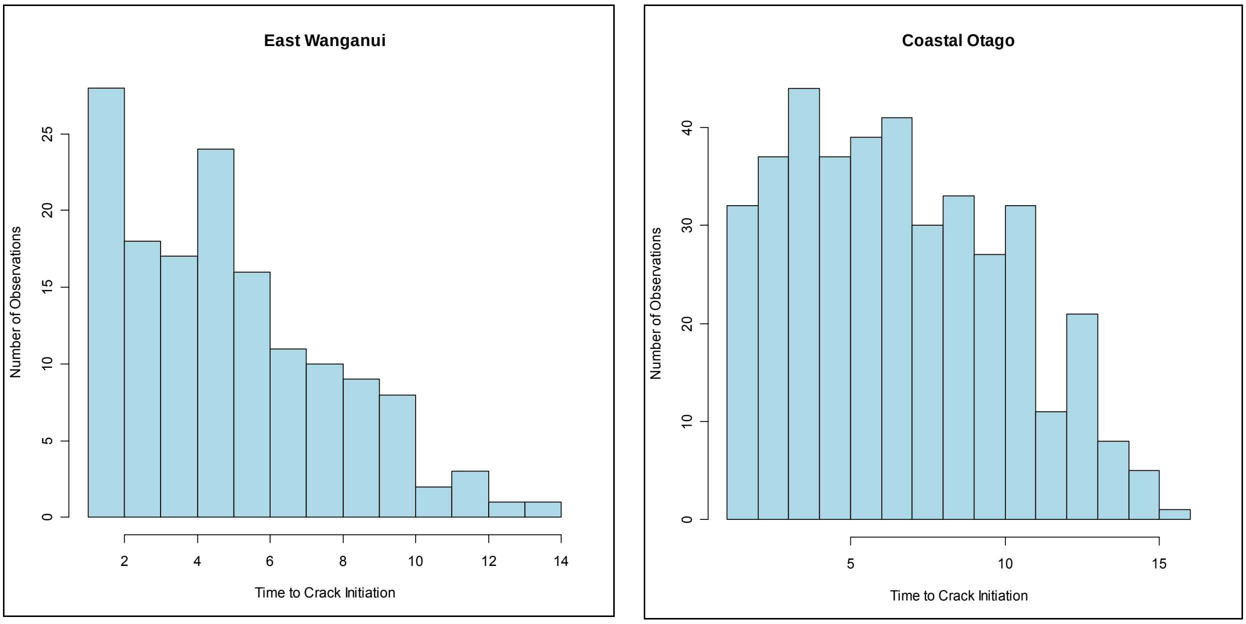
Percentage Cracking	is percentage of the total lane area cracked
Alligator	length (m) of the wheel path showing alligator cracking
insp_length	inspection length in (m)

The accuracy of this conversion does not cause any concern, since the crack initiation is identified at a point when the cracking exceeds 0.5% (or an equivalent of approximately 2 m of cracking on a 50-m rating section), and the accuracy is therefore not too sensitive to the outcome. It was important though, to select appropriate rating sections for the analysis in order to ensure that the same 50-m rating section was assessed for a number of years.

Two Transit regions (East Wanganui and Coastal Otago) were selected for the crack analysis. These two regions represent pavement deterioration for a medium and low climatic sensitivity area respectively (see Henning et al. 2004b). Furthermore, the data availability and knowledge of these networks allowed for an in-depth data interrogation. Specific sections used for the analysis were extracted according to the following criteria:

- Sections were included where the location of the 50-m rating sections have not changed from survey to survey.
- Each section had a minimum of four rating years.
- All information was extracted for comparing before-and-after performance of resurfacing (cracking in particular).

Only chipseal pavements were analysed for the purposes of the model development. Once the model format has been reviewed, further analysis will also be completed on alternative surface types. The data were also categorised for sections that had more than three layers of surfaces, and whether they were cracked before resurfacing. The distribution from the cracking data for the respective regions is presented in Figure 4.3.



Note difference in scale

Figure 4.3 Distribution of crack initiation for the two regions.

As expected, the average time to crack initiation was longer on the Coastal Otago region, thus confirming the appropriateness of the climatic classification. Coastal Otago consists of more stable soils and climatic conditions. It is also observed that more data were extracted from the Coastal Otago region, since the rating sections for this region were more stable over time. The imbalance of the data between the two regions was considered in the analysis.

4.2.2 Refining the existing HDM model format

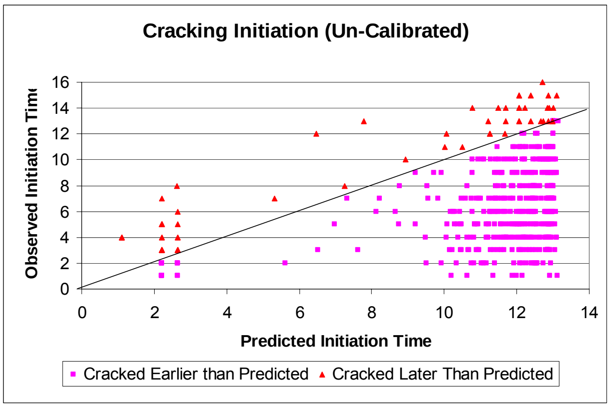
In the 2004 Transit study two methods were shown to be available to improve the crack initiation model:

- accept the HDM model format, but adjust all the model coefficients based on local data,
- a full review of the model that includes multivariate and regression analysis.

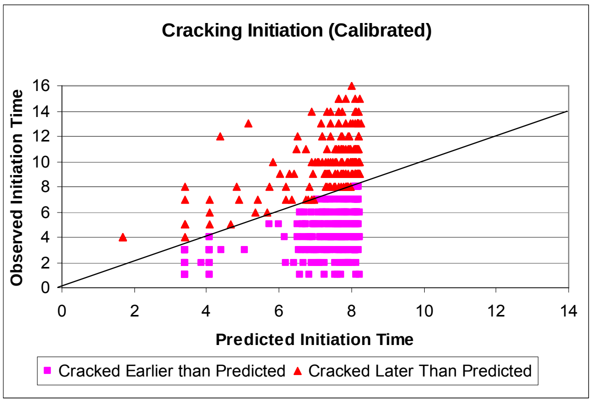
Both these methods were used and the adjustment of the HDM model coefficients is described in this section. Model coefficients a_0 to a_4 were adjusted by minimising the error between the predicted and the observed crack initiation (see Section 3.1.3). Note that both the cracked and uncracked sections were considered during this analysis.

Table 4.2 Resulting model coefficients for existing HDM model format.

Default HDM crack initiation model					HDM crack initiation model with adjusted coefficients				
a0	a1	a2	a3	a4	a0	a1	a2	a3	a4
13.2	0	20.7	20	0.22	8.3	0	18.54	0.01	0.34



Error = 17597.7



Error = 6697.9

Note: For the purpose of clarity, sections with no cracking observed are not indicated on the graphs.

Observations from Table 4.2 include:

- Using the new coefficients has reduced the error significantly from 17,597 to 6,697.
- Uncalibrated, the predicted crack initiation ranged from 1 to 13 years compared to the actual 1 to 16 years. The corresponding range for the predicted cracking initiation using refined coefficients ranged from 1 to just over 8 years. The current model and/or calibration process has limitations since the outcome gives a better fit for the over-all model, but does not necessary reflect the reality (e.g. the actual maximum is 16 years).
- The uncalibrated model does not predict any crack initiation over 13 years, and the maximum time to crack initiation for the calibrated model is just over 8 years.
- Most of the predicted crack initiation periods are between 10 and 13 years, and between 6 and 8 years for the default and calibrated model respectively.

Changes to the model coefficients suggest that cracking initiation caused by the loading/strength relation YE4/SNP^2 is reduced (both a_0 and a_2 have reduced). The influence of the new surface thickness has increased (a_4 increased). The influence of percentage cracking before resurfacing has changed completely. The coefficient a_4 has changed from a value of 20 to a significantly smaller number (0.01). For the default model, the ratio between the previous cracking and the coefficient was a continuous variable, whereas the calibrated model suggested that it is a binary variable (either 0 or 1). This suggests that crack initiation is a function of whether the old surface consisted of previous cracking or not. The actual value of previous cracking is not significant.

Although the calibrated model has a significantly better fit to the actual crack initiation, it is observed that the scatter between predicted and observed crack initiation time is still significant. Having looked at the figures in Table 4.2, one has to conclude that the model has little 'prediction power'.

4.2.3 Variable analysis

New model form development can be divided into three stages as follows:

- Variable analysis is aimed at better understanding the relationships between the possible variables and the predicted variable. During this stage it is also important to search for any inter-variable relationships. General trends and possible relation forms are noted during this phase, since it could simplify regression analysis which follows later.
- Subsequently multi-variate analysis is undertaken to determine the significant variables that influence the independent variable.
- Last, the regression is undertaken to define in which format the variables are combined, in order to predict the outcome of the independent variable.

Table 4.3 shows the variables considered during the analysis in the following sections.

Table 4.3 Variables considered for predicting crack initiation.

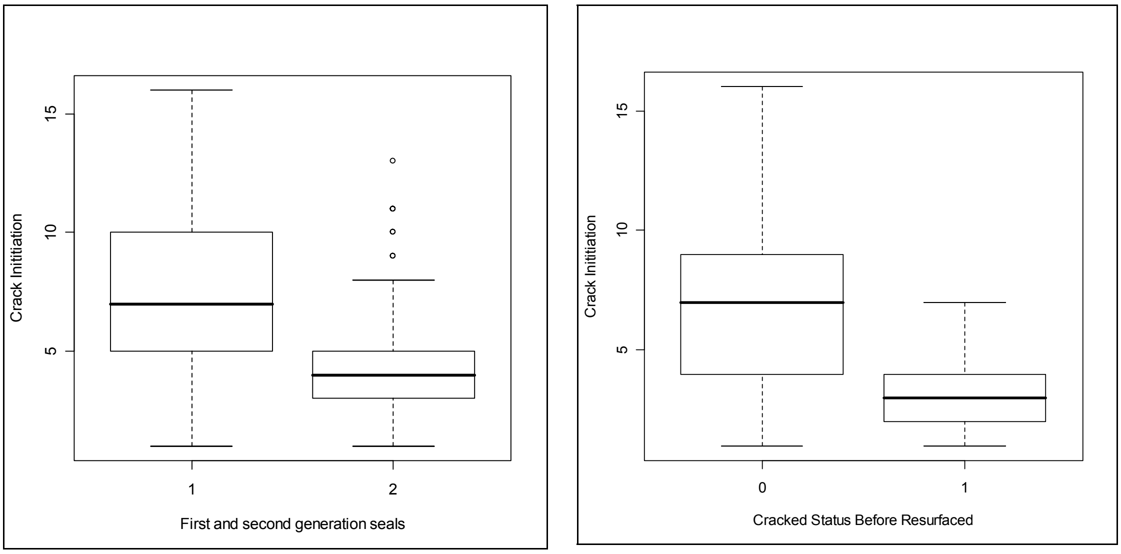
Variable	Description	Variable Type
AADT	the annual average daily traffic	Continuous
YE4	annual number of equivalent standard axles (millions/lane)	Continuous
SNP	Structural Number of the Pavement	Continuous
Surf_Gen	generation of the surface (for example first generation surfaces would be equal to 0 and represent the original surface layer after construction, and 1 representing all subsequent surfaces)	Factor
CS_PCA	cracked status before resurfacing (0 or 1 for uncracked or cracked)	Factor
HTOT	total surface thickness (mm) of all the layers	Continuous
HNEW	surface thickness (mm) of the latest surfaced layer	Continuous

Objectives of the analysis:

- The HDM model form suggests two different crack stages – the first generation cracking which occurs on newly constructed pavements, and the secondary cracks that occur on a resurfaced section which has cracked before. The latter mechanism is also referred to as reflective cracking. The first objective of the analysis would be to established whether the actual data supports these two crack stages.
- All variables need to be investigated in terms of the relationship with the crack initiation time to establish:
  - Does a relationship exist?
  - What is the format of the relationship?
  - Does this relationship change for different ranges of the data?
- Determine any inter-relationships between variables.

4.2.3.1 Condition of surface before resurfacing

Figure 4.4 presents the distribution of crack initiation for first/second generation surfaces (left plot) and for different cracked status before resurfacing (right plot). Both these plots clearly illustrate the distinct difference in crack initiation time for new surfaces and resurfaced seals. This variable was therefore expected to have a significant influence on the final model, and it has therefore been included in the multi-variate analysis.



Note: First generation seals (1) are the original surfacing following construction or granular overlay
Second generation seals (2) are resurfaced sections (i.e. some time elapsed between first generation and second generation seals.
Cracked Status (0) – new surfaces or resurfaced sections that have not been cracked before.
Cracked Status (1) – resurfaced sections where the previous surfaced was cracked before resurfacing.

Figure 4.4 Crack initiation for (left) different resurfacing cycles and (right) status before resurfacing.

The relationship between the crack initiation and the percentage cracking before resurfacing was further investigated, and no conclusive relationship was established (see Figure B.1, Appendix B). This result is consistent with findings in Section 4.2.2. It can therefore be safely concluded that whether a section was cracked before resurfacing will have a significant influence on the crack initiation period. However, the actual crack percentage before resurfacing is not significant.

It remains questionable whether both these variables should be included in the model. One of the aims of this process is to keep the final model as simple as possible, which may result in only one of these two variables being used.

4.2.3.2 Thickness of the new surface and the total surface thickness

Figure 4.5 shows the relationship between surface thickness and crack initiation. Two thicknesses were considered:,

- the new surface thickness,
- the total surface thickness.

No apparent relationship exists between the new surface thickness and the crack initiation time. Note, however, that the new surface thickness was derived from an assumed thickness given the surface code in RAMM. From experience, these data are not always accurate. Furthermore, the thickness does not indicate the film thickness of the bitumen in the surface. The film thickness of the bitumen is the inferred variable adopted in the HDM model. Oliver (2004) has demonstrated the significance of the bitumen age and thickness on the crack initiation period. However, this relationship was not confirmed with the data from the State Highway RAMM database.

Figure 4.5 (right-hand plot) shows an apparent exponential relationship between the total thickness and the crack initiation period. Possible explanations for this observed trend are:

- Multiple surface layers indicate older pavements which are more prone to cracking or were cracked before the last resurfacing. Cracking observed on these sections is therefore reflective cracking of third, fourth or even later generation seals.
- Multiple surface layers are well known to be more unstable (HTC 1999). Significant movement and flexing of the surface layers can therefore be assumed to occur, thus resulting in more strains and subsequent cracking of the newly surfaced layer.

Uncertainty still exists as to whether the total surface thickness should be included as a continuous variable, or whether thickness should be categorised into different levels. This aspect is further investigated in Section 4.2.5.

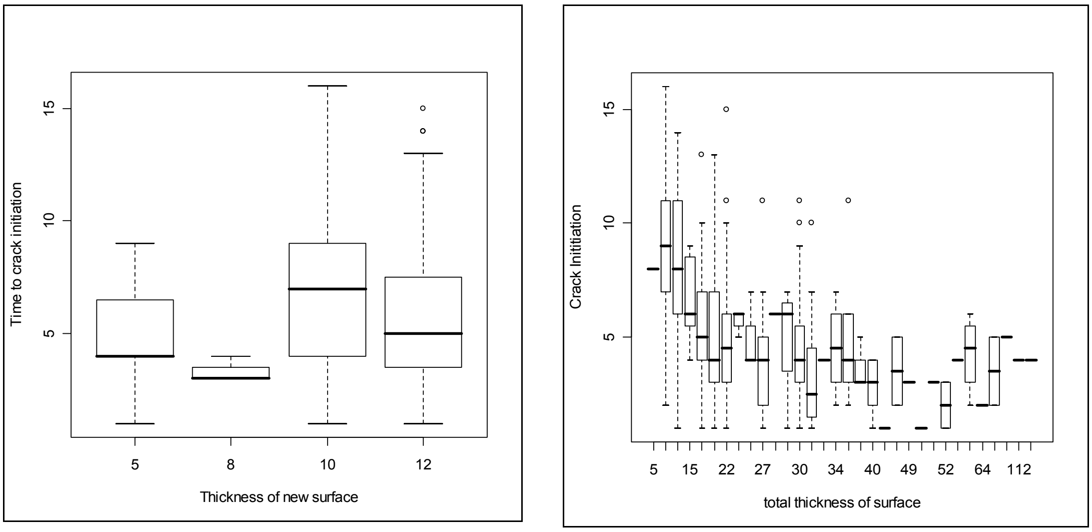


Figure 4.5 Relationship between (left) thickness of new surface and (right) total surface thickness with the crack initiation period.

4.2.3.3 Pavement strength (SNP)

The relationship between the crack initiation and structural number (as an isolated variable) was investigated, and no apparent relationship was observed. Figure 4.6 illustrates the crack initiation as a function of structural number. The figure shows the same relation, but for different surface thicknesses and cracked status. No sensible relation was observed in any of the graphs depicted in Appendix B.

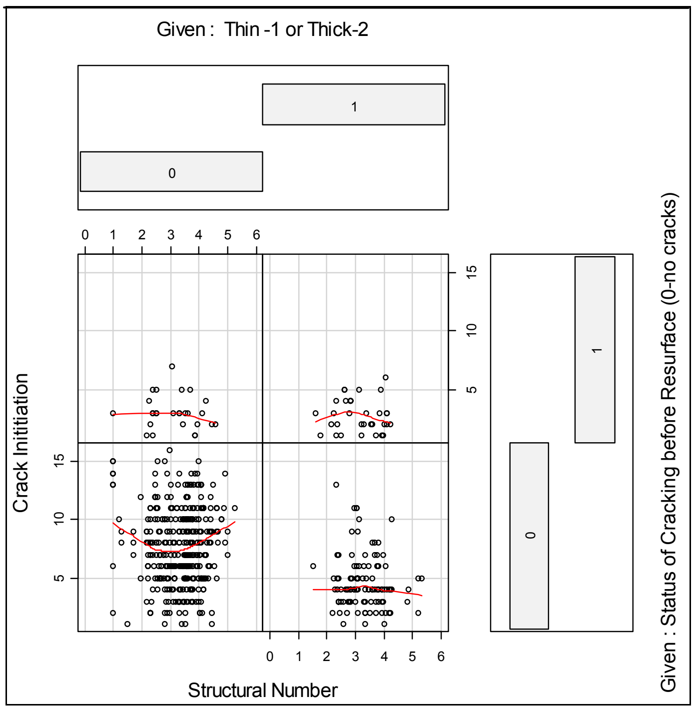


Figure 4.6 Crack initiation as a function of structural number for different combinations of surface thickness and cracked status.

The lack of a relation between SNP and the crack initiation is not completely unexpected, because it excludes the interaction of the traffic. In reality we know that pavements are constructed according to the expected traffic loading. For example, we expect a stronger pavement to show a longer crack initiation period than comparable pavements, which carry the same traffic loading but are weaker. The interaction between cracking, traffic loading and SNP is discussed in Section 4.2.3.5.

Despite the lack of a relationship, it is observed from Figure B.2, Appendix B, that newly surfaced sections, that showed no obvious signs of being cracked before, have a vastly superior performance to sections which have been cracked or have multiple-surfaced layers. The figure indicates an average crack initiation of close to ten years for these new surfaces, and less than five years for other sections.

4.2.3.4 Traffic loading

Figure 4.7 illustrates a possible relationship between the log of traffic loading (YE4), average annual daily traffic (AADT), and the crack initiation. It seems that a consistent relationship exists between traffic loading and cracking, regardless of cracked status before resurfacing. The log format was used for the traffic since all indications are that it is valid for crack initiation.

Since the total traffic loading is derived from the average annual daily traffic (AADT), a strong relationship with AADT is also expected. It is yet to be determined which one of these two parameters will provide the best estimate for the model. This aspect is also discussed in the following sections.

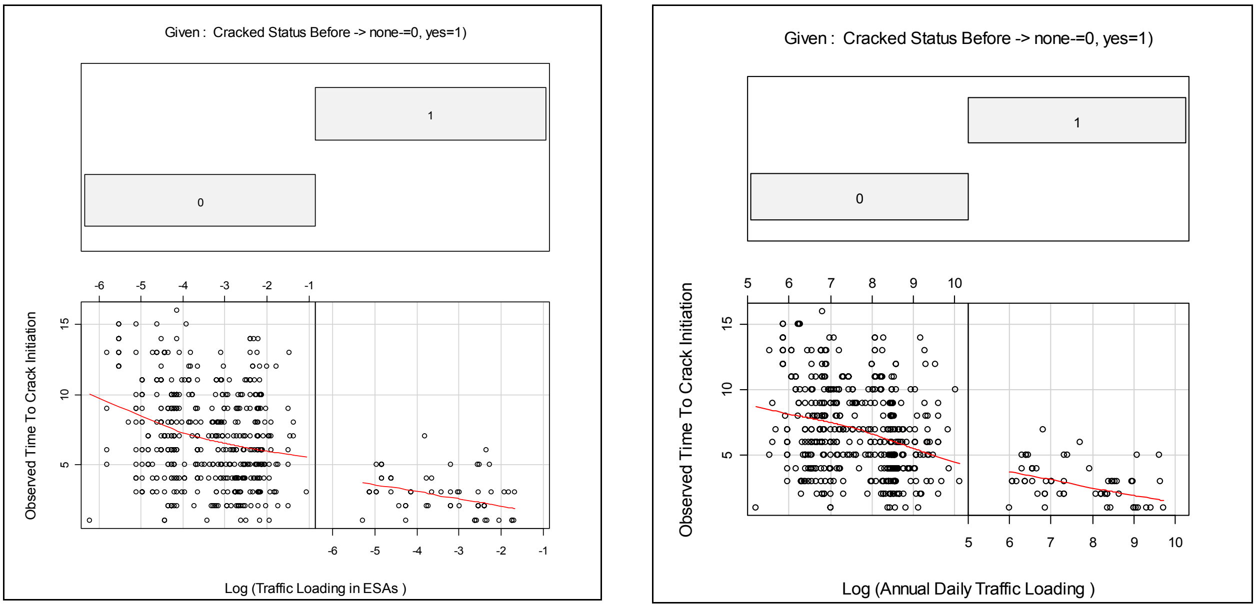


Figure 4.7 Observed crack initiation period as a function of traffic loading and annual daily traffic.

4.2.3.5 Traffic loading and structural number relation

The previous sections investigated the relationship between the traffic and SNP separately with the time to crack initiation. These suggest that not only traffic loading is directly related to the crack initiation. The next question was how the SNP and traffic loading as a combined variable relates to the crack initiation. Of specific interest was whether the (traffic loading/SNP^2) relation differs for new surfaces and previously cracked surfaces.

Note that (traffic loading/SNP^{2}) was investigated, since this is one of the factors in the HDM model (see Section 3.3.1.1). This relationship is illustrated in Figure 4.8, in which there seems to be a strong relationship between the traffic/SNP^2 and the crack initiation. This relationship seems to be of an exponential form. (Specific relationship significance and formats are discussed in Section 4.2.4).

The figure further shows that the relationship has a similar format regardless of whether sections have been uncracked or cracked before resurfacing. However, the relationship appears to be more distinct for previously un-cracked sections.

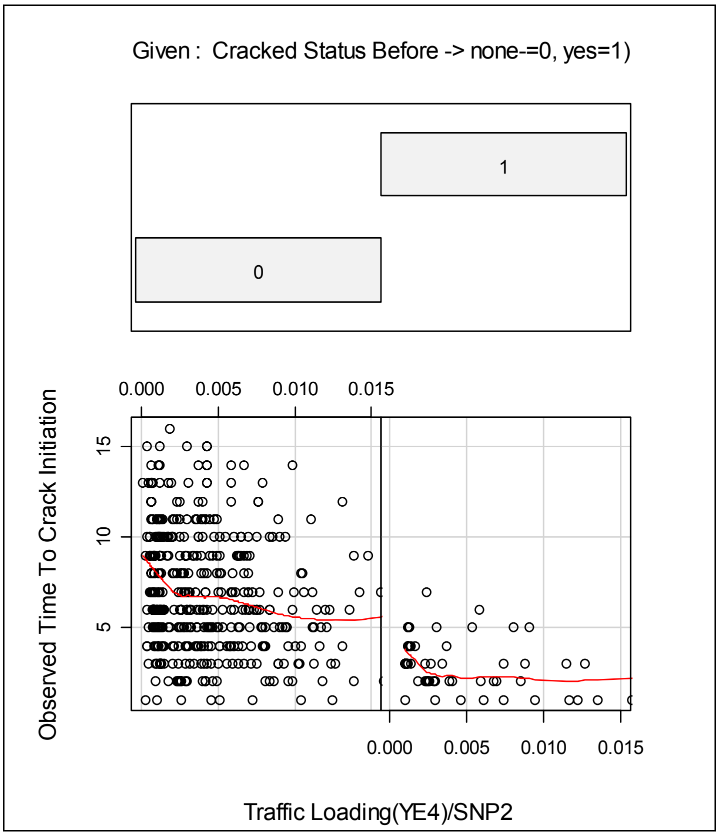


Figure 4.8 Crack initiation as a function of traffic loading and SNP.

4.2.4 Multivariate analysis

Multivariate analysis involves getting a better understanding of the significant factors in the prediction of crack initiation. It is important not only to get an understanding of how the factors relate to the crack initiation, but also to clarify how these factors relate to each other. As a first step, all the factors considered for the model were plotted against each other, and these are presented in Figure B.4, Appendix B. Although the apparent relationships did not reveal any unexpected trends, some issues to address are still worth mentioning:

- The traffic loading (YE4) is derived by assuming a certain percentage of heavy vehicles from the annual average daily traffic (AADT). We therefore expect these two factors to be related. What is more important though, is to consider which one of the two variables would be the most appropriate in the cracking model, and under which circumstances. For example, we may find that YE4 should be used for the first occurrence of cracking, and AADT for the reflective cracking.

- Obviously a relation between YE4 and SNP should exist as individual factors compared with the $YE4/SNP^2$ variable. What the optimal relationship of these factors is as a single predictor should be further investigated. For example, we need to establish whether SNP should be raised to the power of two, or any other value.

Both the AADT and YE4 factors will be used during the regression analysis. The $YE4/SNP^2$ combined factor was tested for significance. It was established that these factors were significant predictors of crack initiation for both cracked and un-cracked sections before resurfacing (see Figure B.5, Appendix B).

4.2.5 Linear model (LM) regression analysis results

A stepwise model regression was used to obtain the significant variables influencing the time to crack initiation. Both forward and backward step methods were used for the analysis. With the forward method, each variable is introduced incrementally and tested to see if it contributes meaningfully towards predicting the outcome. This process is continued until no more variables or combination of variables improve the model. With the backward method, the process starts with all the possible variables included in the model and removed one-by-one until the model outcome is optimal (i.e. ending up with the lowest error). The process resulted in the crack initiation time being predicted by:

$$ICA \sim f\left(Surf\_Gen + CS\_PCA + HTOT + AADT + (HTOT : AADT)\right) \quad \text{(Equation 4.2)}$$

where:

ICA	is the crack initiation time in years after the surface is constructed
Surf_Gen	is the generation of the surface (e.g. first generation surfaces would be equal to 0 and represent the original surface layer after construction, and 1 representing all subsequent surfaces)
CS_PCA	is the cracked status before resurfacing (0 or 1 for uncracked or cracked)
HTOT	is the total surface thickness (mm) of all the layers
AADT	is the annual average daily traffic

The model coefficients and respective model statistics are presented in Table 4.4 and model diagnostics are given in Figure B.6, Appendix B. The model outcome has confirmed the observations made in the previous sections regarding the significance of the model variables as follows:

- The status of the overall surface is the prominent predictor of cracking. That includes how thick the total surface is and whether it has cracked before resurfacing.
- The only other significant variable is the traffic.
- The pavement strength (SNP) and traffic loading (YE4) were not significant factors.
- An inter-relation also exists between the HTOT and AADT, which suggests that the influence of total surface thickness differs for different traffic ranges (see Figure 4.9).

Table 4.4 Results of regression analysis for predicted crack initiation.

Variable	Estimate	Std Error	t value	Pr(>|t|)	Significance[1]
Intercept	$1.088e^{1}$	$4.051e^{-1}$	26.845	$< 2e^{-16}$	***
Surf_Gen	$-7.659e^{-1}$	$3.783e^{-1}$	−2.025	0.0434	*
CS_PCA	−3.288	$3.868e^{-1}$	−8.500	$< 2e^{-16}$	***
HTOT	$-1.858e^{-1}$	$2.489e^{-2}$	−7.465	$3.35e^{-13}$	***
AADT	$-4.693e^{-4}$	$6.578e^{-5}$	−7.134	$3.14e^{-12}$	***
HTOT:AADT	$1.858e^{-5}$	$3.179e^{-6}$	5.844	$8.84e^{-9}$	***

Note [1] Significance codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1
Residual standard error: 2.758 on 540 degrees of freedom
Multiple R-Squared: 0.3581. Adjusted R-squared: 0.3522
F-statistic: 60.25 on 5 and 540 DF, p-value: < 2.2e−16

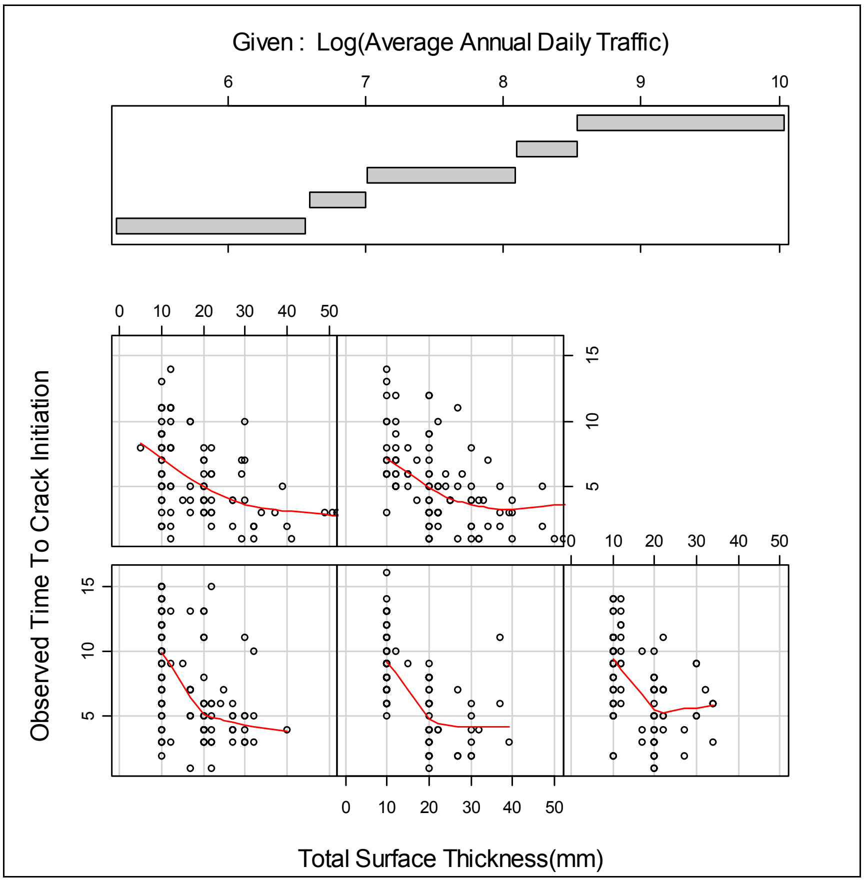


Figure 4.9 Inter-relationship between total surface thickness (mm) and log traffic (AADT).

The model could be further improved by considering other model formats. For example, by transforming the observed crack initiation to a logarithmic scale raised the R^2 to 0.45. (see Figure B.7, Appendix B). A simplified and recommended form of the model can be given by:

For PCA = 0 (sections not cracked before resurface)

$$ICA = Kci * \exp[5.7 - 1.25\log(HTOT) - 0.3\log(AADT) + 0.08\log(HTOT) * \log(AADT)] \quad \text{(Equation 4.3)}$$

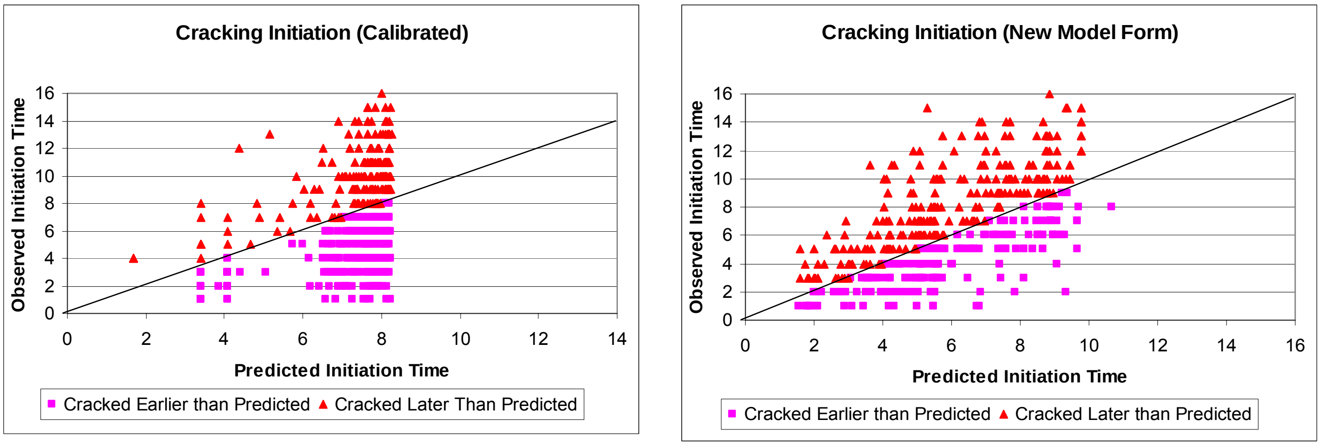
For PCA > 0 (sections were cracked before resurface)

$$ICA = Kci * \exp\left[4.6 - 0.68\log(HTOT) - 0.47\log(AADT) + 0.08 * \log(HTOT) * \log(AADT)\right] \quad \text{(Equation 4.4)}$$

where:

ICA	is the crack initiation time in years after the surface is constructed
HTOT	is the total surface thickness (in mm) of all the layers
AADT	is the annual average daily traffic

The final comparison of the predicted versus the observed crack initiation for the new model format is presented in Figure 4.10. This figure compares the resulting predicted crack initiation for the adjusted HDM-4 model (left plot - see Section 4.2.2.), and the new model format (right plot), resulting from the linear regression and expressions indicated above.



Error = 6697.9

Error = 5157

Figure 4.10 Comparing predicted versus actual crack initiation for new model format.

Comparing the figures, this illustrates an overall better fit of the new model format. Note that the predicted values are more evenly spread across the range of initiation times (two years to ten years), as opposed to the concentration of predicted values between six and eight years for the HDM model. The error of the new model format is lower than the adjusted HDM model and has improved the accuracy by a factor of three, compared to the default HDM model.

However, a large scatter is still observed between the predicted and actual values. Some reasons for this scatter are discussed in the subsequent section.

4.2.6 Generalised linear model (GLM)

From the previous sections we have observed that the best correlation coefficient obtained from the data is R^2 = 0.45 for the linear model (LM) regression. This means that in the case of the LM, 45% of the crack initiation behaviour can be explained by the variables included in the derived expression. Therefore many other factors can be assumed to influence the model outcome that are not included in the expression, and in most cases for which no data exist. Some of these missing variables could include:

- the quality of the bitumen,
- construction practices,
- oxidation properties of the bitumen,
- bitumen film thickness,
- specific rainfall and/or other climatic effects.

Generally, the robustness of the model can be improved by including some of these factors such as construction quality to the model (see Watanatada et al. (1987) and Henning et al. (1998)). The problem with these factors is that this information is rarely available for networks, and it therefore does not contribute effectively to make the model more applicable to the network under consideration.

Second, the pavement model will never be able to predict pavement behaviour 100% accurately, because some random effects will always exist that may influence behaviour outside the scope of the prediction model. For this reason, correlation coefficients of less than 0.5 are common in pavement performance prediction. The question is whether such low success rates are really acceptable within the pavement management system.

An alternative method would be considering the actual statistical distribution of the failing point or defect initiation point. Therefore, by presenting the prediction model in a different way, we incorporate uncertainty resulting from the factors previously ignored in the absolute model. Using this approach the model does not necessarily become more accurate, but it will be more robust in quantifying probabilities of failure.

This approach is commonly used in other engineering applications such as determining the concrete crushing strength for bridge structures. Figure 4.11 illustrates this principle. Seven concrete samples are tested at six crushing values. At the minimum required strength, only one of the seven samples has failed (14%). The specification for the concrete strength may have specified that only 5% of the samples are allowed to fail, which suggests that this concrete batch does not comply with the specification. In statistical terms, this illustration presents a binary model that yields the distribution of whether a sample has failed or not. According to this approach, it is also possible to create a binary crack initiation distribution for pavements. In this instance we will create a distribution of crack status for different surface ages.

For the purposes of this analysis, all the crack data has been transformed to a binary format detailing the age of the surface at which the cracked status changes to 'true'. Similarly, uncracked surfaces will remain cracked status = 'false' at the given surface age. As in the previous section, a stepwise regression was performed, with the only difference being that no intercept was specified. Given the nature of this model, two outcomes are possible and, by specifying an intercept, the model accuracy is greatly reduced. The resulting model from this analysis was:

$$\mathrm{STAT.ACA} \sim f\left(\mathrm{AGE2} + \mathrm{FACTOR(stat.PCA)} + \mathrm{Log(AADT)} + \mathrm{Log(HTOT)} + \mathrm{SNP}\right) \quad \text{(Equation 4.5)}$$

where:

STAT.ACA	cracked status within a given year
AGE2	is the surface age in years, since construction
stat.PCA	is the cracked status before resurfacing (0 or 1 for not cracked or cracked)
HTOT	is the total surface thickness (in mm) of all the layers
AADT	annual number of equivalent standard axles (millions/lane)
SNP	is the modified structural number

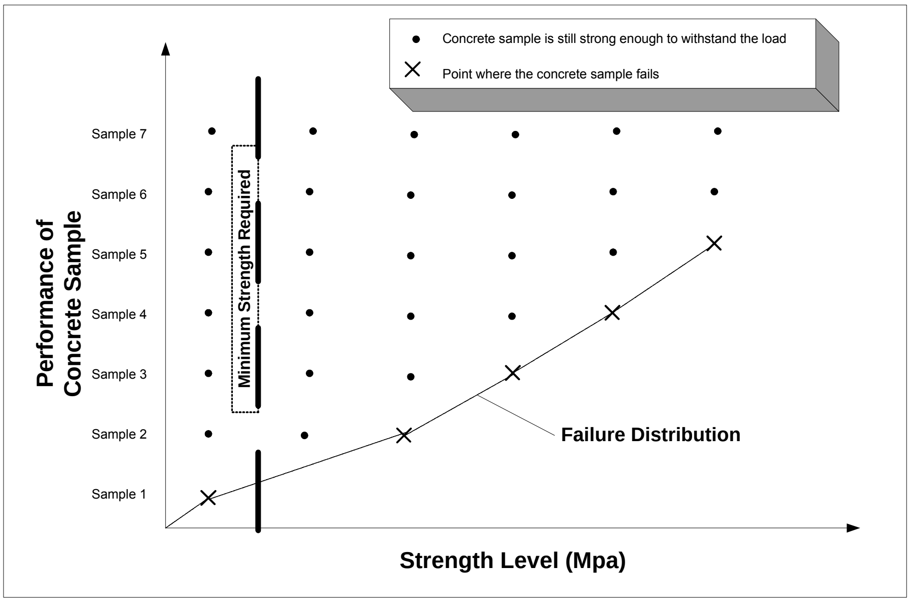


Figure 4.11 Example: Crushing strength of concrete samples.

Table 4.5 Results of regression analysis for predicted crack initiation.

Variable	Estimate	Std. Error	t value	Pr(>|t|)	Signif.[1]
age2	0.141	0.010	13.931	$< 2e^{-16}$	***
factor(stat.pca)0	−5.062	0.496	−10.211	$< 2e^{-16}$	***
factor(stat.pca)1	−3.440	0.508	−6.778	$1.22e^{-11}$	***
log(adt)	0.455	0.057	7.949	$1.88e^{-15}$	***
log(htot)	0.275	0.078	3.542	$3.97\ e^{-4}$	***
snp	−0.655	0.052	−12.721	$< 2e^{-16}$	***

Note: Significance codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1
Null deviance: 10462 on 7547 degrees of freedom
Residual deviance: 5606 on 7541 degrees of freedom
AIC: 5618

Again, the results obtained from the GLM analysis have been consistent with the data observations presented in earlier sections. We are starting to observe a much higher correlation for the coefficients and for the overall model fit.

The following expression can be used to convert the model format into a proportional model (Chambers & Hastie 1992):

$$p = 1 \Big/ \left[1 + \exp\left(-a - Bx\right)\right] \quad \text{(Equation 4.6)}$$

where:

- *p* is the probability that a specific event occurs (*p*(Y=1)
- *a* is the coefficient on the constant term
- *B* is the coefficient on the independent variables
- *x* is the independent variable(s)

Therefore, the recommended crack initiation model is:

$$\text{p(stat.aca)} = 1 \Big/ \left[1 + \exp\begin{pmatrix} -0.141\text{AGE2} + \{(5.062, 3.440)\ \mathit{for}\ \text{stat.pca} = (0,1)\} \\ -0.455\text{Log(AADT)} - 0.275\text{Log(HTOT)} + 0.655\text{SNP} \end{pmatrix}\right] \quad \text{(Equation 4.7)}$$

where:

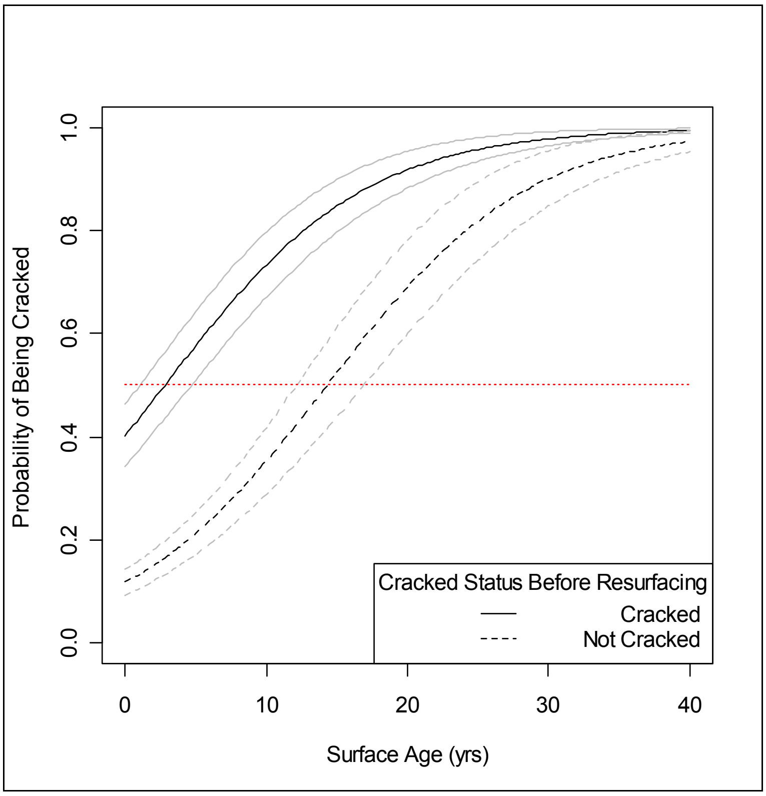
- p(stat.aca) is the probability of a section being cracked
- AGE2 is the surface age in years, since construction
- stat.PCA is the cracked status before resurfacing (0 or 1 for uncracked or cracked)
- HTOT is the total surface thickness (mm) of all the layers
- AADT is annual number of equivalent standard axles (millions/lane)
- SNP is the modified structural number

Figure 4.12 illustrates an example of the output from the GLM model. It shows two probability plots of cracked status for sections being cracked or uncracked before resurfacing. It suggests that, for the given data, one can expect sections to crack in 3 to 15 years, depending on the crack status before resurfacing.

Further outputs from the GLM model are presented in Figures B.8 and B.9, Appendix B. With these outputs, the sensitivity of the model was tested for different levels of traffic and structural number. It is observed that the probability of cracking changes significantly for varying levels of the independent variables. This is a significant observation since it illustrates that the model has significant predictive capabilities.

Figure B.10, Appendix B illustrates the diagnostic plots for the proposed model for the various independent variables. On these plots two lines are fitted to the data. The solid line represents the smoothing line (line of least error) while the dotted line is the fitted line from the model. If these two lines correlate, we can assume that the model format is appropriate. From the plots presented, we can observe that the model format is appropriate for the variables included in the model.

However, it should again be mentioned that this model is a strongly data-driven model, and is only applicable for the data it was derived from. Further tests of the model are required in order to assert the applicability on other state highway networks.



Note: Data plotted for AADT = 2500, HTOT = 60 mm, SNP = 2.5
Confidence interval plotted for two standard deviations
Expected crack initiation where p = 0.5

Figure 4.1 Output from the GLM model giving probability of cracking for a given year.

4.3 Summary of modelling review

This chapter has presented a full modelling review that included:

- an HDM Level 2 calibration resulting in environmental calibration coefficients,
- a model review of the existing HDM-4 model format resulting in new proposed model coefficients,
- a new proposed model format that included the development of a simplified linear model,
- a newly proposed concept in predicting the cracked status using linear logistic methodologies.

This chapter has demonstrated that any reviews resulted in a more robust prediction of crack initiation compared to the default HDM-4 model. This is expected, since all the processes (in the order listed above) are progressively moving towards a more data-driven model that will yield a better fit between predicted and actual behaviour.

In particular, the logistic model provides the most promising results. Various factors contribute towards the logistic model that is recommended for adoption in New Zealand including:

- More explaining variables are included to the logistic model. In particular it contains the surface age (AGE2) as an independent variable. The surface age acts as a moderator for other factors for which no data are available (e.g. oxidation of bitumen). Note the surface age of crack initiation is the independent variable for all the other model formats.

- The model format is relatively simple with most factors included in the model in an additive method.
- With the logistic model, all data on the network are being considered as a basis for the analysis and as a result the model takes account of both under-performing and over-performing pavements. Despite all best intentions this is not achieved with the HDM-type model calibration which mostly considers existing crack information and therefore does not take full account of uncracked/over-performing sections.
- The model not only gives a definitive predicted value such as expected crack initiation; it also gives a probability of a section being cracked for a given set of circumstances. This allows for much flexibility in the adoption of the model into a pavement management system. For example, triggers can be set according to different risk/criticality considerations. This flexibility could also be considered in the interaction with other models such as rutting and roughness. For example, we may want to do crack sealing when the probability of cracking reaches 50%. However, the influence of cracking on the rutting may become an issue if this probability of cracking reaches say 70%.
- The model responded well on varying levels for the variables, thus making it ideal for sensitivity analyses such as investigating the effect of changing traffic volumes and pavement design options. This will greatly enhance the predictability within the current New Zealand system compared to the current approach.

Despite the advantages mentioned, the current model is a very strong function of the data it was derived from. For example, the model contains the average traffic (AADT) instead of the traffic loading (YE4) as is expected. However, from experience we know that we have more confidence in the AADT data compared to the traffic load (YE4)[4]. Therefore, having a less meaningful but more accurate variable sometimes gives better model outcomes, compared to variables with questionable quality.

Given that the logistic model is a very strongly data-driven model, it must be tested for more networks before it is adopted into a national modelling system.

[4] Data reviews in New Zealand have shown that the AADT data on state highways is robust in most cases but that traffic loading (a function of traffic composition and assumed loading per axles) does not always reflect reality.

5. Texture

5.1 A review of the simplified texture model

In this section, a statistical model was used to predict the texture, and in particular, the mean profile depth (MPD), of roads. This builds on previous suggested methods outlined in Section 3.3.5. The process of the texture calibration involved two distinct steps:

- A model development process was followed on network High Speed Data (HSD) measurements.
- These results were then compared with the LTPP texture data, which was collected using the Transit Stationary Laser Profilometer.

5.1.1 Description of data

The data consist of measured MPD and total equivalent vehicles NELV data across six regions for seven different road surface types (referred to as the original data). The data are summarised in Table 5.1. Of particular interest is the relatively sparse data set for Auckland and Christchurch. Also included within Table 5.1 is the calibration data that consists of measured MPD for three surface types. Note that a data point in the MPD data is an average MPD over a 50-m length of high speed measurement. Conversely, a calibration data point represents a more accurate measurement, but over a length of only 1.6 m. It is assumed that the two sets of data can be reasonably compared without any applied weighting because of the differences and accuracy in measurement techniques (i.e. they approximately balance each other out). Further, the differences in length in the original sample can also be somewhat ignored, as they tend to be relatively similar and any weighting would add little value and more complexity.

Table 5.1 Number (N) and length (km) of roads in each region by surface type.

	Surface Type													
	Asphalt		CHIP.G2		CHIP.G3		CHIP.G4		CHIP.G5		CHIP.G6		Open Graded	
Region	N	(km)	N	(km)	N	(km)	N	(km)	N	(km)	N	(km)	N	(km)
Auckland	197	(37)	–	–	57	(38)	53	(14)	–	–	–	–	909	(530)
Calibration	–	–	–	–	485	(1)	60	(0)	39	(0)	–	–	–	–
Christchurch	85	(20)	–	–	67	(58)	28	(12)	61	(63)	9	(6)	70	(36)
Northland	93	(46)	410	(831)	739	(1165)	54	(51)	174	(378)	8	(39)	23	(13)
Otago	87	(45)	100	(183)	825	(1650)	150	(281)	329	(810)	30	(108)	47	(30)
Wanganui	60	(12)	261	(421)	541	(796)	59	(59)	227	(392)	–	–	14	(8)
Wellington	169	(50)	68	(75)	701	(1112)	47	(36)	39	(25)	59	(47)	424	(584)
TOTAL	691	(212)	839	(1509)	3415	(4819)	451	(452)	869	(1668)	106	(200)	1487	(1201)

Note: Total N = 7274 (10060 km) exclusive of calibration data.
Chip.Gx – chipseal with grade 2 to 6.

5.1.2 Variable analysis

The influence of the predictor variables, 'Region', 'Type', and 'NELV', on the dependent variable MPD, was initially investigated by comparing variable analysis observations of MPD across all predictors (Figure 5.1). It is clear that surface 'Type' (the middle plot) has some clear effect on MPD, while 'Region' (the left plot) has little effect on the MPD (excluding Auckland and Christchurch). The logarithm of NELV appears to have some linear relationship with a negative slope.

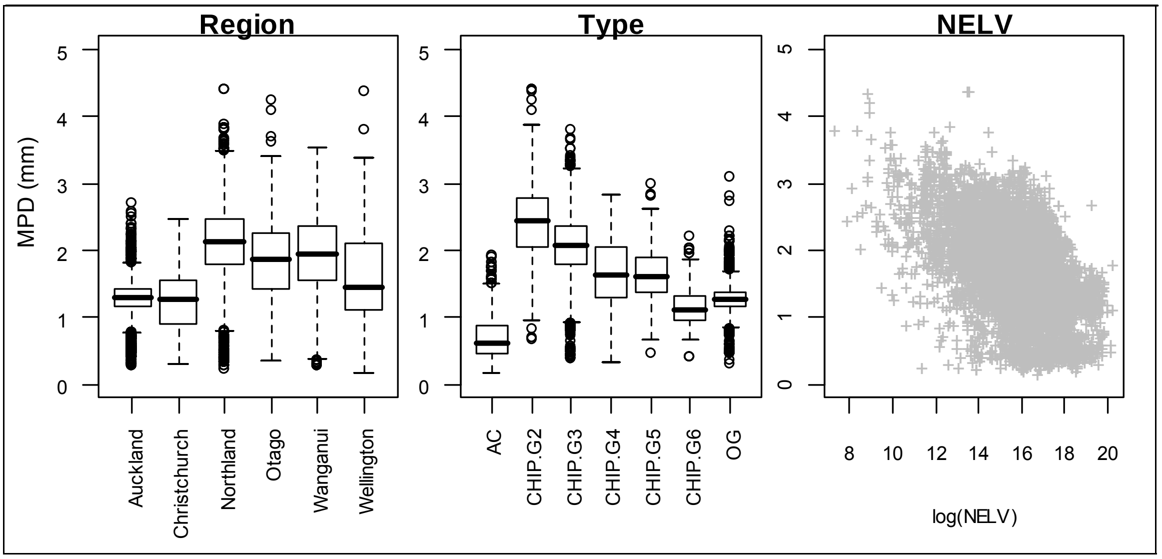


Figure 5.1 Un-standardised plots of predictor variables versus MPD.

A conditional plot of the data is also given in Appendix C, and clearly shows an approximate linear relationship within each plot window (where significant data exist). Further, this plot indicates that the slope in each window is somewhat different for each condition. For example, the slope decreases from Chip.G2 to Chip.G6. We investigate these relationships further with the use of standardised statistical analysis.

5.1.3 Standardised analysis

A stepwise regression was performed on the data, including all second order interaction effects, to establish the form of the model and estimate the effects of each predictor variable. Results and diagnostics are given in Appendix C. The final model form chosen by the stepwise regression process was:

$$\mathrm{MPD} = \log(\mathrm{NELV}) + \mathrm{Type} + \mathrm{Region} + \log(\mathrm{NELV}) : \mathrm{Type} + \mathrm{Type} : \mathrm{Region} - 1 \quad \text{(Equation 5.1)}$$

where log (NELV) : Type and Type : Region are interaction effects between the explanatory variables. Additionally, we have excluded a global intercept term (indicated by the '-1' term) because of the interaction effects. The selection of Type : Region as an interaction effect is thought to be misleading, as it is due predominantly to the lack of data in some cells more than to any real effect. Further, dropping this interaction term resulted in small reduction in the R^2 value. (It should be noted here that the high R^2 values are somewhat misleading, as because of the model setup they do not indicate residual model fit and, rather, can only be used as a comparison between models). The final model was therefore chosen as:

$$MPD = a_0 + a_1 \times \log(NELV) \quad \text{(Equation 5.2)}$$

where: a_0 and a_1 are chip size specific coefficients

Full model details, including fitted values and diagnostic plots, are given in Appendix C. In the above model, the first term on the right-hand side can be described as the slope and the last term as the intercept for each type. The final model fit to the data, including 97.5% prediction levels, are depicted in Figure 5.2. Of particular interest are the wide prediction levels, indicating large randomness in the observed data (i.e. a low traditional R^2 value). This may indicate a need for a more probabilistic approach to the application of the model in practice.

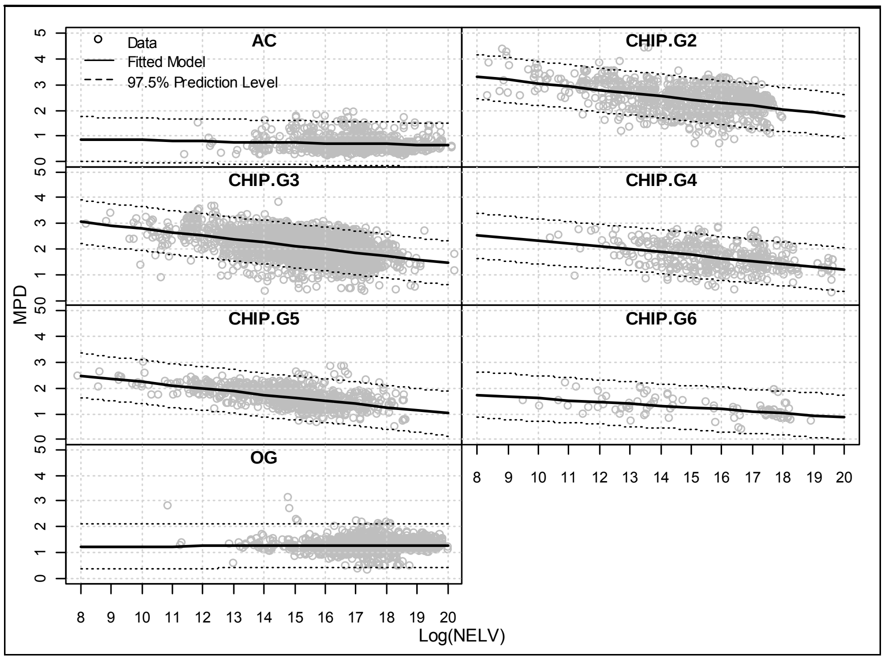


Figure 5.2 Plots of fitted model compared to observed data for each seal type.

Note: The proposed model is only applicable to chipseal pavements since the behaviour of asphalt (in particular porous asphalt) is completely different. For example, as indicated in Figure 5.2, the surface texture of porous asphalt (OG) is increasing with time (caused by chip loss) and not decreasing as found with chipseal.

5.2 Testing the texture model on the basis of LTPP data

In this section the original data are compared with the calibration data (Table 5.1). One way to achieve this is to see how our estimated model, based on the original data, predicts the calibration data. Figure 5.3 shows this. It can be seen that the fitted model generally contains the calibration data within a 97.5% prediction interval. It is interesting to see that the original data displays a much greater spread along the x-axis, i.e. log (NELV) values.

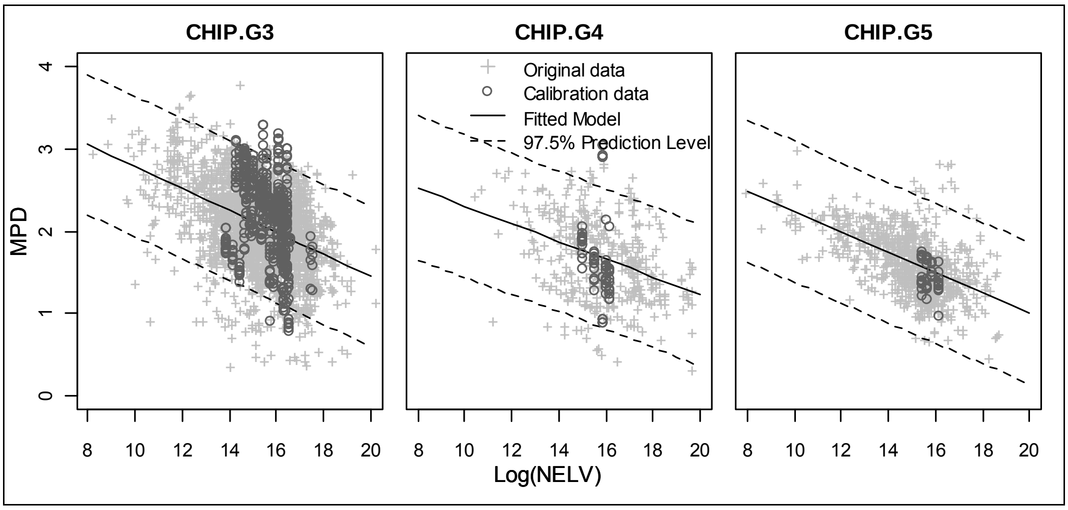


Figure 5.3 Plots of the fitted model (based on the original data) compared to the calibration data.

From Figure 5.3 the conclusion is that the fitted model based on the original data is a reasonable representation of texture deterioration (MPD). As a final model, the recommendation is that the model be re-fitted with all the data, with both the original and calibration included. This is presented in Appendix C.

As indicated, the texture model gives reasonable results, and no significant change is suggested to the model. However, a concern still exists regarding the accelerated phase of texture loss (flushing). It is recommended that this phase of texture loss is investigated further.

6. Rutting

6.1 Rut depth measurements and data

The Transverse Profiler Beam (TPB, see Figure 2.2) is used to measure the transverse profile of the road. From this profile a 2-m straight-edge simulation analysis is used to determine the rut depth. Henning et al. (2004a) demonstrated that this method of measurement satisfies the data accuracy requirements for rut depth model calibration. Figure 6.1 compares the incremental rut change between the three survey periods. This figure contains all the rutting measurements except for those sections where rehabilitation was performed, or where the field notes suggested faulty measurements. The specified accuracy of the rut measurements is approximately 0.5 mm/year. The average incremental rut change is 0.47 mm/year for 380 observations (Standard Deviation = 1 and the changes ranged from −2.5 to 6.4 mm/year).

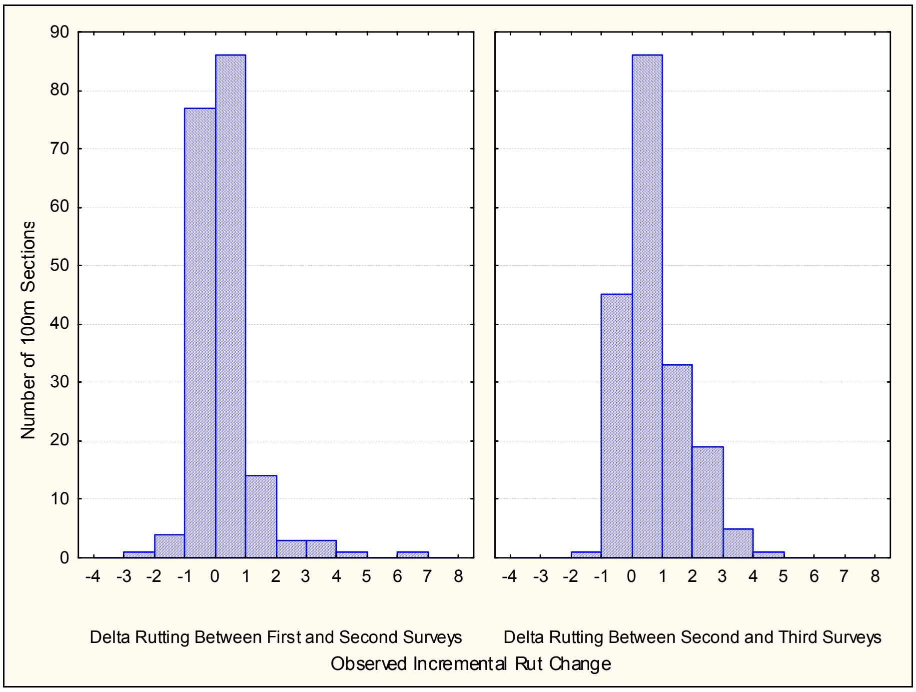


Figure 6.1 Distribution of incremental rut change for first three survey rounds.

Observations from this figure include:

- For both survey periods, most of incremental rutting change is within 1 mm/year.
- The respective distribution of rut change suggested more deterioration observed between the second and third survey rounds than between the first two survey rounds. One explanation for this trend could be that a number of resurfacing projects were carried out between the first and second survey rounds. Most of the field notes indicated the re-orientation of chips was a reason for rut depth improvement between the first two survey rounds.

It has been established from further investigations that once a rut change of 5 mm has occurred, the pavement appears to be in a rapid deterioration phase, as all the LTPP sections with high rut change rates indicated in the figure were rehabilitated subsequent to the surveys.

6.2 Calibrating the existing HDM rutting models

The HDM-4 format of the rutting model has been adopted within the asset management system in New Zealand. However, no conclusive studies have been conducted to demonstrate that the HDM-4 model is more applicable to New Zealand conditions than the HDM-III format. For that reason, both model formats were calibrated according to the LTPP data.

6.2.1 HDM-III rut progression calibration results

The calibration of the rutting model was performed according to the method described in Section 3.3.3. The results of the calibration are summarised in Table 6.1, and graphically represented in Figures 6.2 to 6.4.

Table 6.1 Summary of rutting calibration results.

Sensitivity Risk Area[1]	Rut Progression Calibration Coefficient (Krp)	Error Function[2] (RMSE)[3]
Low and limited	0.87	23,333 (30,746)
Medium and high	0.81	729 (1,346)
All data	0.84	22,583 (32,092)

Notes:

[1] The quantity of data was not sufficient (i.e. not enough data points) to perform successful calibration on individual sensitivity risk areas

[2] The value in brackets indicate the error function result using the default calibration coefficient (Krp=1)

[3] RMSE = root mean square error = square root of the difference between predicted and actual

The resulting calibration coefficients do not support the expected regional differences between the limited/low and medium/high sensitivity risk areas – the expectation being that the calibration coefficients for low/limited should be smaller than for the medium/high sensitivity risk areas (i.e. slower rut progression in more stable areas). However, this trend could be the result of a factor other than climate, e.g. the average pavement ages were observed as 17 and 28 years for medium/high and limited/low sensitivity risk areas respectively. A rut progression calibration coefficient (Krp) of 0.84, based on the full dataset, is therefore recommended for the state highways.

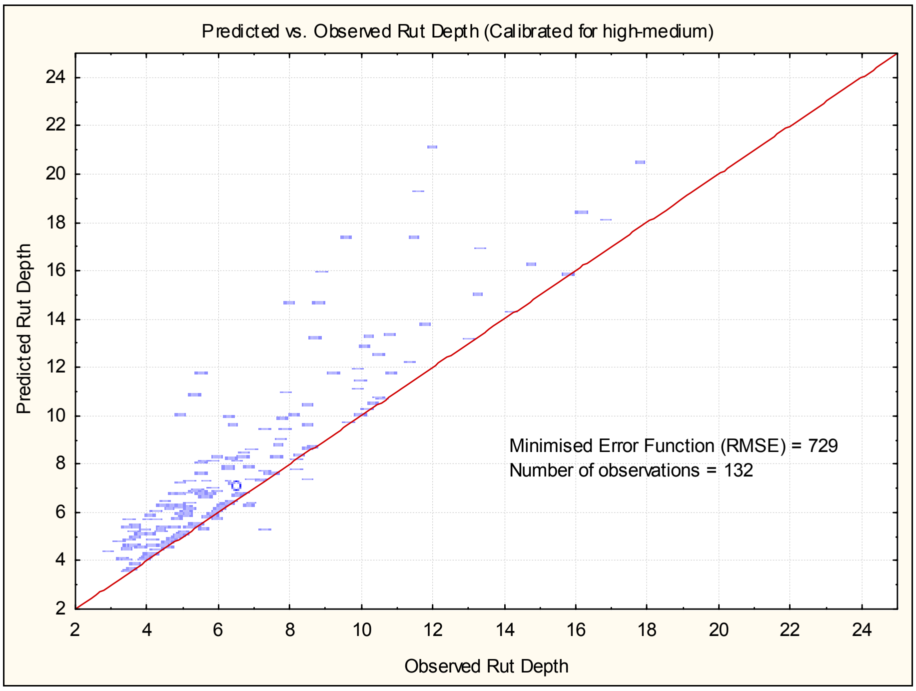


Figure 6.2 Comparing the observed and predicted rut depth for the calibrated model (high and medium sensitivity areas).

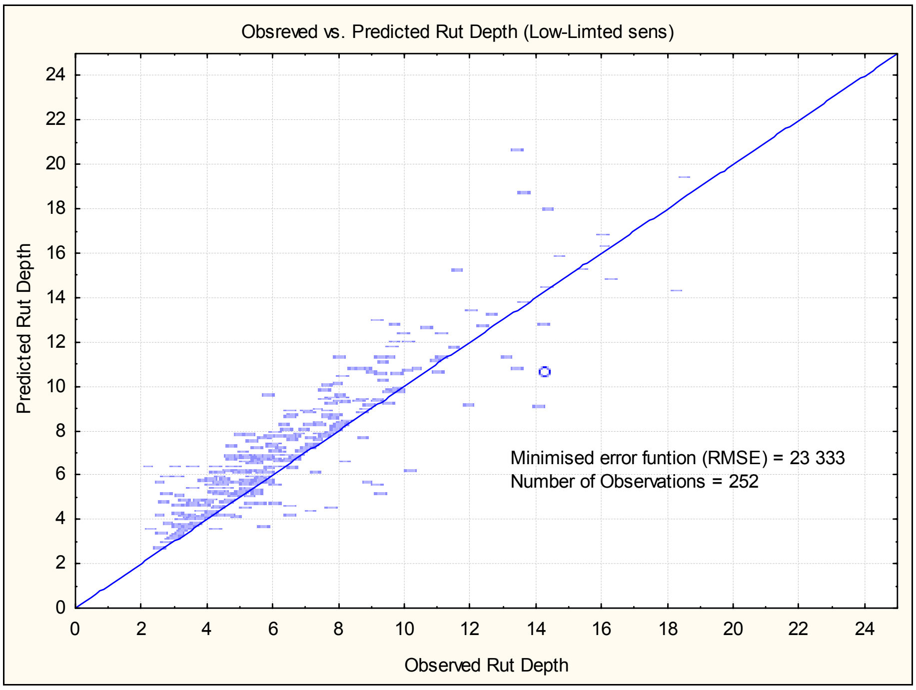


Figure 6.3 Comparing the observed and predicted rut depth for the calibrated model (limited and low sensitivity areas).

Figures 6.2 and 6.3 depict the comparison between predicted and observed rut depths. The limited and low sensitivity risk areas have a better fit compared to the medium and high sensitivityareas.

On both these graphs the correlation between the predicted and the actual data is within the expected range (approximately ± 2 mm). However, if the predicted and actual incremental rutting change is compared, then the correlation seems to be much worse. For example, Figure 6.4 illustrates similar data to Figure 6.3, but only shows the incremental change in rutting from one year to another. Even though the calibration yielded acceptable results, the recommendation is to review the rutting model format in order to establish a better correlation between the model and actual pavement behaviour.

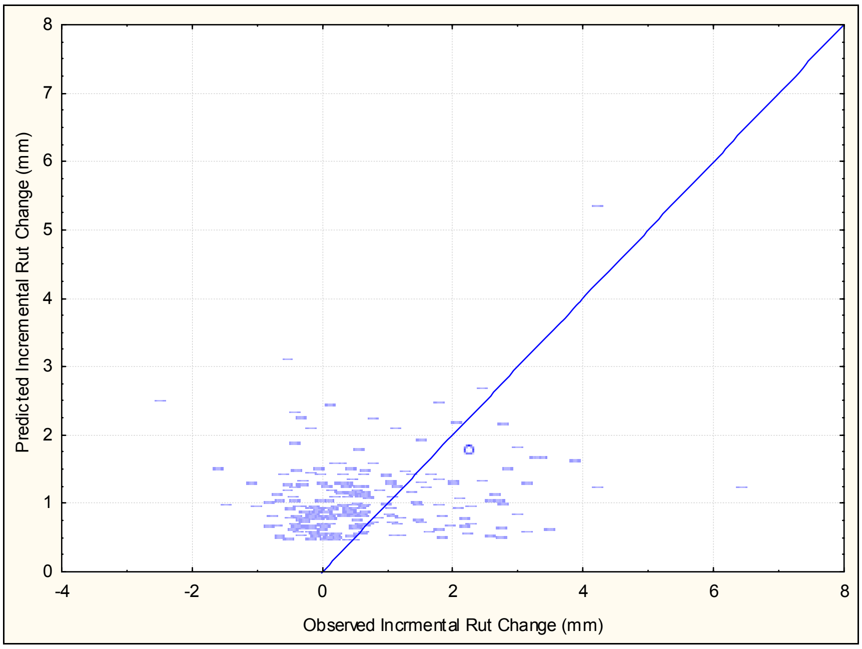


Figure 6.4 Comparing the observed and predicted rut depth incremental change for the calibrated model (limited and low sensitivity areas).

6.2.2 HDM-4 Rut progression calibration results

Table 6.2 lists the resulting calibration coefficients for the rut progression model as defined in HDM-4. It suggests that the default model closely resembles the actual behaviour of pavements in New Zealand, since the required calibration coefficients are close to the default value of 1. In addition, the error function is small in comparison with the outcome from the HDM-III rut model. This can be largely explained by a more normal distribution of observed and predicted points centred around the line of equality (Figure 6.5).

Table 6.2 Summary of rutting calibration results HDM-4.

Sensitivity Risk Area[1]	Rut Progression Calibration Coefficient (Krp)	Error Function[2] (RMSE)[3]-
Low and Limited	1.03	2,719 (2,729)
Medium and High	0.98	931 (933)
All Data	1.01	3,658 (3662)

Notes:

[1] The quantity of data was not sufficient (i.e. not enough data points) to perform successful calibration on individual sensitivity risk areas

[2] The value in brackets indicate the error function result using the default calibration coefficient (Krp=1)

[3] RMSE = root mean square error = square root of the difference between predicted and actual

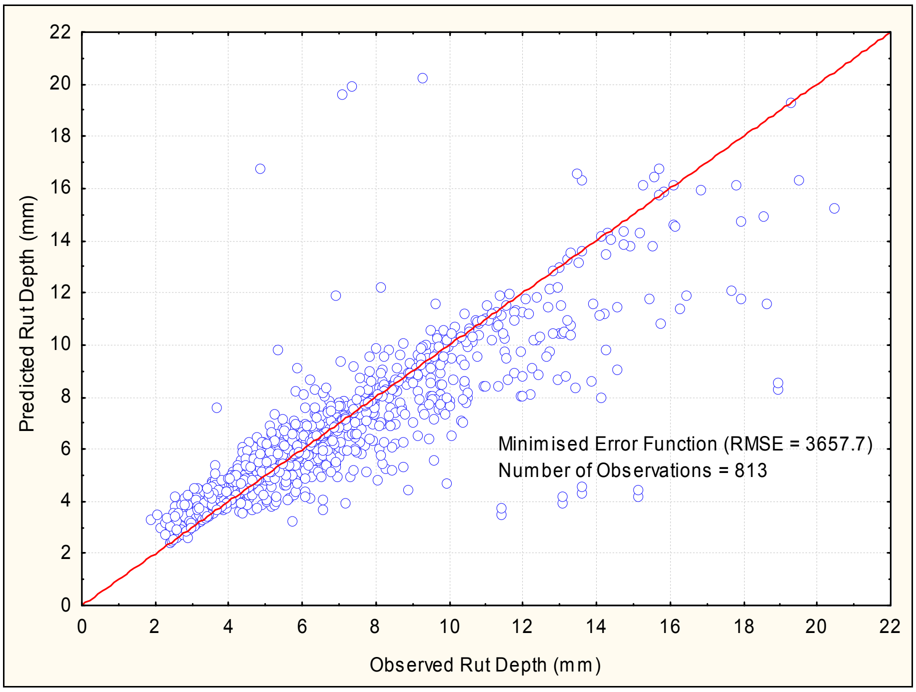


Figure 6.5 Comparing the observed and predicted rut depth for the calibrated HDM-4 model (all data points).

Figure 6.6 depicts the predicted and observed incremental rut change. This figure suggests that the predicted incremental change varies very little around the average prediction of about 0.3 mm to 0.4 mm per year.

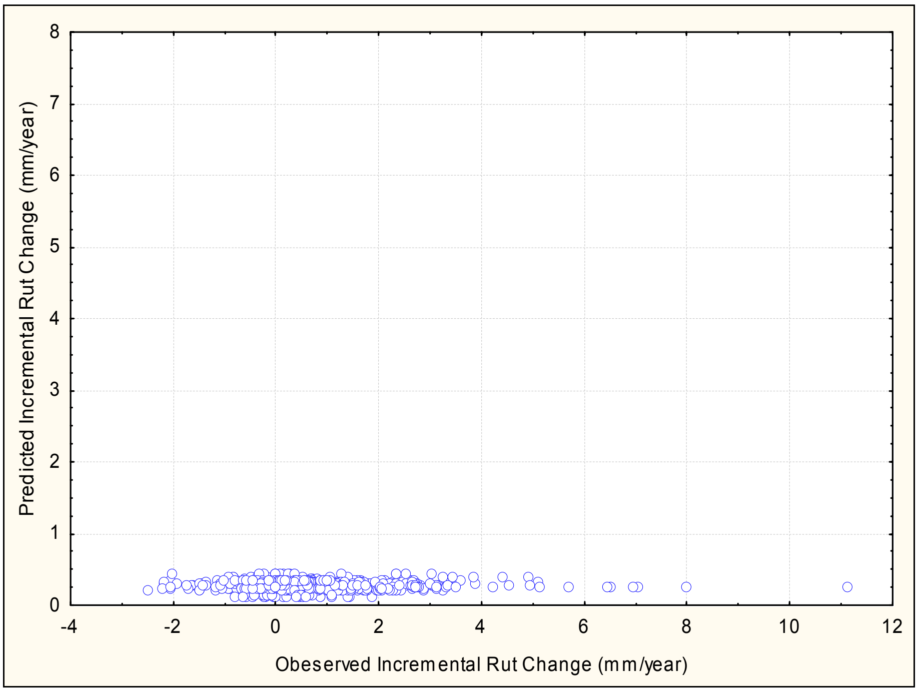


Figure 6.6 Comparing the observed and predicted rut depth incremental change for the calibrated HDM-4 model (all data).

6.3 Refining the rut progression model

6.3.1 Description of the data

Four years of surveyed rutting data have been used for the analysis presented in this section. Because of accuracy requirements, only LTPP data were used for the analysis (see Section 2.3). Figure 6.7 illustrates the incremental rut change measured. Survey numbers indicated in the figure represent the delta rut between two surveys (e.g. 2 is the incremental rut difference between survey 1 and 2).

The figure depicts a growing rut change for each survey. Note that all negative rut changes, corresponding with rehabilitations, were excluded from the analysis. Note that, although the rutting has increased for the last survey period, the outliers in the data have decreased significantly. Although the equipment used has remained the same, some improvements were made to make the equipment more stable during the surveys.

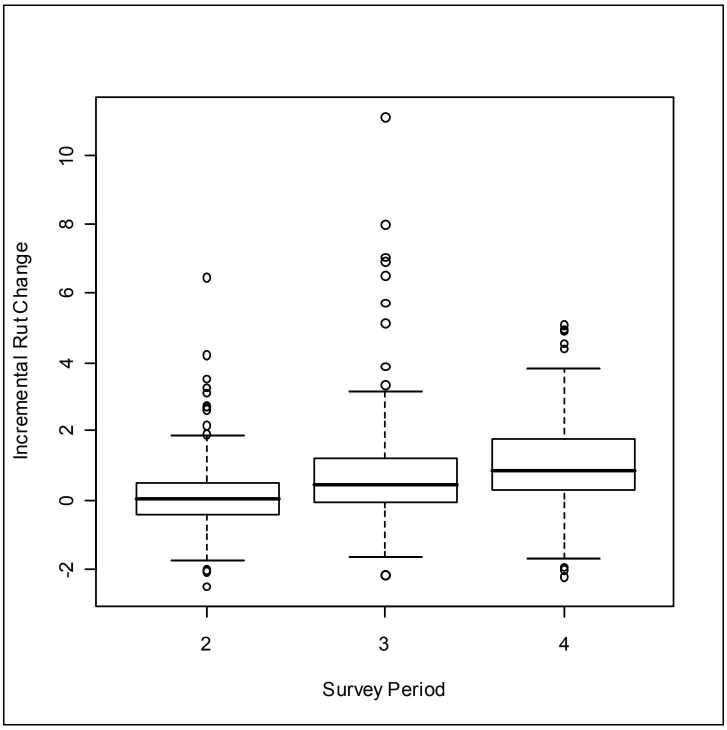


Figure 6.7 Incremental rut change.

6.3.2 Variable analysis

The variables considered during the analysis in the following sections are shown in Table 6.3.

Table 6.3 Variables considered for predicting rut progression.

Variable	Description	Variable Type
AADT	Annual average daily traffic	Continuous
YE4	Annual number of equivalent standard axles (millions/lane)	Continuous
SNP	Structural number of the pavement	Continuous
MMP	Mean monthly precipitation	Continuous
wpiri	Wheel path IRI (mm/km)	Continuous
HTOT	Total surface thickness (mm) of all the layers	Continuous
Sens	Climatic sensitivity area	Factor
AGE3	Age of the pavement (years)	Continuous
AGE 2	Age of the surface	Continuous
OTCI	Time to crack initiation	Continuous
Stat.crx	Cracked status	Binary
D0-D9	FWD-deflections for given geophones (micro mm)	Continuous
maxdef	FWD-maximum deflection from all geophones (micro mm)	Continuous
SF1 & SF2	FWD- deflection shape Factor 1 and 2	Continuous
SCI	FWD- surface curvature index	Continuous
BCI	FWD- base curvature index	Continuous
BDI	FWD- base damage index	Continuous

In terms of the stratified data, not much difference was observed between the average incremental rutting for different climatic regions and for the cracked status of the pavement (see Figure 6.8).

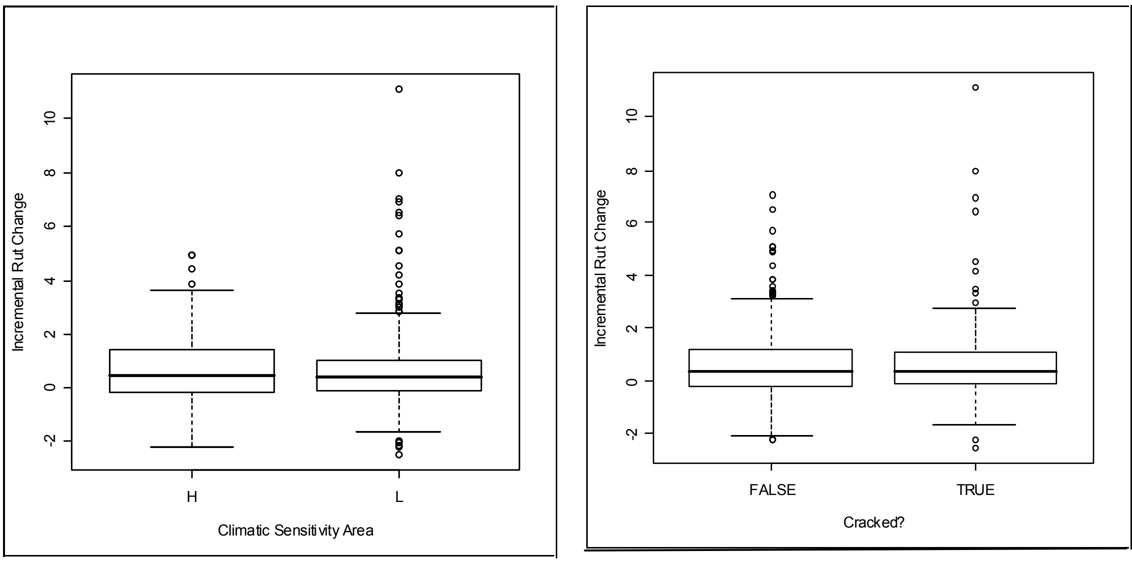


Figure 6.8 Incremental rut change for stratified data.

However, the figures show different distributions for both the stratification according to the climatic area and the cracked status. Cracked pavements (right plot) would be expected to have incremental rut changes that are much higher than the average, typically associated with pavements in a rapid failure mode. Also, pavements in the higher sensitivity climatic area (left plot) would be expected to have a larger spread in the data, and a higher variation in pavement performance in wetter climates. But as indicated in previous sections, these pavements are younger compared with the pavements in the lower sensitivity climatic area, thus resulting in some extreme rut progression in the low sensitivity area. It was observed though, that the rainfall may have a relationship with the incremental rut change (Figure 6.9). The rut change is increasing with increased annual rainfall.

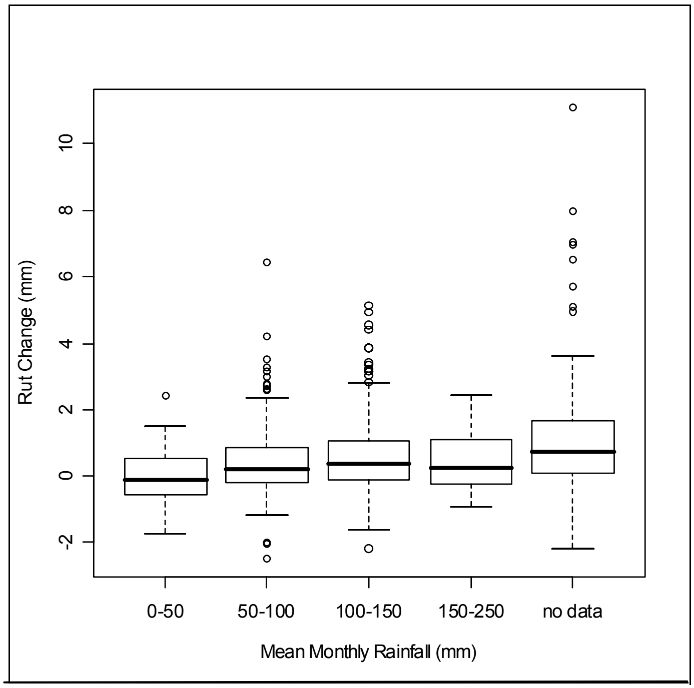


Figure 6.9 Incremental rut change (mm/year) as a function of rainfall (mm/year).

During the investigation of the interrelationship of variables and the predictor very few distinct trends were found. Figure 6.10 shows two examples of some of the most significant interrelationships observed. On the left plot the incremental rutting is plotted as a function of the total traffic loading with the data stratified for surface and pavement age. No clear relationship can be observed except that, for older pavements and older surface ages, a non-linear relationship seems to exist between the incremental rut depth and the traffic loading.

The right plot depicts the incremental rut change as a function of the total surface thickness for different levels of the total traffic loading. For this example, the only possible trend is observed for the higher traffic volume category. This plot suggests a possible logarithmic relationship between the total surface thickness and the rut depth change. It should, however, be noted that this trend is highly influenced by one specific section.

In summary, no conclusive trends were observed for the variable analysis.

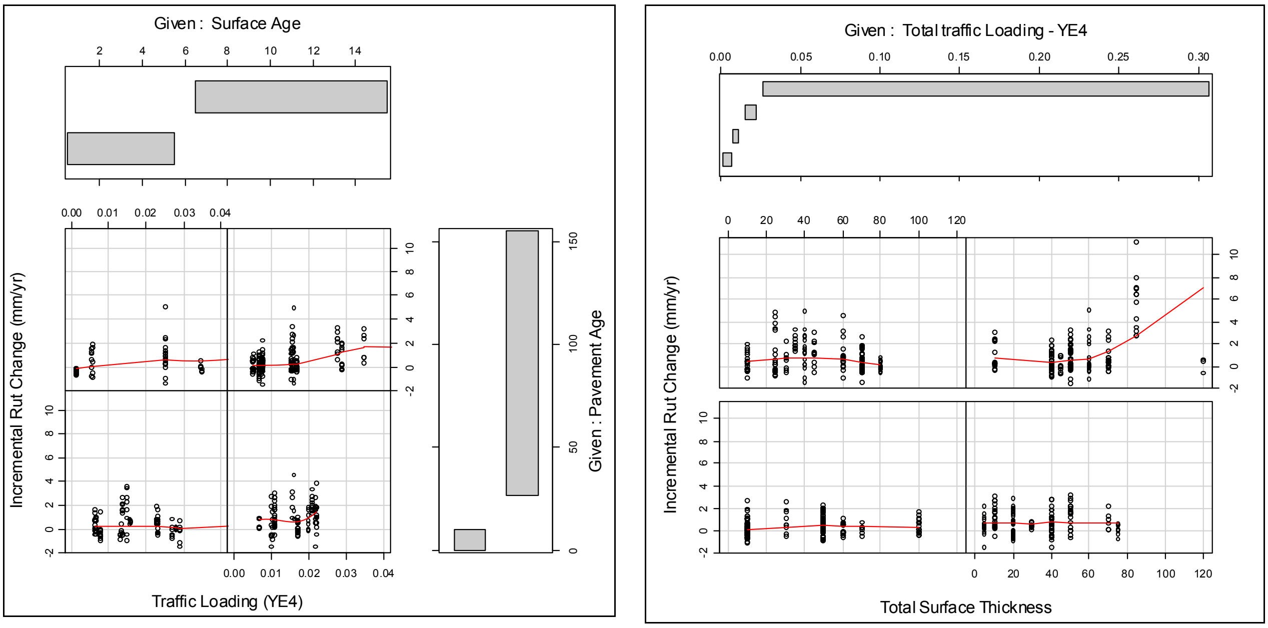


Figure 6.10 Testing the interrelationships of variables affecting rut change.

6.3.3 Linear model (LM) regression analysis results

A stepwise regression was performed according to the same approach as explained in Section 4.2.5. Table 6.4 lists each step of the regression process and its effect on the AIC (Akaike's Information Criterion). The AIC is like a fault term with the lower values indicating better fit with the observed data. See also Appendix D (Figure D.1) for the resulting model coefficients and model diagnostic plots. The resulting model has an overall R^2 of 0.4 and most variables included to the regression were found to be significant.

The table also indicates that most variables included in the model are able to improve the AIC, thus suggesting that the model is significantly contributing towards explaining the independent variable (rut progression). This phenomenon is typical of a trend that has no distinct variable/s explaining it. Also, it is observed that multiplication of variables significantly drops the AIC. Each step, including a multiplication, reduces the AIC more than any individual variable. This confirms the model approach as suggested in HDM-4 (see Section 3.3.3.1).

Table 6.4 ANOVA (analysis of variance) results from linear model regression for rut progression.

Step	Step Factor	Deviance	Residuals Deviation	AIC
1	intercept		1153.4	338.0
2	mmp	54.06	1099.3	305.2
3	ye4	39.02	1060.3	281.0
4	snp	20.60	1039.7	268.7
5	ye4:snp	110.06	929.6	189.5
6	age3	10.61	919.0	183.1
7	ye4:age3	29.57	889.4	161.4
8	sf2	6.71	882.7	157.9
9	ye4:sf2	89.06	793.7	82.7
10	age3:sf2	19.23	774.4	66.9
11	age2	9.47	765.0	60.0
12	ye4:age2	4.88	760.1	57.3
13	adt	5.81	754.3	53.7
14	sci	5.83	748.5	50.1
15	age3:sci	15.73	732.7	36.7
16	ye4:sci	4.47	728.3	34.2
17	sf2:sci	7.84	720.4	28.4
18	bci	16.93	703.5	13.1
19	age2:bci	14.12	689.4	0.4
20	ye4:bci	6.52	682.8	−4.5
21	sci:bci	7.87	675.0	−10.9
22	age2:sci	5.54	669.4	−14.9
23	ye4:sf2	1.11	670.5	−15.7
24	sf1	2.26	668.3	−16.1
25	age2:sf1	6.43	661.8	−21.2
26	(-)age2:sci	0.21	662.1	−22.9

Figure 6.11 illustrates the difference between the predicted and observed rut depth for the resulting model. This figure looks similar to Figure 6.4 in the sense that it has a cluster of data concentrated around the average of the predictor, and beyond that point, very little predictive power. This means that if we want to predict the incremental rut change, we may just assume it to be equal to the average, say 0.3 mm per year. Although an average rut progression rate does not incorporate changing variables, this model simplification may well be adopted within the New Zealand modelling approach. It is important though, to confirm this progression rate with more data.

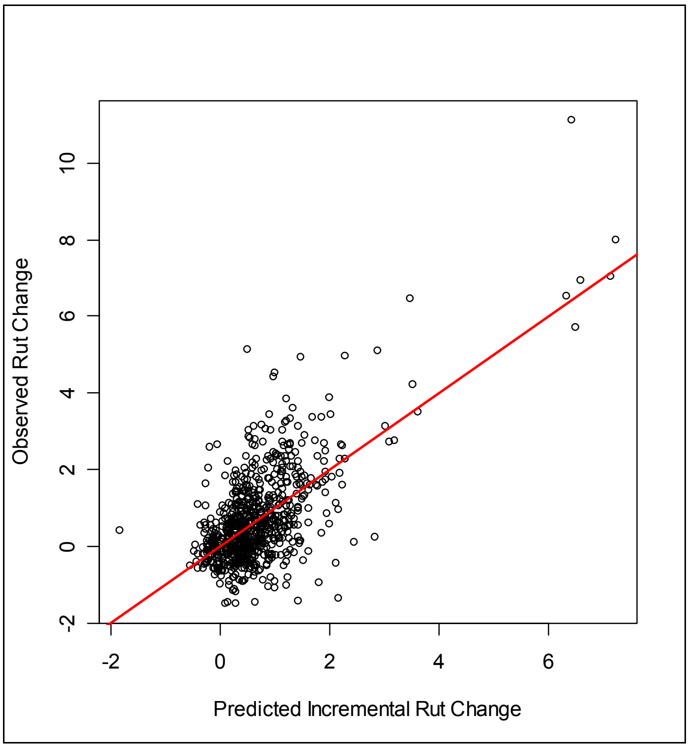


Figure 6.11 Comparing the predicted and observed incremental rut change.

6.3.4 Future development requirements for rut progression model

- Investigate rut progression model forms on the CAPTIF data. Two major trends must be investigated:
  - Determine the factors contributing towards stable rut progression observed during most of the pavement life. In particular see if the CAPTIF data confirms the constant 0.3-mm rut progression of pavements.
  - Determine the factors contributing towards a pavement that starts accelerated deterioration including a rapid rut progression.
- Further tracking of rutting on individual sections over time. Since the LTPP data are becoming available for longer time periods, this analysis would become more useful. Also, by combining trends from the LTPP data and the CAPTIF data, it may be possible to develop model formats from basic principles.

6.4 Rut depth standard deviation

The standard deviation of rut depth (rds) is used in the roughness model. At present, it is calculated from the mean total rut depth (rdm) and has two forms, the HDM model (NDLI 1995):

$$rds = \max\left[0.3, \left(0.9 - 0.04rdm\right)\right] rdm \qquad \text{(Equation 6.1)}$$

and the New Zealand model (HTC 1999).

$$rds = 0.21rdm + 0.69 \qquad \text{(Equation 6.2)}$$

Using the test data, each of these models is reviewed, and a new one is proposed.

Figure 6.12 shows the relationship between rdm and rds. It is clear that a log transform of both variables (the right-hand plot) displays a more normally distributed linear trend. This is also intuitively obvious, as rds is strictly positive and skewed towards the right (likewise for rdm). It should be noted here that none of the existing models utilise this basic property of variance.

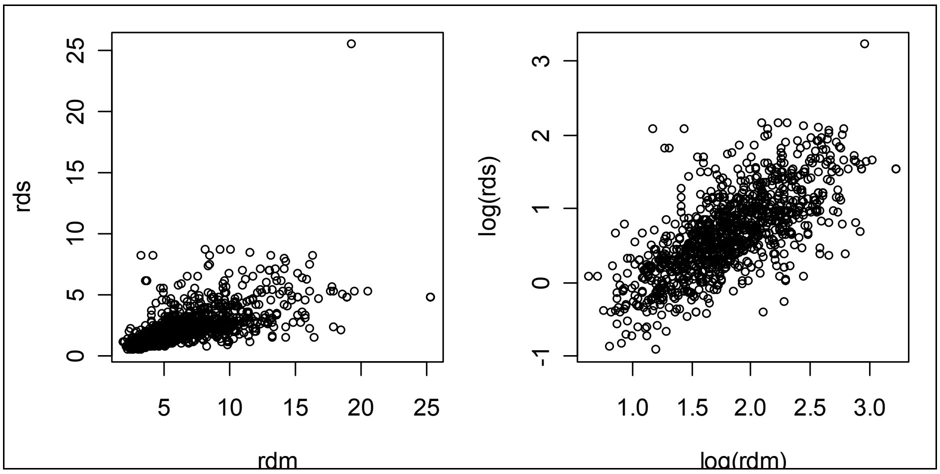


Figure 6.12 Plots showing the relationship between rdm and rds. (The plot on the right-hand side displays a log transform of both variables.)

The basic model developed in this report uses the log transforms of rds and rdm (referred to as Model 1). Using standard linear regression the fitted model was found to be:

$$\log(rds) = 0.8804\log(rdm) - 0.9369 \qquad R^2 = 0.49$$
$$rds = \exp\left[0.8804\log(rdm) - 0.9369\right]$$
(Equations 6.3 and 6.4)

with model diagnostics given in Appendix D.

Figure 6.13 shows the model fit to the observed data (left-hand plot) and a comparison of all three models (right-hand plot). It is clear that the HDM.RDS model is not appropriate for New Zealand use. Comparing the other two models, the NZ.RDS shows an increasing linear trend, while Model 1 shows an increasing concave function. Further, Model 1 decays to zero (i.e. the y intercept) while the NZ.RDS model shows a positive y intercept, which is clearly impossible. Another feature of the new model is that the variance structure is multiplicative as opposed to additive. Therefore, as RDM increases, so does the variance around RDS. This suggests that any deterministic prediction of the RDS based on large values of RDM is somewhat questionable, and should possibly be based on a more probabilistic approach.

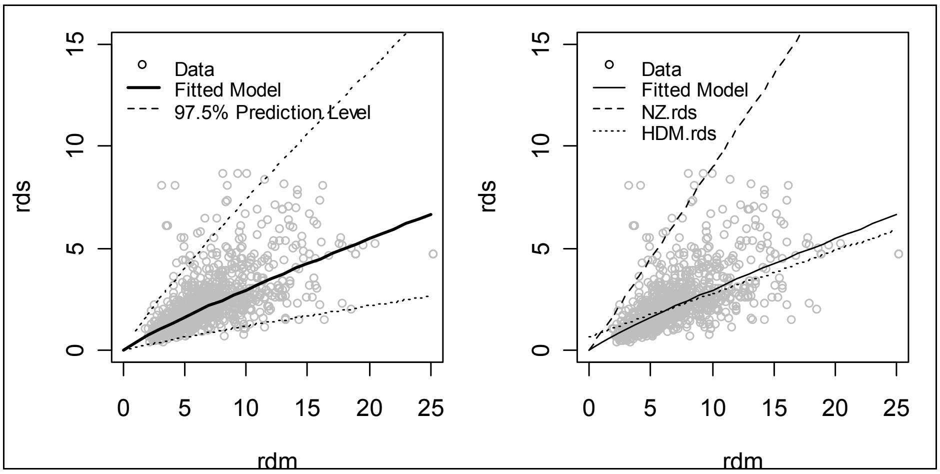


Figure 6.13 Plots of model fit and comparison between models.

7. Roughness

7.1 LTPP roughness data

The HMD-4 roughness model format was developed with the underlying philosophy that the roughness will always increase incrementally, regardless of how lightly the pavement is loaded or how minor the environmental effects may be. This trend could not be supported by the actual data, and as a result the calibration of the roughness model was not possible. Figure 7.1 illustrates the distribution of incremental roughness change between the three surveys. This figure suggests that hardly any change in roughness occurs from one year to another for most of the sections.

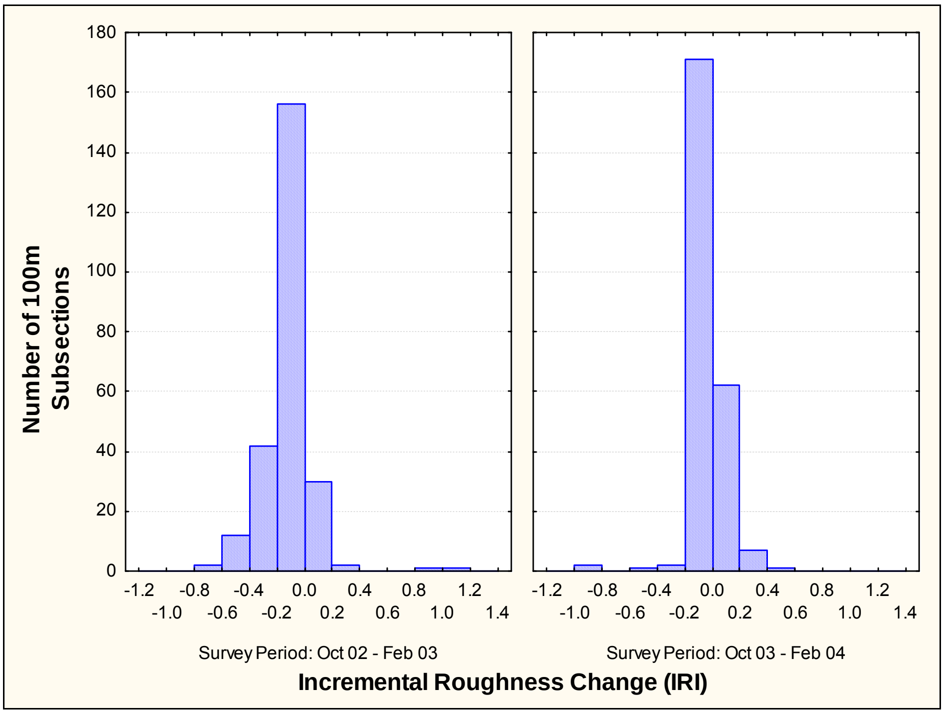


Figure 7.1 Distribution of incremental roughness.

It is further noted that the distribution of the roughness change between the 2002 and 2003 survey rounds is wider than between the 2003 and 2004 survey rounds. According to the site notes, many of the negative changes (i.e. roughness improvements) for the 2002 to 2003 survey periods could be explained by texture loss on newly resurfaced roads, and flushing on older pavements. These effects were less pronounced in the 2003 to 2004 survey rounds because of less resurfacing in that period.

Given that the accuracy of the roughness measurements is specified at ± 0.2 IRI (Transit 2001, Henning et al. 2004a), any measurements within this tolerance do not necessarily indicate a trend towards improvement or deterioration. Referring to Figure 3.3, this would suggest that, for most of the measurements, the roughness can be assumed not to have changed at all. Where section data indicated a definite change in roughness, this was usually explained by the survey field notes. For example, where the roughness improved, obvious signs of flushing or texture loss were present. Sections that showed clear indications of roughness deterioration were in a rapid deterioration phase, and most of these sections were rehabilitated shortly following the surveys.

Given the above, it was not surprising that any attempt to calibrate the existing HDM model did not yield satisfactory results, from which it was concluded that the roughness model format probably needed adjustment as discussed in Section 7.2.

7.2 A review of the roughness model

7.2.1 Environmental component

It is recognised that roughness sometimes increases because of environmental effects only. This is very applicable to New Zealand conditions, given the geological makeup. Even if no traffic travels on certain roads, the roughness will increase. A good example of in-situ material where this phenomenon is common is the peat areas in the Waikato. However, in most cases the total roughness progression is very low, and one would therefore expect the environmental component of the deterioration to be even smaller.

Little work has been undertaken in New Zealand in order to understand the environmental impact on roughness. This research would take a very long time, and understanding some of the more significant variables of the model first would be useful before such a study is undertaken.

7.2.2 Structural component

The structural component of the roughness model is of main interest for this study. This model is of the following format (NDLI 1995):

$$\Delta RI_S = a_0 * \exp(mK_{gm}AGE3)(1+SNPK)^{-5}YE4 \qquad \text{(Equation 7.1)}$$

where:

ΔRI_s	structural component of roughness change
a_0	model coefficient
m	environmental coefficient
K_{gm}	environmental calibration coefficient
AGE3	pavement age
SNPK	structural number including crack influence
YE4	annual number of equivalent standard axles (millions/lane)

The influence of the strength component is illustrated in Figure 7.2. Also indicated in the figure is the relative influence of the strength component in the rutting model.

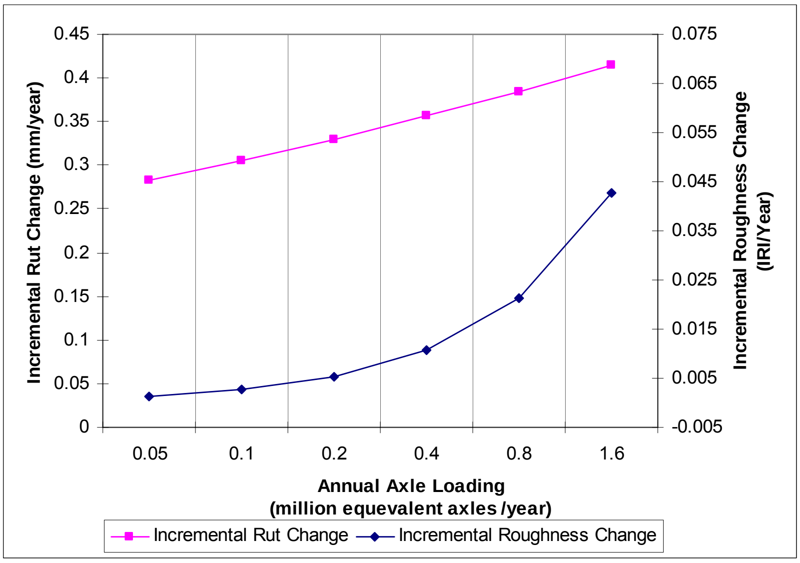


Figure 7.2 Comparing the influence of the strength component on roughness and rutting change (Note, in both cases a pavement age of 10 years was assumed).

Observations from this figure are:

- The strength component in the roughness model is significantly more sensitive for increasing traffic loading compared to the strength component in the rutting model. Although the scale is different for the two parameters, the figure shows an exponential growth of the roughness model because of the strength component;
- The incremental change for the roughness model is very low if compared with the expected accuracy achievable with roughness survey equipment. Calibrating according to such low incremental changes has proven to be problematic. Therefore, we are trying to predict annual changes which we cannot observe, even by using the most accurate instruments in the industry.

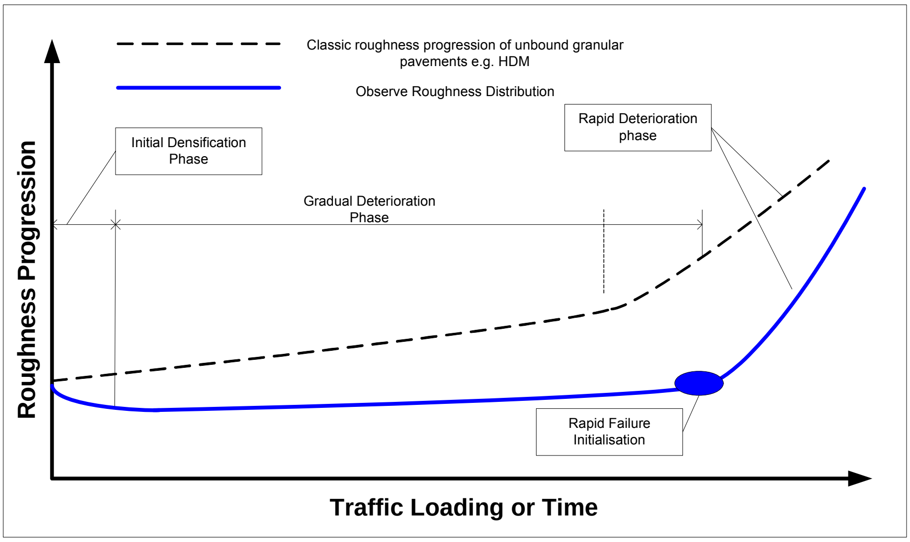
7.2.3 Incremental change caused by rutting

The interaction between rutting and roughness is well observed for network data in New Zealand. However, evidence of initial reduction in rutting variation exists during the early years of pavement life (see Section 6.3). The rutting, in combination with some macro texture effects, is therefore contributing towards an initial reduction of roughness, which is not recognised in the roughness model.

7.3 Proposed changes to the roughness model

Most network analysis performed in New Zealand has suggested difficulties in calibrating the roughness model, or in obtaining meaningful roughness prediction from the modelling system (MWH 2004). In most cases, very low calibration coefficients are used, and regardless of the maintenance quantities scheduled, the roughness deteriorates for the network.

Observing the LTPP data also suggests that the roughness progression is somewhat different from the HDM model. Figure 7.3 illustrates this difference diagrammatically.



Note: The figure is based on very limited data and is only applicable to chipseal pavements

Figure 7.3 Schematic comparison between the HDM roughness model and observed behaviour in New Zealand.

The HDM model would, in most circumstances, predict a continuous roughness progression, even if very low calibration coefficients are selected. The actual roughness data from the LTPP database suggests the following differences:

- Evidence exists of an initial reduction in roughness following construction and resurfacing. The significant reduction of roughness following resurfacing is believed to be caused by the re-orientation of chip, and is more prominently observed for Walking Profilometer measurements used on the LTPP sites (see Figure 7.4). This trend is less evident for HSD type measurements undertaken on the same sections. However, a reduction in roughness still occurs initially following construction. This trend correlates with the initial reduction in rut depth standard deviation.
- Following the initial 'settling-in' phase, the roughness remains unchanged for a very long time. For example, hardly any roughness change can be observed on low volume roads for a significant period of time. This will only change when the pavement reaches the end of its capacity life, the traffic changes significantly, or when water enters the pavement (Land Transport New Zealand 2005). After this the pavement goes into a rapid failure mode, which is characterised by an exponential roughness progression rate.

The model form described above and illustrated in the figure is not uncommon in the engineering industry. It is often referred to as the 'bath tub' model format, which signifies the performance of most mechanical components such as motor engines etc. Should this model format be appropriate for New Zealand, the focus of the model will change from 'how much does the roughness change in a year?' to 'when does the rapid deterioration start?' It is recognised that the classic HDM roughness model may still be valid for some pavement types and pavements constructed for higher volume roads.

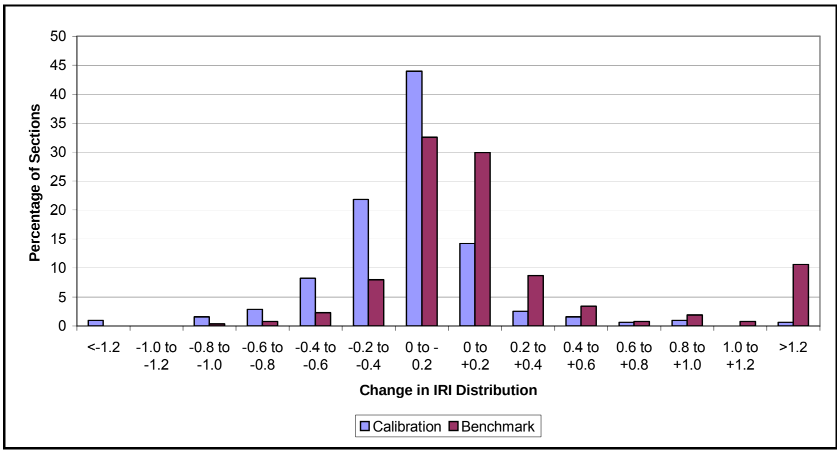


Figure 7.4 Comparing the incremental roughness change between LTPP data (calibration) and HSD data (benchmark) (Transit 2005).

During the past three years only two LTPP sections have progressed to this rapid deterioration stage. It will take at least five to ten years before the LTPP experiment will have sufficient data available to develop the proposed model format. However, by combining the LTPP data with the Transit CAPTIF (Canterbury Accelerated Pavement Testing Indoor Facility) data, it would be possible to investigate the merits of such a model further. A research study to do that has been approved for the 2005/06 financial year. For that reason, this report will not discuss the roughness model any further.

8. Recommendations

8.1 The future of the New Zealand LTPP sections

This report has demonstrated that the level of data collection accuracy is appropriate for calibration and pavement model development. Some intuitive trends with some models (such as roughness) have been confirmed for the first time since the appropriate level of data accuracy existed. However, this study has also highlighted the need for further model development based on an individual section data. It has been suggested that some of the models such as roughness and rutting could be developed utilising some CAPTIF data, but ultimately could only be confirmed with the LTPP data.

Furthermore, the report has highlighted the significance of the rapid failure stages of pavement deterioration modelling. To understand the behaviour during rapid deterioration better, more data during this stage are needed.

The recommendation is therefore that the LTPP surveys continue on both the Transit and local authority networks. It is difficult to put a time period on the required survey duration, but the period estimated is at least another five years.

In terms of the current data collection precision and accuracy requirements, the recommendation is that the current standard be maintained.

8.2 Investigate the rapid failure stage for roughness and rutting progression

As a next stage to this research, a proposal was accepted for the 2005/06 Land Transport New Zealand Research Programme. This study is aimed at linking both the LTPP and the CAPTIF programmes. The objectives of this research are explained in the extract from the research proposal:

> *Linking the outputs from CAPTIF with the LTPP study is the next logical step towards building on the understanding of pavement performance/ deterioration under New Zealand conditions. Comparing field performance (LTPP) with the accelerated load performance (CAPTIF) will significantly increase the confidence in outputs from both these programmes. The specific objectives of the study include:*
>
> 1. *Calibration coefficients and new model formats have been developed based on the Transit LTPP data (Transfund Research Programme 04/05). The first objective would be to confirm and improve the model format and results based on existing CAPTIF data;*
> 2. *To develop relative performance factors for different treatments and material types, similar to the work completed in Australia (Martin et al 2004). It should be appreciated that the data from both*

programmes will greatly extend the range of applicability of both programmes;

3. *To gain a better understanding of the environmental impact on pavements. The LTPP sections are subjected to normal climatic influences whereas the CAPTIF testing was conducted under controlled conditions. It is therefore possible to investigate the specific environmental impacts on pavement performance, something which is relatively complex to do based on LTPP work alone; and*
4. *Confirm CAPTIF life cycle and mass limit study results with the LTPP performance data.*[5]

An obvious emphasis of this study would be to increase the understanding of the deterioration of the pavement during the rapid failure stages.

8.3 Introducing uncertainty and statistical distribution in some models

According to some network experience combined with engineering observations, some of the pavement behaviour cannot be explained according to deterministic model formats. For example, this report has suggested a generalised linear model format for predicting cracking. Likewise, defects such as potholing and failures (shoving) would be easy to fit according to some statistical distribution such as a Proportional Intensity Model.

This change in modelling approach would not be transferable, similar to the HDM approach. However, it would produce modelling outcomes that more closely resemble actual behaviour in New Zealand.

It is recommended that this development is undertaken as part of the dTIMS CT development consortium tasks, since it will optimise the input from all practitioners in New Zealand.

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[5] Research Proposal: *Benchmarking pavement performance between Transit's LTPP & CAPTIF programmes* 05/06 Research Round. MWH New Zealand Ltd.

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Appendix A Paper on the establishment of the LTPP programmes

Long-term Pavement Performance (LTPP) Studies in New Zealand – Lessons, the Challenges and the Way Ahead

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SYNOPSIS

New Zealand embarked on a Long-Term Pavement Performance (LTPP) programme with the establishment of 63 LTPP sites on the State Highway network during 2000. In 2003, this programme was expanded to include more than 21 local road controlling agencies by instituting a further 82 LTPP sites. By including rural sites and low volume rural roads in the study, the LTPP programme now covers a wide spectrum of pavement construction, traffic composition and climatic zones experienced in NZ.

Before commencement in New Zealand, earlier LTPP studies undertaken overseas were reviewed, namely: World Bank Highway Design and Maintenance Standards (HDM-III) study in Brazil, the Strategic Highway Research Programme (SHRP) in the USA and International Study of Highway Development and Management (ISOHDM) calibration studies performed in both Australia and South Africa. This paper describes the many cases where new principles and ideas have been applied to address the specific challenges presented by the New Zealand roading environment. As an example, a performance based survey contract was developed for this project in order to achieve the accuracy requirements for the data collection.

The value of reviewing international 'best' practice was most useful in developing the specific New Zealand LTPP study objectives, which were:

1. establish a representative sample of LTPP sections across New Zealand considering the combined effects of climate and sub-soil moisture sensitivity;
2. enhance the existing data collection being undertaken on a network level;
3. collect data to a precision level suitable to perform incremental model calibration and to adjust model formats if required.

For the state highway study, a third round of surveys has been completed. Hence, it is now possible to assess the philosophy adopted for this New Zealand programme. This paper reviews the data collection methodology, relative to the original study objectives, assumptions and specifications. The statistical characteristics recorded for the design matrix factors for the state highway LTPP sections has indicated that the full range of expected values on the New Zealand network are being monitored. The statistics also show that there is a reasonably even distribution for most variables, thus not creating a bias towards the mean value. Furthermore, it is concluded that an appropriate level of data collection accuracy has been adopted for the New Zealand programme.

At this early stage, the LTPP New Zealand programme has been successful in achieving the goals and objectives. On the other hand, a review of the data has highlighted areas where further research is required and proceeding – for example further work is required to confirm that all the main drivers that contribute to deterioration in the New Zealand environment are currently being recorded as part of the LTPP programme. The next major stage of the programme is to investigate the model format and determine the calibration requirements.

INTRODUCTION

Background to Pavement Deterioration Modelling in New Zealand

During 1998, New Zealand significantly enhanced its asset management approach through the incorporation of predictive capabilities to forecast long-term pavement maintenance needs. A system consisting of an optimisation analysis process (dTIMS) was combined with the World Bank Highway Development and Management (ISOHDM) pavement deterioration models. The system has been adopted by Transit New Zealand and most of the 74 territorial road-controlling authorities throughout the country.

Since initiation, it was realised that in order to achieve better correlation between the predictive models and actual pavement deterioration, it would be necessary to enhance current desk-top calibration of the models with actual long-term pavement performance studies (LTPP). In this regard, two LTPP studies are underway in New Zealand:

- Transit New Zealand established 63 sections on the state highways across the North and South Islands; and,
- Approximately 21 road controlling authorities have established a total of 82 sections in both urban and rural areas.

The Transit sections were surveyed for the first time in 2000. The survey of road controlling authority sections commenced in 2003.

Transfund New Zealand, the funding agency for roading in New Zealand, is co-ordinating and managing the LTPP study for road controlling authorities.

This paper documents the process of the experimental layout and planning of the LTPP studies which has been largely based on similar international studies.

Objectives of this Paper

The main objectives of this paper are to outline and discuss the resulting experimental layout and data collection regime adopted. Secondly, to discuss the results from the data collected to date. This paper also gives an outline of the comprehensive literature review which was undertaken prior to the study.

LTPP studies are a long-term investment therefore it is important to determine the appropriateness and relevance of the data at an early stage of the research project. Such concerns are discussed in this paper in an attempt to answer the following questions:

- Is the sample size sufficient?
- Are all factors (independent variables) sufficiently covered with the experimental layout (design matrix)?
- Are the expected range for each factor measured? and,
- Is the data collection methodology appropriate and is the precision of measurements sufficiently accurate?

SUMMARY OF INTERNATIONAL CALIBRATION STUDIES

Scope of Literature Review

A literature review was undertaken on those LTPP studies which closely resembled the intended work in New Zealand. From the outset, the intention was to use the original HDM pavement models (Paterson, 1987) as a basis for the LTPP programme. This prompted the need to review the original HDM studies (GEIPOT, 1981) along with known calibration studies performed elsewhere to calibrate World Bank models, for example in South Africa and Australia (Rohde, et. al, 1998 and Tepper and Martin, 1999).

In addition some non-HDM studies were reviewed, particularly in relation to the experimental design matrix and data collection regime for example USA and Australia (FHWA, 2000 and Martin, 1994).

Original Studies for HDM-III (Brazil)

The World Bank HDM-III model is comprised of three different levels: a life-cycle maintenance analysis philosophy, pavement deterioration / work effects / road user cost models and lastly the analysis software. The Brazil study (GEIPOT, et. al, 1981) mainly focused on the development of the pavement and road user cost models.

The primary interest in this Brazil study was the design matrix used for the site establishment, especially the road characterisation in the matrix. The technical team conducting this study was faced with similar objectives and constraints that most countries face during the planning stage of the experiment such as:

- The data had to be sufficient for statistical analyses but the study scope was limited by the total number of sections and the measurements to be taken; and,
- It was difficult to populate all the cells of a design matrix, such as, strong pavements on low volume roads.

The resulting design matrix for the Brazil study is depicted in Figure 1.

SURFACING TYPE / BASE TYPE / TRAFFIC (ADT)			ASPHALTIC CONCRETE				DOUBLE SURFACE TREATMENT			
			GRAVEL		CRUSHED STONE		GRAVEL		CRUSHED STONE	
STATE REHAB.	AGE (YEARS)	VERTICAL GEOMETRY (%)	50-500	>1000	50-500	>1000	50-500	>1000	50-500	>1000
OVERLAID	≥ 6	≥ 6	128	129						
		0 - 1,5 %				158		109 009		
	0 - 2	≥ 6		125			035	032		
		0 - 1,5 %		006			034	031		159
AS CONSTRUCTED	≥ 12	≥ 6		119			123 172	110 008		
		0 - 1,5 %		003 113		168	173	007		121
	0 - 4	≥ 6	022	025	151	162	002	024 111	155	103 165
		0 - 1,5 %	001 021	033 112	152	161 026	004 023	106	101	102

THE NUMBERS IN EACH CELL ARE THE SECTION NUMBERS.

Figure 1: Design Matrix for the Paved Road HDM-III Study in Brazil (GEIPOT, et. al, 1981)

The numbers in the cells of the design matrix represent the section identification numbers and the study was conducted on a total of 65 test sections. Some pertinent observations obtained from the study included:

- The number of sections identified for the study 65, was the absolute minimum in order to perform reliable statistical analyses;
- The selected sections included in-service pavements and some specifically constructed pavements - the latter were constructed under controlled conditions;
- The design matrix included a factor for the maintenance regime - some of the pavements were subjected to normal maintenance while others only underwent emergency routine maintenance.

One aspect, which has been adopted in recent calibration studies including the New Zealand study, is the crack rating method. According to the HDM-III definition, cracks are measured linearly or according to the area, depending on crack type, and expressed as an area of the section affected by the cracking. The area effected by a linear crack is calculated by multiplying the length with a standard width of 0.5m.

Strategic Highway Research Programme (SHRP) LTPP study - USA

The SHRP study (FHWA, 2000) is undoubtedly the most elaborate study of its kind ever conducted. Started in 1989, this LTPP programme is planned to observe pavements for a period of 20 years across the USA and Canada. As the SHRP study had a much wider scope compared to the intended studies in New Zealand, only relevant aspects of it were reviewed. Relevant to New Zealand was the long-term monitoring of in-service pavements. The objective for this study component of the SHRP program was to investigate the effect of loading, environment, material properties, construction quality and maintenance levels on the performance of pavements. Two aspects considered from the SHRP program, included the approach followed for the experimental design and the planning and documentation of data collection.

Of specific interest was the review undertaken on the actual sections established versus the original design matrix. This approach was extended by Benson (1991) who has developed a technique to evaluate the success of the LTPP site establishment relative to the original design matrix and study objectives. For this review an effectiveness ratio was developed to test the variances of the expected versus the achieved variances of the sampling pattern. The effectiveness ratio is defined as:

$$\text{Effectiveness} = 100 \times \frac{\text{Desired Variance}}{\text{Median Variance Achieved}}$$

This effectiveness ratio has also been used in the identification of the priority cells (i.e. those cells which would be harder to populate) in the design matrix during the site establishment. Some of the Benson's (1991) conclusions were:

- For cases where the factorial design was not balanced, regional model development must be considered due to the concern over the inference space;
- In cases where pavements with similar failure mechanisms exist, data for these sections could be combined in order to achieve better balances of factors in the design matrix;
- Regional data collection operations had to be reviewed in order to identify and illuminate sources of bias;
- Distributions have to be investigated to check for non-normality, bi-modalism and extreme cases.

Another important aspect of the SHRP-LTPP study (SHRP-LTPP, 1990 and 1999) was the detail to which data collection regimes and methods were specified. Appendix A summarises the pavement monitoring conducted for the study.

LTPP Studies in Australia

Two LTPP studies conducted in Australia were reviewed:

- A study into the development of new pavement deterioration models (Martin, 1994) - this Australian Road Research Board (ARRB) study commenced during 1990 and one of the main focuses was to assist in the calculation of road agency expenditure (road track cost);
- A second study (Tepper and Martin, 1999) was the calibration of HDM–4 models - this study, also done by ARRB, included eight LTPP sections in Victoria, Queensland, New South Wales and Tasmania.

The first study was aimed at a strategic level and for this reason, network level type data collection was used (namely, high speed data) as opposed to a more detailed level as proposed for HDM calibration studies. This approach resulted in the affordability of a higher number of sections, 150 sections were surveyed.

The second study was specifically designed to calibrate HDM-4 models. The study layout was also planned to focus on the work effects models. The work effects models predict the influence of different maintenance options on the immediate and long-term performance of the pavements. For this reason, the experiment consisted of constructing sections of roads with different maintenance treatments - an example of the sections layout is given in Figure 2.

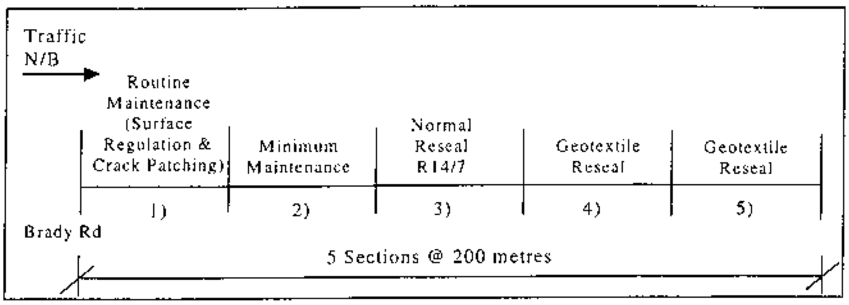

Figure 2: Example of Maintenance Treatments Applied on Australia's LTPP Study Sections (Tepper and Martin, 1999)

A falling weight deflectometer (FWD) was used for all strength measurements and the ARRB Multi-laser profilometer was used for measuring the longitudinal and transversal profiles. All the sections were tested prior to and on completion of the maintenance treatments. Subsequent measurements were also undertaken on an annual basis. Appendix B illustrates the section characteristics used in this ARRB HDM-4 study.

South Africa (Gautrans) HDM-III and HDM-4 Calibration Studies

The Gauteng Provincial Government road department (Gautrans) initiated a LTPP study in 1993, which consisted of 36 sections (Rohde et al, 1998). A review of this study has been particularly useful for the New Zealand projects since it had similar objectives and it had been conducted over the last 10 years.

Particular aspects of technical interest were:

- The design matrix that included factors such as traffic, base types, environments and pavement condition;
- Each site was 500m long and divided into 50m subsections;
- The data collection regime on the sections consisted of test pit and FWD measurements to quantify the pavement characteristics and a combination of manual rating, high speed data (HSD) roughness and manual rutting measurements to determine the condition data.

Appendix C illustrates the range of the parameters from the Gautrans LTPP sections.

Ten years later, Rohde et al (2002) reviewed the calibration results and concluded the following:

- The original HDM-4 cracking model was satisfactory in its current form;
- The rutting model format required improvement;
- The roughness model performed satisfactorily
- Overall, the pavement condition distribution predicted, correlated well with the network trends and, therefore, was satisfactory for the PMS application

In summary the results from the study suggested that the level of data accuracy was appropriate for the HDM model calibration coefficient adjustment. However, if a model format needed to be reviewed, then data collection at a higher level of accuracy would be required.

Summary of the LTPP Studies Review

Below is a summary of the main aspects gained from the review of the international LTPP studies outlined in the previous sections.

Table 1: Relevant Issues from International LTPP Studies

Study	Relevant Items	New Zealand Context/Recommendation
HDM-III – Brazil	The design matrix is simple and incorporates only major factors effecting pavement deterioration.	A similar approach applicable to New Zealand.
	Specific pavements were built to monitor certain material types.	The New Zealand programme is to monitor in-service pavement deterioration
	On sterilised sections only limited maintenance to ensure safety.	A similar approach applicable to New Zealand.
	A well defined measure of visual crack detection was used	A similar approach applicable to New Zealand.
	Traffic monitoring was done at a high level	A similar approach applicable to New Zealand.
SHRP-USA	The scope of the study was vast.	The New Zealand programme is on a much smaller scale.
	Multiple objectives were addressed with the study.	Main objective is to investigate pavement performance.
	Experimental design only considers major pavement deterioration factors.	A similar approach applicable to New Zealand.
	An un-weighted stratified sampling method was used rather than a random method. The aim is to incorporate extreme points.	A similar approach applicable to New Zealand.
	An effectiveness ratio was developed to test the applicability of chosen sites, relative to the original design matrix.	A similar approach applicable to New Zealand.
HDM-4 Australia	Different maintenance treatments were tested within one section to determine work study effects.	The New Zealand programme is to monitor in-service pavement deterioration
	Experimental layout effectively isolates certain factors on the design matrix e.g. everything else is the same while maintenance treatment is varied.	The New Zealand programme is to monitor in-service pavement deterioration.
	Mix of different LTPP site types were used –e.g. specially built and in-service pavements.	Good benchmarking for in-service and specific type pavements.
HDM-III and HDM-4 Gautrans SA	Experimental design only considers major pavement deterioration factors.	A similar approach applicable to New Zealand.
	The site layout included 500m long sections divided into 50m sub-sections	A similar approach to be adopted in New Zealand. However a shorter total length will be used.
	Visual Rating to include all HDM distresses.	A similar approach applicable to New Zealand.
	Data collection precision was sufficient for Level II calibration.	Level III precision will be required to allow for model format adjustment.
	Sterilised sites did not get any maintenance.	A similar approach applicable to New Zealand.
	In-service pavement were monitored over an extended period	A similar approach applicable to New Zealand

LAYOUT OF THE NEW ZEALAND LTPP STUDIES

Objectives for the New Zealand LTPP Programme

In all the international LTPP studies reviewed, it was evident that specific factors, that ranged from technical to environmental, were specifically addressed by the study objectives. A similar methodological approach was taken in the New Zealand LTPP studies – below is a table showing the factors and resulting main study objectives.

Table2: New Zealand LTPP Factors and Resulting Objectives

Topic	Factors Considered	Resulting Objective
Climate and Geology	New Zealand has a diverse climate (rainfall) and geological features. Both rainfall and the geological makeup affect the behaviour of pavements. Some soils are sensitive to variation in rainfall (e.g. winter summer differences), while other soils are comparatively stable even if the rainfall varies throughout the year.	**Objective 1:** ***A representative sample of LTPP sections have to be established across New Zealand***. This objective also implies a careful consideration of the classifying areas according to the combined effect of climate/rainfall and the moisture sensitivity of the soils.
New Zealand Maintenance Practice	Authorities apply micro maintenance management on pavements. For example, treatments are applied on relatively short section lengths (e.g. 200 to 500 m) plus maintenance decisions are taken based on relatively accurate condition data information (e.g. 100% condition assessment using HSD technology).	***Objective 2: Condition measurements should reflect or enhance the accuracy requirements of maintenance planning (for example HSD).***
Data Accuracy	The pavement modelling system utilises the incremental format of the HDM models, therefore the change in condition from one year to another is predicted rather than predicting the absolute condition status based on the value during the original construction date. Data collection accuracy had to be consistent with the annual change of each condition parameter. For example, if a rut change of say 0.5 mm per year is expected, the survey error and variation have to be consistent within this predicted range. Some limited experience in using the HDM models for New Zealand roads suggested that the calibration of the models may require some model adjustment rather than simply adjusting the calibration coefficients. According to the HDM-4 requirements (Bennett and Paterson, 2002), model adjustments require a higher level of accuracy in the measurements	***Objective 3: All condition measurements must be recorded to an accuracy that will allow for model form adjustment and calibration based on incremental pavement deterioration.***

In addition to these main objectives, there were a number of other objectives and goals of the New Zealand study – however, for this paper they have been omitted and are given in Henning and Hart (2000).

Experimental Design

Regional Classification

Most of the international LTPP studies have used regional classification according to climatic/rainfall factors. For these studies, indexes such as Thornthwaite Index and the Weinert N value were used to classify regions according to precipitation and evaporation characteristics (Weinert, 1987 and Thornthwaite, 1954)

For New Zealand regions, a combined factor for climate and the geological makeup was required. Cenek (2001) proposed an index, which is a ratio of the subgrade strength over the sub-soil moisture content. This index is an indication of the soil sensitivity as a function of variance in rainfall and the wet strength characteristics of the soil. The Cenek (2001) expression is:

$$\frac{\text{Subgarde Strength}}{\text{Moisture Indicator}} = \left[\text{Log}_{10}\left(\frac{\text{Wet Strength}^{3}}{\text{Moisture}^{0.5}}\right)\right]^{1.5}$$

The wet strength is a ranking from 1 to 100 of the soil wet-strength in the New Zealand Soil Classification. The moisture has been derived from the National Institute of Water and Atmospheric Research (NIWA) soil moisture deficit data. According to Cenek (2001), there is a direct relationship between the moisture deficit index and Thornthwaite Index. The wet strength is a good predictor of subgrade strength to moisture susceptibility, since this reflects the worse case for pavement failure.

Considering this approach, New Zealand was divided into three and four sensitivity areas for the Transfund and Transit studies respectively. These sensitivity areas were not necessarily neighbouring geographical areas - for example, most of northern part of the Northern Island (Northland) and the west coast of the South Island, have similar sensitivity numbers and were, therefore, classified as one calibration area.

Pavement Loading (Traffic)

The majority of New Zealand rural roads are relatively light trafficked in comparison to many other countries. Only the major urban highways and arterial routes have traffic volumes exceeding an annual average daily traffic (AADT) of more than 10,000 veh/day. For this reason, the LTPP study design matrix on the state highways used three pavement loading classes including (Henning, 2000):

- Less than 100 Equivalent Standard Axles (ESA) per day;
- Between 100 and 200 ESA's; and
- Greater than 300 ESA's per day.

For the local authority LTPP study, only two traffic classes were used, with 200 ESA's/day being the traffic limit between high and low volumes.

Pavement Type/Strength

No provision was made in this LTPP study for separate surfacing categories. According to New Zealand pavement maintenance strategies, only pavements with higher pavement loadings are surfaced with asphalt concrete (AC), which automatically ensured representative sampling of chip seals and AC surfaced pavements.

In order to achieve a better representation of the variability of pavement behaviour, the pavement type/strength has been considered in combination with the pavement loading. This method ensured that relatively weak pavements carrying high pavement loads and relatively strong pavements carrying lighter pavement loads were included.

As no strength data, such as FWD, existed prior to the site selection process, the pavement composition was used as a predictor of the strength. The strength was classified as shown in Table 3.

Table 3: Strength Classification Used for the New Zealand LTPP Studies

Study	Weak Pavements	Strong Pavements
Transit State Highways	Unbounded chip seal with total pavement shallower than 300mm	Unbounded chip seal with total pavement deeper than 300mm or, (Asphaltic surfaced pavements)
Local Authorities	Unbounded chip seal with total pavement shallower than 250mm	Unbounded chip seal with total pavement deeper than 250mm or, Asphaltic surfaced pavements

The strength classification system depicted in Table 3 needed to be validated by actual strength tests. Some results of the validation are presented later.

Pavement Condition/Age

In order to have continuous distribution of pavement deterioration, both the age and the condition were considered. Care was taken not to only consider the pavement age in the selection process in order to prevent bias towards superior performing pavements.

Resulting Design Matrix

The resulting design matrix parameters for the New Zealand LTPP studies are illustrated in Table 4.

Table 4: Design Matrix Parameters for the New Zealand LTPP Studies

Factor	Transit State Highways	Local Authorities
Environments (Sensitivity Areas)	4	3
Traffic Classes	3	2
Pavement Types/Strength	2	2
Pavement Age/Condition	2	2
Urban/Rural	N/A	2
Maintenance Regime (with or without maintenance)	N/A	2
Total Number of Cells (Sections established)	**48 (63)**	**96 (82)**

For the local authority study, not all the required number of sections have been established. Once sufficient data becomes available, the remaining sections will be established in order to populate the empty cells in the design matrix.

The state highway study includes five additional sections in order to provide a contingency for possibly having to omit some of the chosen sections that have to be rehabilitated earlier than expected. For example, to date, three sections have been rehabilitated since 2000.

For the New Zealand programme, Cenek, et. al. (2003) proposed a relationship between the number of calibration sections required in relation to the number of years of the monitoring programme. According to this relationship, model coefficient adjustment can be expected to have an accuracy of ± 20% provided there are at least:

- 120 sections being monitored for two years;
- 60 sections being monitored for five years.

Therefore, it is expected, that the New Zealand programme, currently 125 sections, will produce sufficient data to initiate the calibration process within two to three years of data collection. The duration of the study is undetermined but it is expected to be at least ten years.

Section Selection Criteria

It was decided to adopt most of the geometric and condition homogeneity criteria as specified in the HDM-4 Calibration Guidelines (Bennett and Paterson, 2000) and New Zealand Calibration Guidelines (Henning and Riley, 2000). These criteria included:

- avoiding steep gradients;
- disallowing sag curves;
- excluding sections with major drainage structures;
- restricting the total surface thickness.

The major advantage of adopting the above was consistency for the data collection. However, by excluding highway sections according to the above criteria, it is not known whether the resulting models to be developed will be appropriate for these sections.

Data Collection Regime

Survey Accuracy and Intervals Requirements

In accordance with the study objectives, the data collection needed to be undertaken with an accuracy that would allow calibration of the models (namely, adjusting calibration coefficients) in the short term as well as providing for adjusting the model format at a later stage. In order to achieve both of these, a number of factors were considered, including:

- The precision output of the various data collection equipment;
- The referencing methodology of measurements – to ensure that consecutive surveys are performed at exactly the same location (within ± 100mm of the original location);
- The expected variation in data due to the change in the physical condition and in conducting the measurements itself.

Ultimately, results from the LTPP study will have to be applied to the network level pavement deterioration which is based on HSD measurements. Therefore, consistency in the approach between the two levels of data collection was imperative. For example, for Transit New Zealand, a standard wheel track spacing (distance between left and right laser beams of 1.75m) is used for all HSD roughness measurements. During the LTPP condition measurements a similar approach had to be followed.

Referencing Method and Section Layout

The minimum length of 300m for a LTPP section was based on the requirements for a longitudinal profile analysis (roughness calculation) as specified by Sayers and Karamihas (1996). For New Zealand highways, which have a typical curvilinear alignment, a longer minimum length would have been impractical, as it would have required compromise on the geometrical requirements mentioned earlier.

The 300m sections were divided into 50m subsections similar to the layout used for the Gautrans LTPP study (Rohde, et. al 1998). Both the start and end point of the LTPP sections were indicated with marker posts alongside the road; on the centreline of the road, a steel rod was driven flush with the road surface. In addition, the GPS co-ordinates of these positions were recorded. The combination of methods ensured the exact positioning of the start and end position, should physical marks be lost due to maintenance or resurfacing.

Measurement positions were temporarily marked using spray paint on the road surface and the location of these marks was recorded on site layout plans. The photograph in Figure 3 illustrates a typical layout of a LTPP section.

Figure 3: Photo of a LTPP Section on the State Highway

Condition Measurements

Once established, the pavement strength and pavement layer composition were determined for all LTPP sections using the FWD and test pits. A sample of the test pit material was subjected to typical soil indicator laboratory tests. On the state highways, FWD surveys are repeated on an annual basis.

Annual pavement condition measurements for both LTPP studies in New Zealand are conducted based on an outcome-based contract. Rather than specifying the equipment used for the surveys, the accuracy and the repeatability of the measurement are specified. The methodology used for the various measurements is summarised next. Further details are provided in Transit (2000) and more recently in Henning, et. al. (2004).

For the **roughness** measurements, equipment that met the World Bank Class 1 (Sayers and Karamihas, 1996) and ASTME950 requirements was specified. The contractor responsible for the surveys opted to use the ARRB Walking Profilometer

(WP). According to the repeatability requirement for the WP, three measurements are recorded in each wheel path.

Transversal profiles **(rutting)** measurement required equipment capable of recording the profile at 50mm spacings to a precision of 0.5mm. Each transversal profile measurement is repeated twice for each measuring position at a sampling interval of 10 m along the section. An auto-driven Transversal Profile Beam (TPB) is being used for both LTPP studies. This equipment records relative height differences of a measuring wheel with displacement instruments recording the offset and vertical distance.

For the state highway LTPP sections, **surface texture** measurements are undertaken using a stationary laser profilometer. Ten metre continuous profiles are recorded at a 50m sampling interval.

All **visual distresses** are recorded in terms of their type, the area/length affected and their location within the LTPP section length. The distress types plus the associated accuracy of measurement are specified in the survey specification document (Transit New Zealand, 2000).

TESTING THE APPROPRIATENESS OF LTPP SECTIONS AND DATA COLLECTED

Representative Sample Testing

At this time (when this paper was prepared), only data for the Transit LTPP study was available. On completion of the first survey round, the distribution of the design matrix variables were reviewed in order to test whether all cells were sufficiently represented. Table 5 illustrates the resulting descriptive statistics of the factors used in the design matrix.

Table 5: Descriptive Statistic for the State Highway LTPP Sections

	Mean	Median	Minimum	Maximum	Standard Deviation
ESAL's	387.6	198.8	14.2	3301.2	693.3
Pavement Age	20.0	17.0	0.1	52.0	15.7
Surface Age	4.7	4.0	0.0	12.0	2.6
Mean Monthly Rainfall	108.1	98.0	44.1	247.1	43.1
SNP	3.1	2.9	0.1	6.0	1.6
ESAL/SNP^2	731.8	22.8	2.9	21070.0	3355.6

Note: The ESAL/SNP^2 ratio is used as it is consistent with the current HDM–4 model format

Table 5 shows the value ranges of the variables for the state highway network. For example, the structural number (SNP) was distributed, more or less, equally as shown in Figure 4 – although, there was an 'excessive' number of sections with a SNP ranging between 2.5 and 3.0. Figure 4 also depicts an equivalent Normal

Distribution for comparison. For LTPP studies, evenly distributed samples are more desirable, thus preventing any bias towards the mean value.

The results in Table also show that the ESAL/SNP^2 ratio has an extremely wide range, thus indicating over- and under-designed pavements are being monitored.

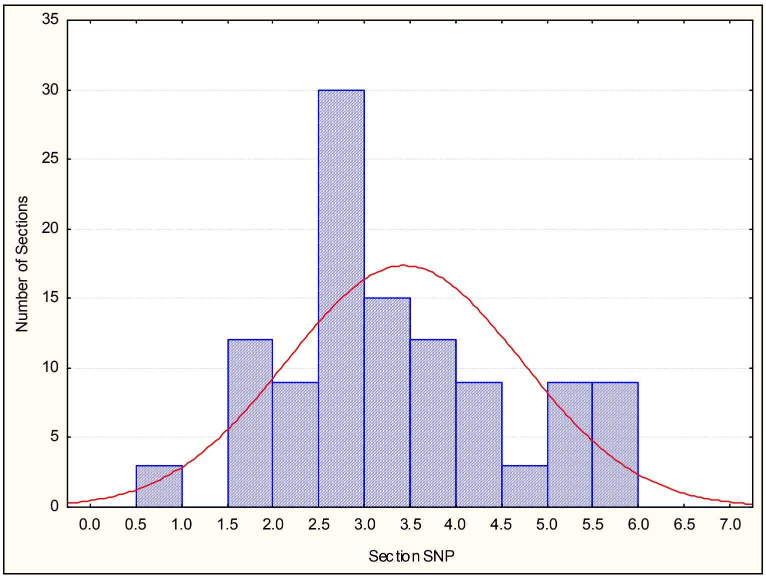

Figure 4: Structural Number Distribution of State Highway LTPP Sections

Testing the Relevance of Data Accuracy Requirements

Visual Surveys

Figure 5 depicts the distribution of crack percentages on all the state highway LTPP sections. It was observed that the number of cracked observations decreased between the survey rounds due to maintenance/resurfacing being undertaken on some sections – N has decreased from 57 to 44. For those sections where maintenance did not occur, the progression or growth of the total crack percentage was evident – the mean has increased from 2.3 to 3.6 and the standard deviation from 2.5 to 4.5. Observations were based on the visual distress data, correlated with the maintenance history on the LTPP sections and appeared to be of sufficient detail and accuracy.

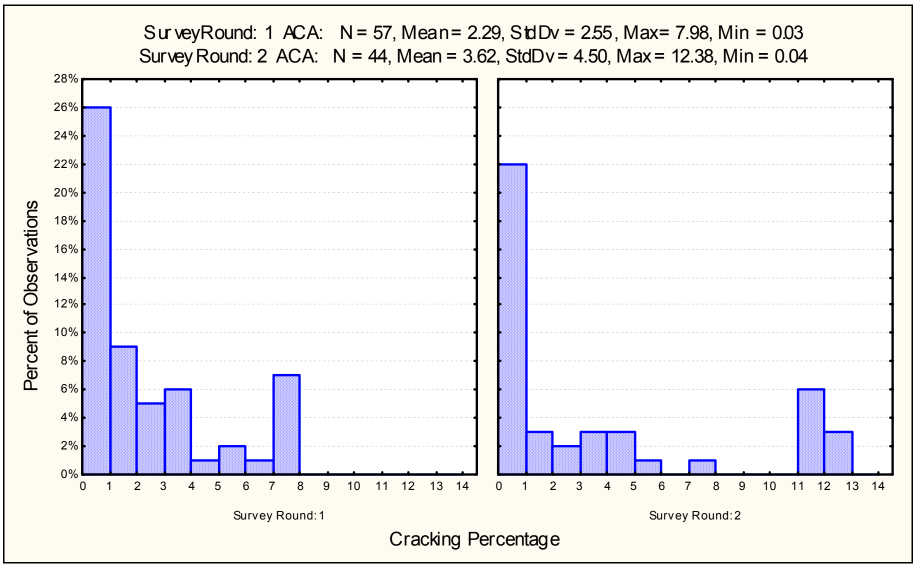

Figure 5: Comparing Crack Status between Survey Rounds (Transit, 2003)

Rutting Measurements

Figure 6 illustrates the change of rut values between the two survey rounds. The negative changes shown in rut depth suggests either maintenance has been undertaken or there were measurement errors. The data was interrogated and it was established that the measurements were undertaken within the specified tolerances.

Over 70% of the incremental change was less than 0.6 mm compared to the assumed 0.5mm per year - therefore it can be accepted that the rut measurement accuracy specified was appropriate.

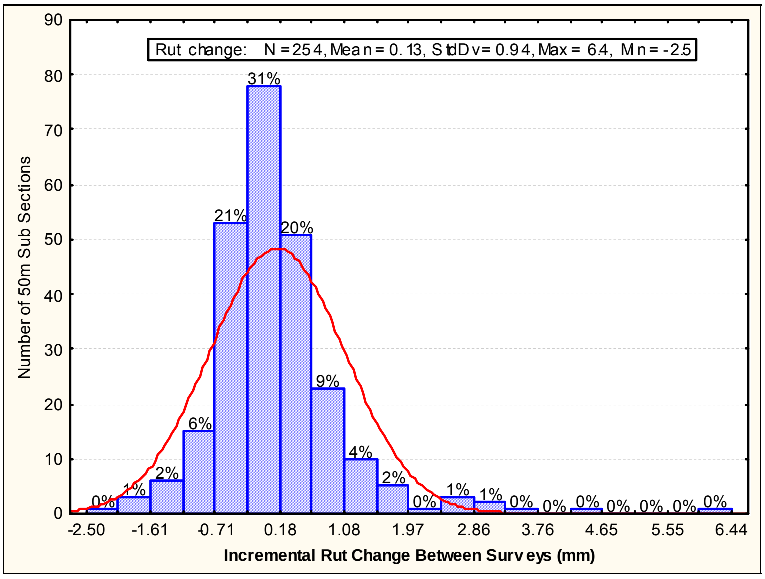

Figure 6: Comparing Incremental Rut Change between Survey Rounds (Transit, 2003)

Figure 6 shows the equivalent Normal Distribution of the rut change results. Except for the sections recording 'extreme' rut changes over 2mm, the distribution shows Normality. These 'extreme' rut change sections required rehabilitation.

Roughness

Figure 7 illustrates the incremental change in roughness between the survey rounds. As indicated before some sections have been maintained and there has been an improvement in roughness between the survey rounds.

More than 70% of the 100m observations had a roughness change less than ± 0.2 IRI between the survey rounds. This relatively small change in roughness thus warranting the accuracy at which the LTPP sections are surveyed.

The average roughness change is slightly negative, implicating an average roughness reduction/improvement. This could be due to either a small bias in the measurements or due to other external factors. Barraclough, et. al. (2003) has indicated that the roughness measurements are sensitive to macro texture – for example some roughness reduction is due to chip embedment. However, this has not been confirmed for the LTPP data.

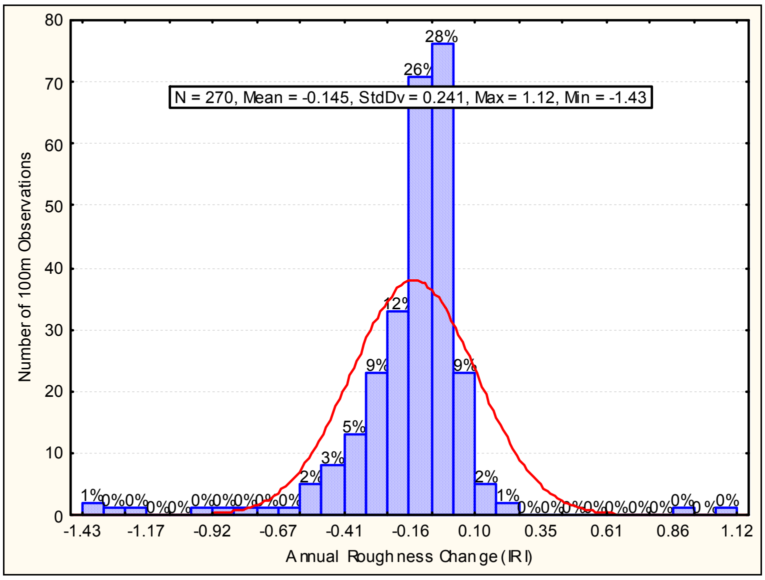

Figure 7: Comparing Roughness Change between Survey Rounds (Transit, 2003)

Henning, et. al. (2004) also compared these manual measurements with HSD. The conclusion was that the HSD accuracy was reduced due to, amongst other factors, location referencing of the measurements. Variances of these HSD measurements were outside the expected annual deterioration thus reducing the value of the data for calibration purposes.

SUMMARY AND LESSONS LEARNED

The New Zealand LTPP programme consists of two components. The main objective of both these LTPP studies is to monitor the pavement deterioration to an accuracy which will enable the calibration of the World Bank HDM models and, where required, provide sufficient data for model format adjustment. From these objectives, the study layout and data collection regime were developed having undertaken a literature review of relevant international LTPP studies.

The Transit New Zealand study, which commenced in 2001, is monitoring 63 sections on the national state highway network. For these sections, data from the two survey rounds, 2001/02 and 2002/03 was presented.

The road controlling authority LTPP study has 82 sections, which have been established across 21 road authorities networks including urban and rural sections. The first survey on these sections commenced in November 2003.

A literature review of international studies was undertaken to assist the setting up of the design matrix and methodological approach for the New Zealand LTPP studies. The international studies reviewed were:

- HDM-III studies in Brazil;
- SHRP studies in the USA and Canada;
- ISOHDM calibration studies in countries such as Australia and South Africa.

The literature review of international LTPP was valuable for the development of the scope, goals and objectives during the initial stages of the New Zealand programme. It assisted in highlighting factors to be considered. For example, a decision was required, whether to monitor specially constructed pavements or to limit the study to in-service pavements. It was decided to focus the New Zealand LTPP studies on the pavement performance of typical roads existing on the network, that is to study only in-service pavements taking into account normal maintenance and limited maintenance regimes. Furthermore, the literature review was useful in determining the design matrix, site layout and data collection regime on these pavements. Some of the aspects, which were of particular importance for use on the New Zealand LTPP programme were given in Table 2 earlier.

The design matrix developed for the New Zealand programme included factors such as environment, traffic, pavement type/strength, pavement condition/age, network type and maintenance regime. The New Zealand LTPP programme has highlighted the importance of the following:

- If pavement strength or composition is used as a factor in the design matrix, it must be categorised according to the design loading it is expected to carry. This will ensure that the study includes pavements which are under- or over-designed;
- Regional classification should not only include climate or rainfall factors but should also include the impact of the climate on the regional geological make-up. For this programme, a ratio between the subgrade strength and a moisture indicator was used;
- Considering pavements of different ages, reduces the time required for the study. However, the pavement age should be considered in combination with condition data. For example, pavement showing deterioration at an early age must be included with some equally aged pavements without any distresses visible;
- Normally, not all the data are available during the initial establishment of the sections. The New Zealand LTPP programme adopted an incremental establishment of sites where say 80% of the sections were established prior to the first survey round. Following this, the remaining sections are being established to cover gaps in the design matrix.

In defining the data collection regime, the HDM calibration guidelines (Bennett and Paterson, 2000) and the Gautrans study (Rohde, et. al, 1998) suggested that HSD type data is sufficient only for adjusting the model calibration coefficients. If the model formats need adjustment, then higher precision data would be required. Henning, et. al. (2004) demonstrated that referencing (location of the data measurements) significantly contributes to the variance in the results – this applies particularly to HSD. Hence, the New Zealand LTPP studies recommended manual measurement as it was intended to adjust the model formats if required.

A review of the data collection regime indicates that it is at an appropriate level for the New Zealand studies. For example 70% of the incremental rutting change was less than 0.6 mm compared to the assumed 0.5mm per year. Similarly, more than 70% of the 100m observations had a roughness change of less than 0.2 IRI between the survey rounds.

This study has demonstrated the importance of aligning data collection regimes with the study outcome and with current data collection regimes on a network level.

At this early stage, the LTPP New Zealand programme has been successful in achieving its goals and objectives. On the other hand, a review of the data has highlighted areas where further research is required and proceeding – for example in determining the factors contributing to the annual condition change and actual deterioration compared to recorded characteristics.

The next major stage of the programme is to investigate the model format and determine the calibration requirements.

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