The Development of Pavement Deterioration Models on the State Highway Network of New Zealand

Theunis F.P. Henning

Supervised by

Associate Professor Roger C.M. Dunn

and

Dr. Seósamh Costello

A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Engineering, The University of Auckland, 2008

Abstract

This thesis presents the results of developing road pavement deterioration models for the State Highway network in New Zealand pavement deterioration models are an integral part of pavement management systems, which are used to forecast long-term maintenance needs and funding requirements on a road network.

As part of this research, a Long-term Pavement Performance (LTPP) programme has been established on 63 sections of the State Highways. These sections are representative of typical road sections and climatic conditions on New Zealand roads. Data collection on these sections is undertaken on an annual basis and consists of high accuracy manual measurements. These measurements include road roughness, rutting, visual defect identification and strength testing with a Falling Weight Deflectometer.

Based on the LTPP data, new model formats for New Zealand conditions were developed including a crack initiation model and a three-stage rut progression model. The rut progression model consists of three stages, initial densification, stable rut growth and a probabilistic model to predict accelerated rut progression. The continuous probabilistic model developed predicts the initiation of pavement failure events such as crack initiation and accelerated rutting. It has been found that this model type has a strong agreement with actual pavement behaviour as it recognises a distribution of failure on roads rather than failure occurring at an particular point in time, namely, a year.

The modelling of rut progression in the three stages including, initial densification, stable rut progression and accelerated rutting has resulted in a significant increased understanding of this defect, especially for thin flexible chip seal pavements. It has been established that the in-service performance of these pavements is relatively predictable. However, incorporating both the in-service performance and the failure of pavements into one model was unrealistic. Therefore, by having the different stages of rutting, resulted into a more accurate forecasting of this defect.

Although this research has covered the two priority pavement models including cracking and rutting prediction, it has established the model framework for other pavement models to be developed. As more data become available, further work can be undertaken to

refine the models and to extend the research into the performance of alternative construction materials.

Dedication

For my wife:

"*Dankie Tania, sonder jou was dit nie moontlik nie*."

Proverbs 31:29

Acknowledgements

The following people and organisations are acknowledged for their contributions and input to the research:

PhD Supervisors

Associate Professor Roger C.M. Dunn

Dr. Seósamh Costello

Research Assistance

Derek Roux

Imedla Muljadi

Dr. Tim Watson

PhD Steering Committee

Prof. Bruce Melville

Dr. Christopher Bennett

Chris Parkman

Gordon Hart

Dr William Paterson

Dr. Gustav Rohde

Funding Organisations

New Zealand Transport Agency (formally Transit and Land Transport New Zealand)

Foundations for Research Science and Technology

MWH New Zealand Ltd.

Glossary of Terms

AADT	Annual average daily traffic
Distress modes	The method or process of failure of pavements, e.g. Cracking of the cemented base course normally occurs due to the tensile stresses at the bottom of the layer.
Calibration Coefficients	Constants applied to a numeric equation (model) to adjust the development of the model in order to make provision for external factors such as climatic or environmental conditions
ESA	Equivalent Standard Axles – the number of equivalent 80 kN axles
Falling Weight Deflectometer (FWD)	A stiffness test performed on pavement as an indicator of strength. A standard load is dropped from a standard height and resulting deflection is measured at given offsets.
Flexible Pavements	Pavements constructed with granular or asphalt materials.
HDM-III	World Bank Highway Design and Maintenance Standards Models
HDM-4	World Bank Development and Management Model
High-speed Data (HSD)	Various condition measurement instruments installed on a vehicle (e.g. roughness, rutting, texture and friction). The recorded measurements are automatically stored in electronic format based on a referencing system (e.g. linear or global positioning)
IRI	International Roughness Index (in mm/km)
Load associated cracking	Appears within the wheel tracks and is an indication of the induced traffic loading is starting to cause damage to the pavement
LTPP Sections	Long-Term Pavement Performance monitoring sections – designed to monitor pavement behaviour as a function of (amongst others) traffic, climate and maintenance.
Model	A numeric equation that quantifies the change of an outcome as a function of different input parameters
Pavement Deterioration	The decay of a pavement or surface as a result of traffic or environmental induced failure modes.
Pavement Management Systems (PMS)	A computer integrated system that incorporates network condition data with Long-Term maintenance planning processes. Most modern systems also include some form of pavement prediction capabilities
Repeatability	An indication of a measuring system being able to measure a consistent value when the measurements are repeated in the same location

Reproducibility	An indication that a measurement in one location would be statistically the same as a measurement undertaken in the same location after some time has past and the equipment had re-established in the same location
Sterilised Sites	A site that will received minimum maintenance only to ensure safety

Table of Contents

List of Figures

List of Tables

CHAPTER 1 INTRODUCTION

1.1 The Context of Pavement Deterioration Models

Pavement deterioration models form a small but essential part of the over-all pavement management systems (PMS) and processes. Figure 1.1 illustrates the typical 'building blocks' of a PMS - data management, forecasting capabilities and the decision logic and optimisation.

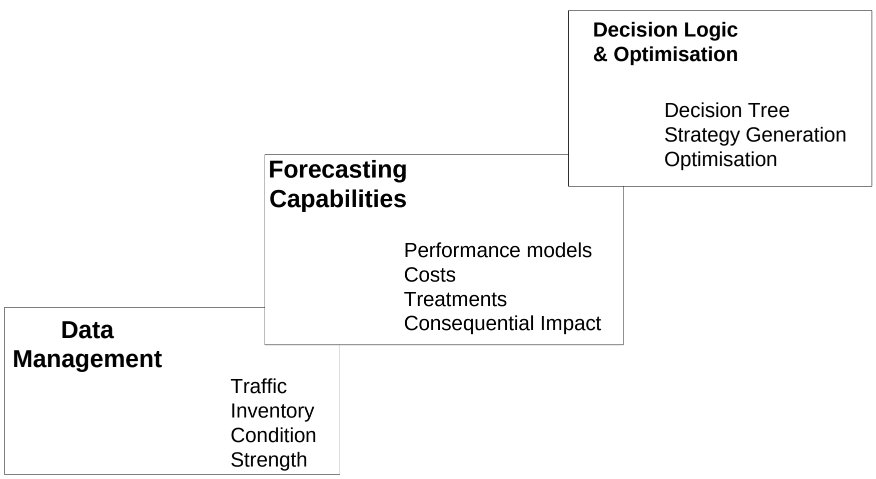


Figure 1.1: 'Building Blocks' of a Pavement Management System

Data management involves all items related to understanding the extent and status of the road network. Typically the following information would be crucial to the asset engineer:

- The full extent of the network including length and total area of all pavements layers and surfaces;
- Quantities of various material types for both the pavements and surfaces;
- The condition and status of each individual section length of the network; and,

- Current and future demand for the network in terms of traffic loading.

Most advanced PMSs are capable of performing to varying degrees, sophisticated **analysis and optimisation routines**. Some apply decision logic on the data to determine when to treat a road length based on a set of intervention criteria. Others will include an optimisation process that determines the best possible maintenance programme for each road, depending on the budget and the life cycle cost aspects of a particular road section.

For any forecasting of future maintenance, funding requirements and future condition, **pavement deterioration models** are essential. These pavement condition models are therefore the 'heartbeat' of any system that is capable of predicting the future network status and budget requirements. In addition, other models such as road user cost and level of service outcomes are dependent of the pavement models for their predictions. As illustrated in Figure 1.1, there are a number of predictions made within the PMS, with the pavement prediction models being only one component of the forecasting process. Yet all other predictions such as cost, treatments and impacts of treatments are reliant on the pavement deterioration models. Therefore, these models are a small yet vital component of modern PMSs.

1.2 The Historical Development of Pavement Modelling in New Zealand

New Zealand embarked on a national PMS during 1998. This system included an off-the-shelf software application, dTIMS, combined with New Zealand maintenance practices. The dTIMS system consists of a sophisticated optimisation routine that includes predictive capabilities to forecast long-term maintenance needs. In order to achieve this, the NZdTIMS system adopted the World Bank HDM-III and, later, the HDM-4 pavement condition deterioration models. From the onset of the NZdTIMS project, the need to calibrate the models to local conditions was realised and this initiated the establishment of the New Zealand Long-Term Pavement Performance (LTPP) programmes. These programmes comprised of two major initiatives:

- Transit New Zealand (Transit) established 63 LTPP sections on the State Highways during 2001; and
- Land Transport New Zealand, in association with 21 local authorities, established 82 sections on typical local authority roads in both urban and rural networks in 2003.

This thesis covers the work by the author related to the establishment of the State Highway LTPP programme and the development of the deterioration models that were based on this data. The research outcomes also led to the author establishing the local authority LTPP programme on the same basis. However, none of the latter data was used in the model development described here.

1.3 Problem Statement

1.3.1 The Need for Calibration of Models in New Zealand

The World Bank's HDM pavement deterioration models were adopted in the NZdTIMS PMS with the realisation that these models needed calibration to local conditions. However, it was also realised that calibration research could only commence once appropriate data was collected and the interaction of the models was clearly understood. With the implementation of the NZdTIMS system, the calibration of the pavement deterioration modelling reached a stage of great urgency. This research had to focus on the areas discussed in subsequent sections.

1.3.2 Establishment of a Monitoring Programme at the Appropriate Level

The establishment of representative calibration sections is imperative for successful calibration. Once the condition measurements are performed on these sections, the data quickly becomes valuable. It could be a significant loss if it is realised later that the data is of little value due to some data not being collected or collected to inappropriate accuracy levels. The unknown questions in this regard were:

- What should be included as part of the design matrix;
- How to conduct the survey on calibration sections;
- How should the sections be laid-out, including other operational issues such as the maintenance allowed on the sections?

Generally even relative high accuracy High-Speed Data (HSD) surveys are not accurate or appropriate for calibration purposes. In most cases, the accuracy of the measurement exceeds the annual change for a condition parameter (e.g. rutting might be measured to an accuracy of 1 mm but the annual change might be only 0.5 mm). Although the HDM models are used internationally, there has been no Level 3 (detailed research) calibration conducted at this accuracy level (Bennett and Paterson, 2000). There is little guidance on effectively collection of data for calibration and validation, and this omission needs to be fully addressed.

Through this research, a full set of specifications were developed for both the establishment of calibration sections and for the data collection accuracy and repeatability. As these specifications developed for the 'Transit LTPP' were proven to be most successful, the same principles were also used for the combined Land Transport New Zealand and Local authority programme.

1.3.3 Undertake the Model Calibration and/or Develop a New Model Framework for New Zealand

It was appreciated that simply adjusting the calibration coefficient may not be sufficient to establish models that reflect reality. Henning and Riley (2000) suggested that if calibration coefficients outside a range of 0.5 to 3 are required, this could indicate that the model is inappropriate for the prevailing conditions of application. For such cases, it will be required to investigate alternative models or a new model form.

Some initial analysis indicated that not all the condition indicators, which drive maintenance planning, are necessarily included in the HDM models. Typical examples of additional models which may be needed include distresses such as shoving and flushing. Therefore, the possible inclusion of additional models should also be investigated.

Lastly, most popularly used pavement deterioration models consist of either empirical or deterministic models that predict the absolute condition of the pavement, at a point in time. There is a need to investigate alternative modelling techniques to better understand the variation of failure or condition. For example, it is well known that a given condition level (normally the average condition) for a road section, it is not always sufficient to understand the pavement status (Kadar et al, 2006). In these cases it is better to also know the distribution of the condition for the section. Likewise, to have an average predicted condition, is not always that effective in understanding the actual predicted condition of the road section. The model development therefore had to consider a method to predict the condition distribution for road sections.

1.4 Objectives of the Research

The objectives of this research were to:

- establish a LTPP monitoring programme in order to produce pavement deterioration data that would be adequate for fundamental pavement model development; and,
- undertake development of a new modelling framework for application on the State Highways. This framework was established on the basis of the two priority pavement models namely crack initiation and rut progression.

In order to achieve the first objective, the applicability of the LTPP data was a function of establishing a sufficient number of monitoring sections across the State Highways in order to ensure a representative sample of the main factors influencing road deterioration. Secondly, the correct data collection regime had to be adopted to ensure the required accuracy and repeatability of the measurements.

It was realised that the NZ pavement types and climatic conditions are significantly different to those upon which the HDM models were originally developed so the calibration issues were untested. Some earlier analyses on the State Highways (Transit, 2000) have suggested that simply calibrating the HDM models may not be able to ensure robust predictions in the PMS system. However, in order to replace the HDM World Bank models with new fundamental model formats and before it would be considered to replace existing models, this research had to:

- undertake robust testing of the existing models; then,
- substantiate significant improvements in forecasting capabilities.

Given the significant work required to achieve this, only two models are covered in this research namely the crack initiation and rut progression models, both deemed as priority models for the maintenance planning on the State Highways.

1.5 Scope and Structure of the Research Report

The structure of this report is shown to Figure 1.2.

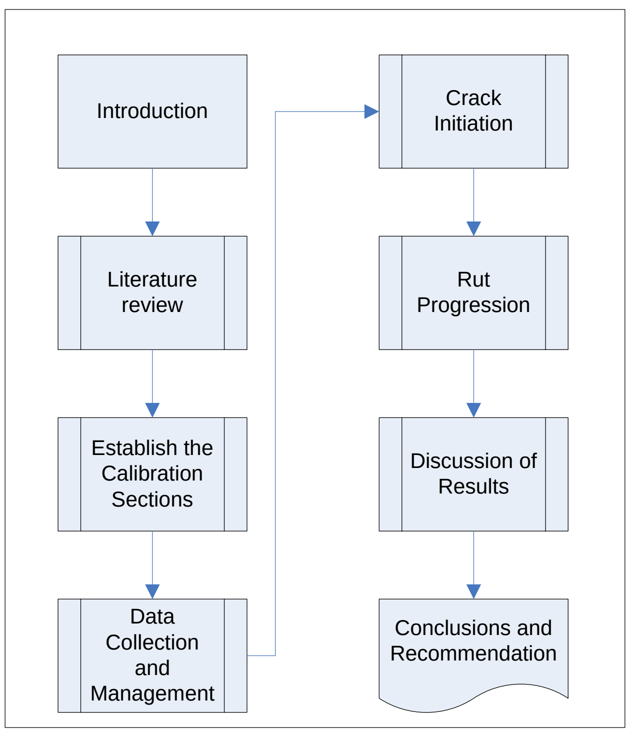


Figure 1.2: Structure of this Research Report

The literature review gives the background to the research in this thesis. Topics included:

- Discussion on previous research undertaken in pavement modelling. This section focuses specifically on the survey techniques and the modelling philosophy; and,
- Background on the philosophy of the HDM-III and HDM-4 models, their shortcomings and the experience in NZ.

The chapter on the literature review is followed by the establishment of the calibration sections and includes:

- Section selection criteria;
- Design of the experiment; and,
- Discussion on the sections used on the State Highway Network.

One of the major outcomes of this thesis is the condition survey of the calibration sections. Since limited information on this aspect was available, this research provides a valuable contribution to the international industry. The philosophy behind the techniques followed is discussed in detail and, where appropriate, more detail is provided on some of the measuring techniques. The data management and analyses are discussed where appropriate as the level of data used differed. Some of the initial analysis was completed based on less accurate data and is only valid for the testing and the adjusting of the calibration coefficients of the models. Later, with the availability of more accurate data, more advanced calibration and model developments is documented.

The anticipated outcome of this research has shifted as a result of some initial analysis outputs. Originally, it was anticipated to review and calibrate most of the models used in NZ. Following some of the initial analysis, it became clear that some more fundamental development work would be required. It was established that a new modelling approach could result in more accurate predictions. Instead of following a deterministic modelling approach, it was recommended to use a continuous probabilistic model that predicts a distribution of failure rather at a point in time. Naturally, the development work included significantly more work than just calibrating existing models. As a result, this research fully investigates two models, namely crack initiation and rut progression.

The significant shift away from the traditional modelling approach warranted each of these condition models to be discussed in detail; they are described in individual chapters. This thesis describes the research which has become the basis of all subsequent model development in New Zealand.

The discussion chapter does a critical review of the major research findings. It deals with questions on the effectiveness of the monitoring programme and the model outcomes and application. The conclusions and recommendations reflect the major outcomes of this research, together with some recommendations on future studies of the same nature, the identification of best practice for modelling and calibration studies, and the scope of future research needs.

CHAPTER 2 LITERATURE REVIEW

2.1 Literature Review - Long-Term Performance Studies

Most countries involved with pavement deterioration modelling have undertaken some form of Long-Term Pavement Performance study (LTPP). This literature review has been carried out with the purpose of better understanding of other methodologies and philosophies and using this knowledge to contribute to this research and the establishment of the NZ LTPP monitoring programme. The major studies reviewed are:

- World Bank
- SHRPP
- LTPP Australia
- Calibration of HDM in South Africa

2.2 World Bank HDM-III LTPP Studies in Kenya and Brazil

2.2.1 Background of the World Bank Studies

The HDM-III studies were conducted in Kenya and Brazil. The major work on pavement and road user costs was conducted in Brazil. The original objective of this research was to (GEIPOT et al, 1981):

'develop methods and models to minimise the cost of transportation on both paved and unpaved low-volume roads in Brazil'

The World Bank model (i.e. HDM-III) comprises of different components as illustrated in Figure 2.1.

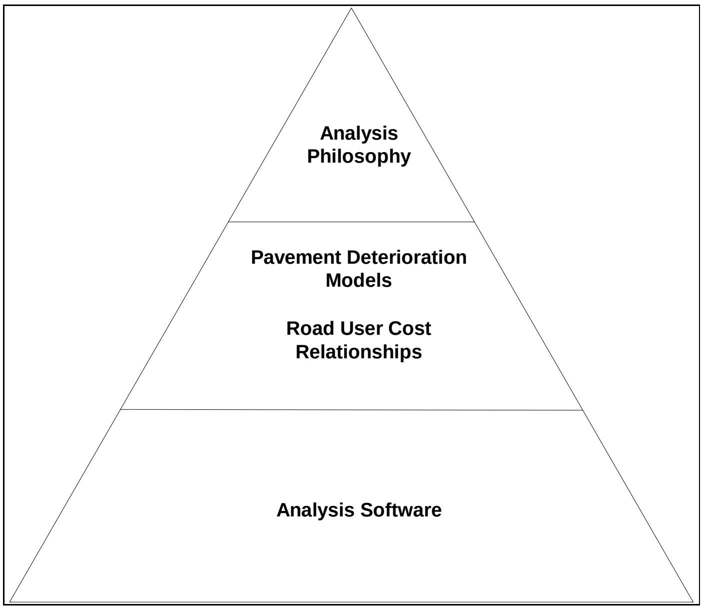


Figure 2.1: Components of HDM-III

Significantly, this research led to much more than just the applications within Brazil. From the HDM studies, software was developed for the application of pavement management on a Network and Project level. Some users have decided to use the HDM analysis philosophy and pavement models, but not the software. In New Zealand, both the analysis philosophy and the pavement deterioration models are used. The software was used before the nation-wide dTIMS implementation.

The Brazil study incorporated research in the following areas:

- Road user costs;
- Vehicle behaviour and performance; and,
- Pavement deterioration and maintenance.

2.2.2 Factorial Design

The technical team for the Brazil study had objectives and limitations which are common to most pavement studies, namely:

- The design matrix was undertaken in a manner to maximise the usefulness of data collected on the sections;

- The scope of the Brazil study had a limited number of sections and measurements and yet the data still had to be sufficient for statistical nalysis
- Once the design matrix was established, not all the factorial cells could be filled. It was, for example, difficult to find sections constructed with crushed rock on lower traffic volume roads as this construction technique is normally only used for higher volume roads

The resulting design matrix is depicted in Figure 2.2 (GEIPOT et al, 1981). Note that the numbers in the cells represent section identification numbers. A total of 65 test sections were used for the experiment that included sealed and unsealed roads.

SURFACING TYPE			ASPHALTIC CONCRETE				DOUBLE SURFACE TREATMENT			
BASE TYPE			GRAVEL		CRUSHED STONE		GRAVEL		CRUSHED STONE	
TRAFFIC (ADT) / VERTICAL GEOMETRY (%) / AGE (YEARS) / STATE REHAB.			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 2.2: Factorial Matrix for Paved Roads in the Brazil Study

Figure 2.2 shows that not all the cells in the design matrix were filled. GEIPOT et al (1981) states that the number of sections used was considered as the absolute minimum. Also, the Brazilian Government specifically built some additional sections for this research expanding the data for the analysis. There is a difference between "in-service" pavements and some sections built under controlled conditions. The latter is aiming at evaluating specific outcomes in the study, while the in-service pavements were used to monitor pavement decay expected from typical pavements in Brazil. The latter approach is common when engineers are experimenting with alternative pavement designs or construction techniques.

The parameters used for the experimental design were only to represent the major factors that influenced the rate of pavement deterioration thus resulting in the simple design matrix. These factors included:

- surface type - asphalt and double chip sealed surfaces
- base type
- traffic volume
- geometry
- rehabilitation history
- maintenance regime

The maintenance regime was classified as 'no maintenance' and 'more than normal maintenance'. In reality, the 'no maintenance' sections received pothole patching where safety was an issue. Routine maintenance allowed crack sealing using slurry seal but no crack sealing was allowed on 'no maintenance' sections in order to monitor the crack progression.

2.2.3 Condition Monitoring

The methodology for the condition measurements used in the Brazil study was state of the art for the 70's. The **roughness** measurements were undertaken with Maysmeter, which can be classified as a response type device. A sensor/transmitter system measured the relative movement between the rear axle and the vehicle body. An electronic distance meter recorded the travel distance and the electronics developed allowed storing roughness values at 80 or 320 m intervals. For the calibration of the Maysmeter a GM surface Dynamics profile was used.

The **rutting** measurements were undertaken using a stationary rut depth gauge as illustrated in Figure 2.3 (GEIPOT et al, 1981). Note, the beam length was 1.2 m where the standard measurement following USA practice, where NZ a 2 m straight edge approach. The rutting was measured at a four to six months time interval with four measurements per sub section recorded.

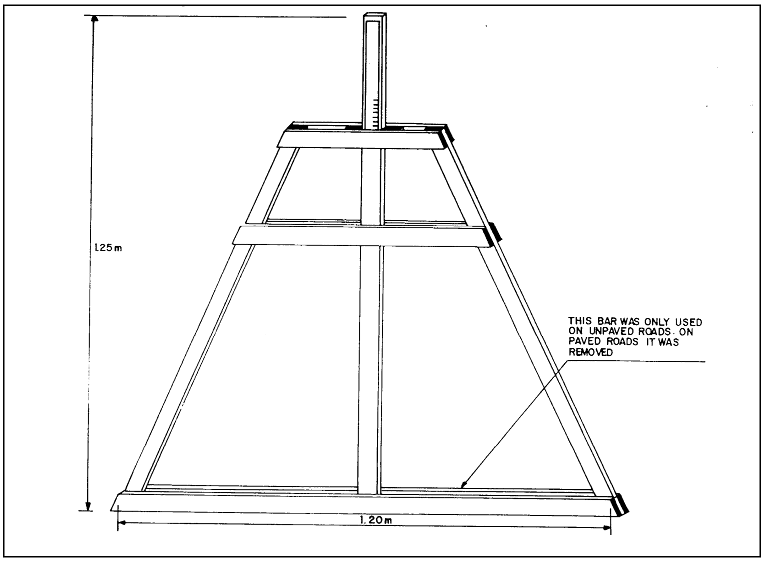


Figure 2.3 Stationary Rut Depth Gauge

The **deflection** was measured by the Benkelman Beam (BB) and Dynaflect system. A high number of deflection tests of 20 readings per lane were performed on each site. All the measurements were taken in the wheel paths. The deflection measurements were used for the strength characterisation of the pavements. The parallel measurements also resulted in the relationships between the BB and the Dynaflect system.

Visual condition surveys recorded the distresses such as cracking and potholes while detailed measurements were taken of the distress sizes as illustrated in Figure 2.4.

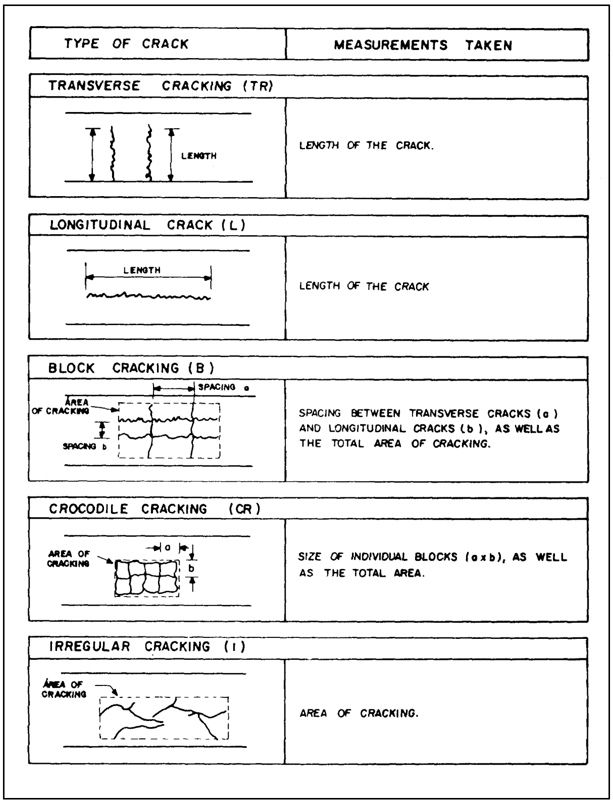


Figure 2.4: Types of Cracks and Measurement Method Used in Brazil

Crack widths were classified in four classes of crack width, namely cracks <1 mm, 1 - 3 mm , > 3mm and cracks wider than 3 mm that may also include secondary failure associated with the cracks.

2.2.4 Traffic Monitoring

Because the Brazil study was focused on the interaction between road condition and vehicle operating cost, the traffic monitoring was undertaken to a very high standard. Traffic volumes were recorded utilising permanent counting stations and vehicle classification was undertaken by means of manually classified counts with durations of 24 hours for 7 days of the week. This data was also supplemented with five-day counts (8 hours of the day) with the vehicle weighing team.

The axle weights were determined using portable scales calibrated against weigh stations operated by the police and some permanent weigh-in-motion stations. A relatively good correlation was established between the portable scales and the weigh in motion stations.

2.3 North Americas – SHRP Study

2.3.1 Scope of the SHRP Study

One of the most comprehensive pavement studies undertaken was the Strategic Highway Research Programme (SHRP) in the USA which, commenced in 1989. The programme mission was to (FHWA, 2000):

'increase pavement life by the investigation of the Long-Term performance of various designs of pavement structures and rehabilitated pavement structures, using different materials and under different loads, environments, subgrades, soils and maintenance practices.'

This 20-year study of in-service pavements across the USA and Canada had six objectives namely:

- Evaluating existing design methods
- Improving design methods and maintenance strategies
- Developing rehabilitation design methods
- Investigating the effect of loading, environment, material properties, construction quality and maintenance levels on the pavement performance
- Determine the effects of specific design features on pavement performance
- Establishing a national Long-Term pavement performance database

The specific objectives and outcomes targeted by the SHRP study are depicted in Figure 2.5 (SHRP-LTPP, 1999a).

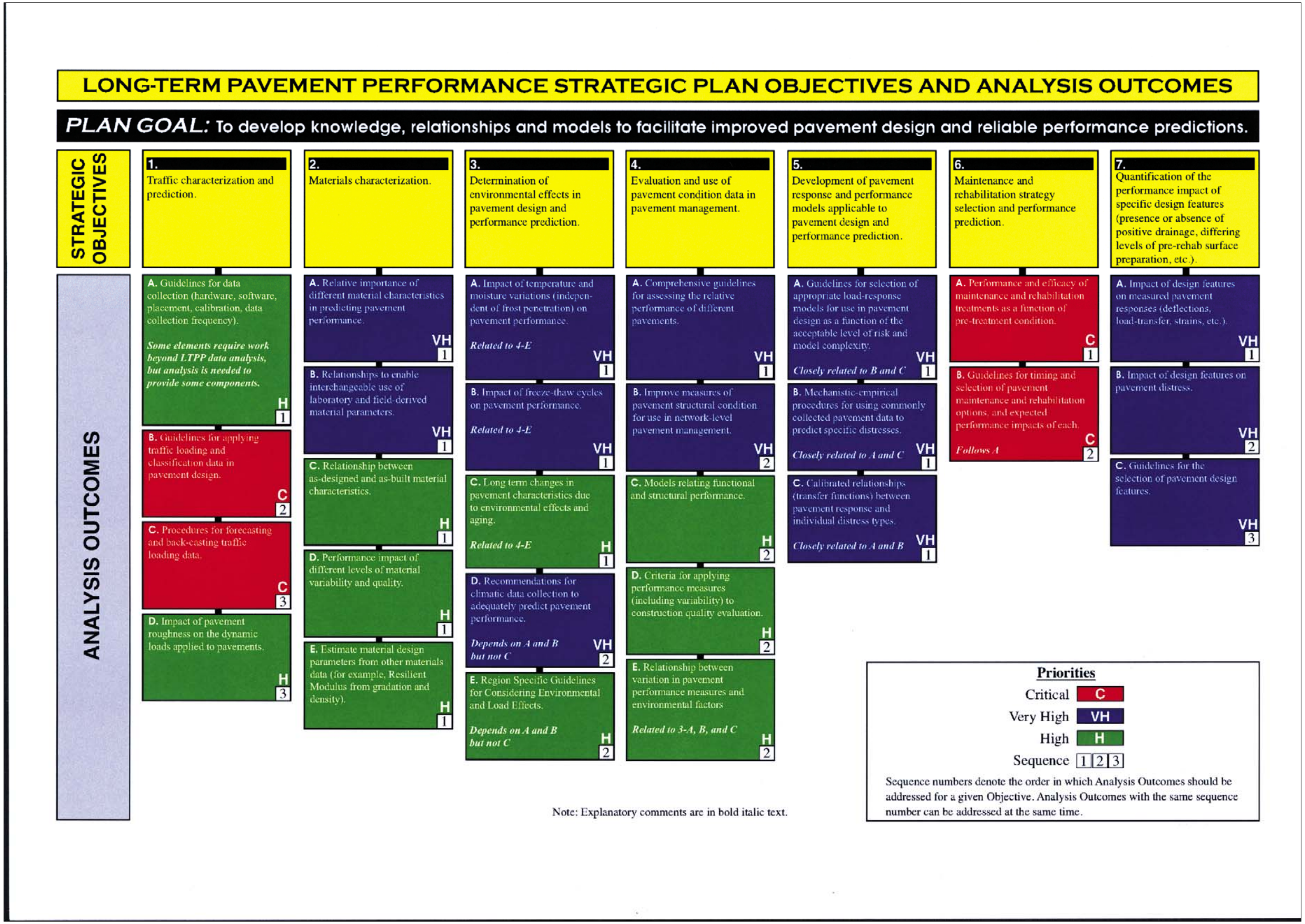


Figure 2.5: LTPP Strategic Plan Objectives and Analysis Outcomes

2.3.2 Experimental Design

The SHRP design matrix was based on work completed by an expert task group. A simple design structure was developed with most factors investigated at two levels (e.g. high and low traffic). It was originally planned that each cell within the design matrix would be populated with two sections of a different age and state. A comment was made that pavement age was treated as a covariant in the design matrix. (SHRP-LTPP, 1999a)

Benson (1991) reviewed the success of the LTPP site establishment relative to the design matrix and the original study objectives. He described the process as follows:

"*The sample matrices formed the basis of an un-weighted, stratified sampling plan that would overcome the predominance of standardised designs in the existing pavement population. Had a completely randomised site selection approach been followed, few unusual pavements designs would have been studied and conclusions would have been confined to performance of standardised pavements. In stretching the limits of the sampling matrices to include unusual combinations of design and environmental factors, it was expected that some cells will remain unfilled. It was hoped that they would occur in a relative balanced pattern throughout the matrix.*"

This approach was also adopted for this research i.e. some sites represent a small portion of pavement/environmental factors occurring on the network. Benson (1991) also discussed an effectiveness ratio that measures the relative balance of the sampling plan. This ratio is further discussed in Chapter 3.

The main conclusions from the sampling review were (Benson, 1991):

- Regional model development must be considered as there were concerns over the inference space in cases where the factorial design was unbalanced;
- Study pavement types could be combined in order to achieve better balances if the suspected failure mechanisms are the same;
- Regional operations have to be reviewed in order to identify potential sources of bias; and,

- The distributions have to be investigated to check for non-normality, bi-modalism and extreme values.

The review (Benson, 1991) further recommended to conduct residual analysis as a function of age, state, region and any factor that could have contributed to bias in the data.

2.3.3 Material Characterisation

Due to the objectives and the nature of the SHRP study, considerable effort went into the characterisation of the pavement materials. Comprehensive guidelines were developed not only for the test methods but also for the sampling of in situ materials (SHRP-LTPP, 1990 and SHRP-LTPP, 1993). Some typical examples of the tests conducted included (SHRP-LTPP, 1999b):

- Laboratory-Measured Modulus of Elasticity
- Asphalt Creep Compliance
- PCC Strength (Portland Cement)
- Unconfined Compression Strength (UCS) for unbound pavements
- Material soil classification

The material tests were also supplemented with field measurements which include:

- Layer thickness
- Layer type – (e.g. generic types such as AC, granular etc.)
- Geometry – i.e. lane and shoulder width. Features such as longitudinal grade etc. are not available
- Drainage including type but not functional performance

The as-built construction detail varies depending on the type of study. For example, comprehensive detail is available for the special pavement studies, whereas the general pavement study only contains non-detailed construction information similar to the information normally kept in management databases.

2.3.4 Pavement Monitoring

Specialised contractors performed the LTPP pavement surveys. By having dedicated contractors, standard equipment and procedures a high degree of uniformity in the measurements was maintained. The measurements performed on the test sections are summarised in Table 2.1 (SHRP-LTPP, 1999b)

Table 2.1: Pavement Monitoring Performed on SHRP-LTTP Sections

Measurement	Detail	Time-Interval[1]
FWD	Standard procedure was used for all measurements. Data supported with test pits.	Varies between bimonthly (on thaw) experiments to 5-year intervals. Before and after measurements are performed for overlays
Longitudinal Profiles (Roughness)	HSD type measurements Some sections included digital incremental profile measurements	Varies between 5 times per year to 1.5 year intervals. All measurements consisted of 5 consecutive runs in a day per section
Distress (e.g. cracking)	Initially 35-mm black-and-white, continuous-strip photographs but later a manual rating was performed	Varies between 3 times per year to 2-year intervals
Load Response	Linear Variable Displacement Transducers (LVDT) strain gauges, and pressure cells were used in some SPS experiments	N/A
Climate	The National Climate Data Centre (NCDC) and Canadian Climate Centre (CCC) were used to create statistics on all sections.	

Note: 1: - Shorter intervals were used for special pavement studies (SPS) sections

2.3.5 Traffic Monitoring

The traffic monitoring was limited only to test lanes. Based on the available data it was assumed that telemetry type information would be available for most sections. However, the quality and availability of the data varied between authorities who participated in the experiment.

It is evident that the traffic monitoring methods were not undertaken consistently across all sections. For example, some sections had some weigh-in-motion data available, others did not.

2.4 Australia – Development of New Pavement Models

2.4.1 The Purpose of the ARRB-LTPP Study

The Australian Road Research Board (ARRB) research in developing the ARRB pavement models commenced in 1990 (Martin, 1994). This research was aimed at developing Long-Term pavement models to establish the life cycle costs of roads (LCC) based on the usage of the roads. One outcome of this was to assist in the calculation of road agency expenditure (road track costs), on a 'pay-as-go basis', relative to the type of traffic (i.e. light vehicles vs. heavy vehicles).

The ARRB pavement model was developed to cater for granular and asphaltic flexible pavements. It was further developed to contain the following features (Martin, 1994):

- Prediction of expected pavement surface condition, based on a varying surface maintenance expenditure
- Prediction of surface condition based on varying structural maintenance expenditure
- Prediction of surface deterioration costs (surface condition consumption), due to maintenance under constrained budget conditions

For the purpose of the ARRB model, surface condition is equivalent to road roughness.

The road categories used for the ARRB study included:

- national highways;
- rural arterials, and;

- urban arterials including freeways.

The ARRB models were developed for strategic analysis only and it was thought that it would be too expensive also to calibrate the HDM models to fulfil the same purpose. More emphasis was therefore placed on HSD type data instead of more detailed surface distress data as required for the HDM III type models.

2.4.2 Experimental Design

The factorial design included traffic range, pavement strength, pavement age, daily temperature and annual rainfall. A total of 105 sections were used across Australia as depicted in Table 2.2 (Martin, 1994). Although the pavement age was used in the design, no consideration was taken of the actual condition (i.e. whether a pavement was in a poor condition when it was still relatively new).

Little information is available regarding the physical characteristic of the sample sections, such as length and number of lanes. The roughness measurements were also not documented, but it is assumed that HSD type data was used. The data collection is therefore not similar to the objectives for Level 3 calibration studies as being used in this research.

Table 2.2: Pavement Samples Used in the ARRB Model Study

Arterial Road Category	State	No. of Samples	Range of AADT	Years of Deterioration (max)	Range of Mean Daily Temp (^{o}C) *	Range of Mean Annual Rainfall (mm) **	Range of SNC Values
National Highways	VIC	11	2900 - 21400	12	12 - 15	450 - 650	2.0 - 4.1
	SA	6	370 - 5100	4	16 - 20	200 - 500	1.8 - 3.6
	NT	6	90 - 450	6	21 - 27	250 - 1150	2.7 - 5.0
	WA	2	340 - 1550	8	18 - 23	230 - 430	2.6 - 5.2
	QLD	2	490 - 2750	9	24 - 25	390 - 3800	2.0 - 3.2
	NSW	4	4900 - 9700	7	11 - 18	640 - 880	2.7 - 3.4
	TOTAL	**31**					
Rural Arterials AADT > 2000	VIC	10	2000 - 30700	12	12 - 15	450 - 1050	2.0 - 4.7
	SA	5	2100 - 4200	4	13 - 18	450 - 700	1.5 - 2.5
	TAS	1	3000 - 3850	5	12	600	2.2
	WA	2	3850 - 4100	8	16 - 17	870 - 1220	2.5
	NSW	4	3650 - 15300	7	12 - 19	660 - 1710	2.4 - 3.8
	TOTAL	**22**					
Rural Arterials AADT < 2000	VIC	7	500 - 1600	12	13 - 17	290 - 530	2.3 - 3.4
	SA	9	650 - 1600	4	16 - 17	250 - 500	0.6 - 2.3
	TAS	6	600 - 1400	5	11 - 13	190 - 2400	0.5 - 3.1
	WA	2	1500 - 4100	8	15 - 17	800 - 1220	2.7 - 3.8
	TOTAL	**24**					
Urban Arterials (incl. Freeways) AADT > 20000	WA	8	20300 - 98600	8	19	870	3.7 - 4.7
	VIC	8	21400 - 54000	4	14 - 15	560 - 900	2.7 - 4.2
	TOTAL	**16**					
Urban Arterials (incl. Freeways) AADT < 20000	VIC	7	4000 - 12600	4	13 - 15	560 - 1500	1.1 - 4.2
	NSW	5	8800	4	17	800	1.8 - 4.6
	TOTAL	**12**					

* Based on the simple average of the mean daily maximum and minimum temperatures over the years of record. Source: Bureau of Meteorology 1988.
** Based on the mean annual rainfall over the years of record. Source: Bureau of Meteorology 1988.

2.4.3 Study Outcome

The ARRB model development study outcome was compared with HDM III models as illustrated in Figure 2.6 (Martin, 1994)

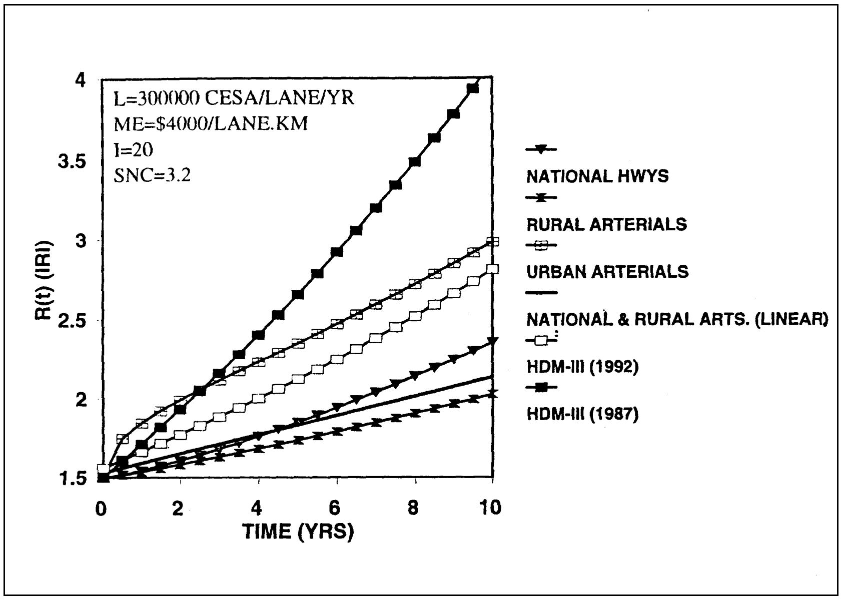


Figure 2.6: Comparison of Roughness Predictions for Various Models

Both versions of the HDM III roughness models (1987and 1992) shown in Figure 2.6 resulted in higher roughness predictions over the life cycle of the roads compared with the ARRB models.

2.5 Australia – Calibration of HDM-4 Pavement Models

2.5.1 Australia's Involvement in HDM-4 Development

Australia was one of the main contributors towards the HDM-4 study. In terms of calibration of the HDM-4 models, a study was still in progress during this research that includes eight LTPP-Maintenance (LTPPM) sections; four in Victoria, two in Queensland, one in NSW and one in Tasmania (Tepper and Martin, 1999). These sections were specifically established to calibrate the HDM-4 Works Effects models

(WE). There are also another five LTPP sections under varying traffic conditions and environmental conditions. The objective of studying these sections is to calibrate pavement models according to (Martin, 2003):

- Relative effects of maintenance on performance of pavements subject to accelerated traffic loading tests
- Estimating the actual rates of decay of pavements subject to varying conditions of maintenance, environment and loading (8 LTPPM sections)
- Estimating the actual rates of decay of pavements subject to varying conditions of environment and loading (5 LTPP sections)

2.5.2 Site Criteria and Details

The factorial design for these sections included factors such as (Tepper and Martin, 1999):

- Traffic levels
- Pavement type
- Surface condition
- Maintenance history
- Pavement strength

The site selection was undertaken based on the following factors:

i. Homogeneous in terms of:
    - Pavement age
    - Surface condition
    - Maintenance history
    - Pavement strength

ii. The sealed section should have a high reseal demand e.g. a surface which is cracked

iii. Should be located in close proximity to a permanent traffic measurement facility

The details of the sections are depicted in Table 2.3 (Tepper and Martin, 1999).

Table 2.3: ARRB LTPPM Site Details

Road	Location	Lane/ Direction	Chainage	Location Type	Thornthwaite Index, I[1]	Estimated Traffic Loading # (ESA/ lane/yr)
Stud Rd	Dandenong North (Vic)	Outer northbound lane	1km from Brady Rd towards Police Rd	Urban	40	3.0×10^5
Western Hwy	Gerang Gerung (Vic)	Westbound slow lane	346km to 347km	Rural	-22.5	4.2×10^5
Princes Hwy West	Heywood (Vic)	Southbound carriageway	364.5km to 365.5km	Rural	49	5.8×10^4
South Gippsland Hwy	Woodside (Vic)	Eastbound carriageway	233.25km to 234.25km	Rural	10	8.1×10^4
Bruce Highway	North of Ingham (Qld)	Outbound North Lane	73.0 km to 74.0 km	Rural	+100	8.5×10^4
Flinders Highway	West of Townsville (Qld)	Outbound West Lane	41.42 km to 42.42 km	Rural	+27	1.7×10^5
Great Western Highway	Blacktown (NSW)	East Bound Slow Lane	Doonside Road to Walters Road	Urban	+82	5.6×10^5
Esk Main Road	Fingal (Tas)	Eastbound Lane	2.612 km to 3.612 km	Rural	+5	7.4×10^4

 # 1998 figures, except 1997 data for Western Hwy and 1996 data used for Great Western Hwy.

The experimental layout of a typical section is illustrated in Figure 2.7 (Tepper and Martin, 1999). Note that different maintenance treatments are tested on the various sections. Also note that with five sections (each 200 m long) per site a total of 40 sections are being monitored.

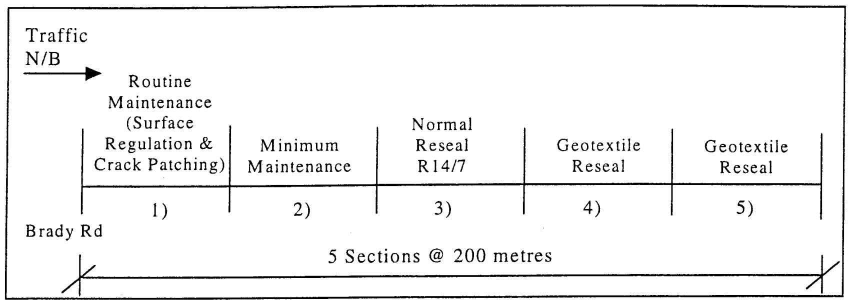


Figure 2.7: Example of Maintenance Treatments Applied on ARRB's LTPPM Study Sections

2.5.3 Data Collection

Only two instruments were used for the data collection on LTPPM sections: the FWD for strength calculation and the ARRB Multi-Laser Profiler (MLP) for the longitudinal and transversal profiles. The MLP used 13 lasers, two accelerometers and high accuracy distance transducers to collect pavement condition data at highway speeds.

The longitudinal profile was analysed to yield the roughness (in IRI) using the 'quarter car model' (Sayers et al, 1986). The MLP also measured the transverse profile, using a 3m rut bar equipped with lasers spaced at 50 mm intervals. The profile was analysed using a wire simulation in order to establish the rut depth. No details were provided regarding any repeated measurements performed on the sections.

All the sections were tested prior to the treatments and to date two post-treated measurements were taken since the sections were treated. The sections characteristics are summarised in Table 2.4.

Table 2.4: Summary of ARRB-LTPP Section Characteristics (Martin, 2003)

Exp. No.	Description	Nominal SNP	SNP Std. Dev.[3]	RDS/Rut[4]		
				Range[5]	Initial[6]	Average
#1	Single seal cracked (wet)[1]	4.9	0.45	0.4 – 1	0.4	0.7
#2	Single seal uncracked (wet)[1]	6.7	0.6	0.2 – 1.3	0.4	0.5
#3	Geotextile seal uncracked (wet)[1]	5.0	0.4	0.2 – 0.6	0.5	0.4
1.	Single seal cracked (wet)[2]	4.2	0.1	0.1 – 0.3	0.3	0.2
2.	Single seal uncracked (wet)[2]	4.1	0.2	0.3	0.3	0.3
3.	Single seal uncracked (dry)[2]	4.6	0.1	0.2	0.2	0.2
4.	Double seal uncracked (wet)[2]	3.8	0.2	0.4 – 0.7	0.4	0.5
4A.	Double seal uncracked (wet)[2]	3.7	0.1	0.3 – 0.5	0.3	0.4
5.	Geotextile seal uncracked (wet)[2]	4.5	0.2	0.2 – 0.3	0.2	0.3
5A	Geotextile seal uncracked (wet)[2]	4.0	0.2	0.2 – 0.4	0.2	0.3
6.	Single seal uncracked (dry)[2]	4.6	0.2	0.3 – 0.4	0.4	0.3
7.	Single seal uncracked (wet)[2]	4.2	0.2	0.1 – 0.4	0.2	0.2
8.	Single seal cracked (wet)[2]	4.7	0.1	0.1 – 0.3	0.2	0.2

Note:				
	1.	Experiments at Old Longwood on natural subgrade.	4.	Standard Deviation of Rut depth/Mean Rut depth.
	2.	Experiments at Dandenong under controlled environment.	5.	RDS/Rut estimated during the course of the experiment.
	3.	Standard Deviation of SNP along each test section.	6.	Initial RDS/Rut after 'bedding in'.

It is observed that although Table 2.3 shows varying traffic loadings, the pavement strengths given in Table 2.4 do not vary significantly between the sites.

2.6 South African (Gautrans) HDM-III and HDM-4 Calibration Studies

2.6.1 Scope of the SA Calibration Study

After the 1994 general elections, the provincial arrangements in South Africa changed. The Gauteng Provincial Government became one of the smaller provinces, which was part of the former Transvaal Province. Just before these changes, the Transvaal Provincial Administration (TPA) embarked on a LTPP study to calibrate the HDM-III and HDM-4 pavement models. The first survey of this research started in 1993. In an effort to continuo with the study, permission was grated from the neighbouring provinces for the Gauteng Provincial Government Roading Department (Gautrans) to continue with the data collection. As a result, all of the original 36 sections have yielded 15 years of LTPP data.

Due to the social needs within two of the four provinces, a lack of road maintenance funds resulted in some of the LTPP sections not being maintained for extended periods. Hence, this research has the benefit of effective "sterilised" sections, which provided pavement deterioration data for roads without any maintenance.

In an assessment of the calibration results, Rohde et. al. (2002) compared the results from the calibration study with actual PMS trends of the past nine years. The main conclusions from this report were:

- Calibration data showed a good correlation with network data where maintenance was accurately recorded;
- In particular, a good correlation was found with the cracking models, while better correlation with rutting was expected;
- Higher variability between calibration data and network data existed for sections where no-maintenance was recorded at network level; and,
- Overall, predicted conditions were mostly conservative in comparison with actual network conditions (i.e. the predicted condition was usually worse than the actual values).

Recommendations from the report (Rohde, et. al, 2002) included:

- The cracking model can be adopted in its current form;
- The rutting model form requires improvement;
- The roughness model performs satisfactorily
- Overall, the predicted condition distribution correlated well with the network trends and is therefore satisfactory for the PMS application

2.6.2 Experimental Design

The Gautrans study experimental design resulted in a total of 36 sealed road sections presenting the network that consisted of the old Transvaal Province (Rohde, et. al, 1998). The experimental design included:

3	Traffic Levels
2	Base Types (Un-bound and bound)
2	Environments (Dry and Moderate)

3 Conditions (Poor, Moderate and Good)

36 Resulting number of sections

The condition in the design matrix was calculated based on a composite pavement condition index (PCI) used in the Gautrans PMS (Henning, et.al, 1998).

The design matrix used was therefore relatively simple and resulted in the establishment of sections with a range of characteristics as depicted in Table 2.5.

Table 2.5: Range of the LTPP Section Characteristics in the Gautrans Experiment (Rohde, et. al, 1998)

Parameter	Range
Number of sections	36
Number of sub-sections	720
Length of sections	10 x 50m x 2 directions
Pavement Age (years)	6 – 40
Surface Age (years)	2 – 19
Annual Rainfall (mm/year)	480 – 810
Cumulative Traffic Load (10^6 ESA)	0.05 – 3.43
Traffic Loading Rate (10^6 ESA/lane/year)	0.003 – 0.22
Traffic volume (veh/day)	772 – 9894
Total Surface Thickness (mm)	10 – 100
Total Pavement Thickness (mm)	280 – 850
Maximum FWD Deflection (mm)	0.15 – 0.8
Modified Structural Number (SNP)	2.3 – 6.5
In situ Subgrade CBR (%)	3 – 80

2.6.3 Site Layout

As indicated in Table 2.5 each site has been divided into 50 m subsections. This sectioning method was adopted to achieve a higher accuracy with the visual condition rating.

Each section was identified with marker plates on the road reserve fence and each section layout was painted on the road with spray paint. Later it became apparent that it was difficult to re-establish sections where resurfacing had occurred and/or the marker plates were removed/lost.

2.6.4 Data Collection

The measurements on the sections included:

- Visual rating using a rating form developed by Van Zyl (1994)
- Pavement Strength using the FWD at 50m intervals
- Test pit on each site to determine layer thickness and in situ CBR
- Roughness using HSD type measurements (for the first two years the Merlin wheel was also used)
- Rutting measurements undertaken using a manual rut depth measurement device
- Traffic volumes and loading were determined based on Weigh-in-motion measurements

2.7 Guidance for the Research

Four major calibration/LTPP studies during the past two decades have been investigated and the main points relevant to NZ are summarised in Table 2.6

Table 2.6: Relative 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 to be adopted in New Zealand
	Specific pavements were built to monitor certain material types.	Aim is more towards modelling in-service pavements.
	Allowed limited maintenance on sterilised sections.	New Zealand also has to comply with safety requirements.
	A well defined measure of visual crack detection was used	A similar approach to be adopted in New Zealand.
	Traffic monitoring was undertaken at a high level	Similar approach to be adopted in New Zealand
SHRP-USA	The scale of the study was astronomical.	New Zealand is not in a position to conduct a study to this scale.
	Multiple objectives were addressed with the study.	Main concern is to investigate pavement performance.
	Experimental design only considers major pavement deterioration factors.	A similar approach to be adopted in 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 to be adopted in New Zealand.
	An effectiveness ratio was developed to test the applicability of chosen sites, relative to the original design matrix.	A similar approach to be adopted in New Zealand.

HDM-4 Australia	Different maintenance treatments were tested within one section.	Aim of the New Zealand Study was more aimed at monitoring in-service pavements
	Experimental layout effectively isolates certain factors on the design matrix e.g. everything else is the same while maintenance treatment is varied.	Best option for monitoring maintenance effects.
	Mix of different LTPP sites types were used –e.g. specially built and in-service pavements.	Good benchmarking for in-service and specific type pavements.
HDM-III and 4 Gautrans SA	Experimental design only considers major pavement deterioration factors.	A similar approach to be adopted in 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 to be adopted in New Zealand.
	Data collection precision was sufficient for Level II calibration.	A higher precision level would be required to allow for model form adjustment.
	Sterilised sites did not get any maintenance.	New Zealand has to comply with safety requirements.
	In-service pavement were monitored over an extended period	A similar approach is recommended for New Zealand.

CHAPTER 3 EXPERIMENTAL DESIGN - ESTABLISHMENT OF THE STATE HIGHWAY CALIBRATION SECTIONS

3.1 Introduction

Effective experimental design is imperative for a successful calibration study. It has to be ensured that all the data required is collected, that the sample of calibration sections are representative of the total network and that the calibration sections include most of the expected independent variables that influence the rate of pavement deterioration. This chapter discusses the experimental layout that was used to determine the:

- number of sections required for the study;
- distribution of these sections across the country;
- location of the sections in order to incorporate the factors from the design matrix of the study;
- methodology and criteria for section selection; and,
- layout of the sections in terms of the length and site layout.

This chapter documents the process of selecting and establishing, on site, the calibration sections.

3.2 Climatic Stratification

3.2.1 Background

All the pavement performance studies discussed in Chapter 2 have used some form of regionalisation method in the experimental design. Pavements will behave differently in different parts of a country like New Zealand, due to climatic and geological factors. As an example, it is known that pavements in Northland will behave differently to pavements in the Canterbury area. The regionalisation factors considered for the LTPP study in New Zealand are listed and discussed below.

The **geological** history affects the make-up of the pavements in both the in situ layers as well as the constructed pavement layers. As an example, granite material found in the South Island is a relatively stable material compared to sensitive rhyolitic silts which are found in the North Island (Prebble, 1998). The characteristic of these material types varies so significantly that while granite could be considered for a pavement layer material, rhyolitic silts would be less desirable material to have as an in situ material.

The influence of **climate** on the geology is significant enough that engineering geologists have established relationships between climatic conditions and the behaviour of soils. Most common examples of climatic indices include those developed by Thornthwaite (1948) and Weinert (1980). Both these indices express the climate as a function of the rainfall, evaporation and the temperature. For pavement, engineering it is a popular method to classify geological regions in terms of climatic conditions rather than pure geology formation classifications. The climate is therefore, a good moderator for the behaviour of material.

3.2.2 International Practice

During the original HDM-III study in Brazil, annual rainfall was used to map the varying climatic areas of the country. It is noted from GEIPOT (1981) that these rainfall maps were considered during the section establishment. The section locations also suggest that a good representation of sections occurred within each climatic area. However, no evidence was found that these climatic regions were considered in the experimental design since they do not feature in the factorial matrix for the experiment.

Rohde et al. (1998) used the Weinert N value as the regionalisation factor in the Gautrans LTPP study. The Weinert N value was developed in South Africa after it had been observed that the same type of material performed differently in certain parts of the country with different climates (Weinert, 1980). This regionalisation value differs from the other indices considered (e.g. the Lang rain factor and Thornthwaite's moisture index) as it not only considered the total rainfall, but it also took into account the seasonal variation of rainfall in combination with potential evaporation. The latter factor had a significant influence on the availability of water as an evaporation agent. The N-value is defined as (Weinert, 1974):

$$N = \frac{12E_J}{P_a}$$ **Equation 3.1**

where N is the Weinert N Value

E_j the computed evaporation in January

P_a the total annual precipitation

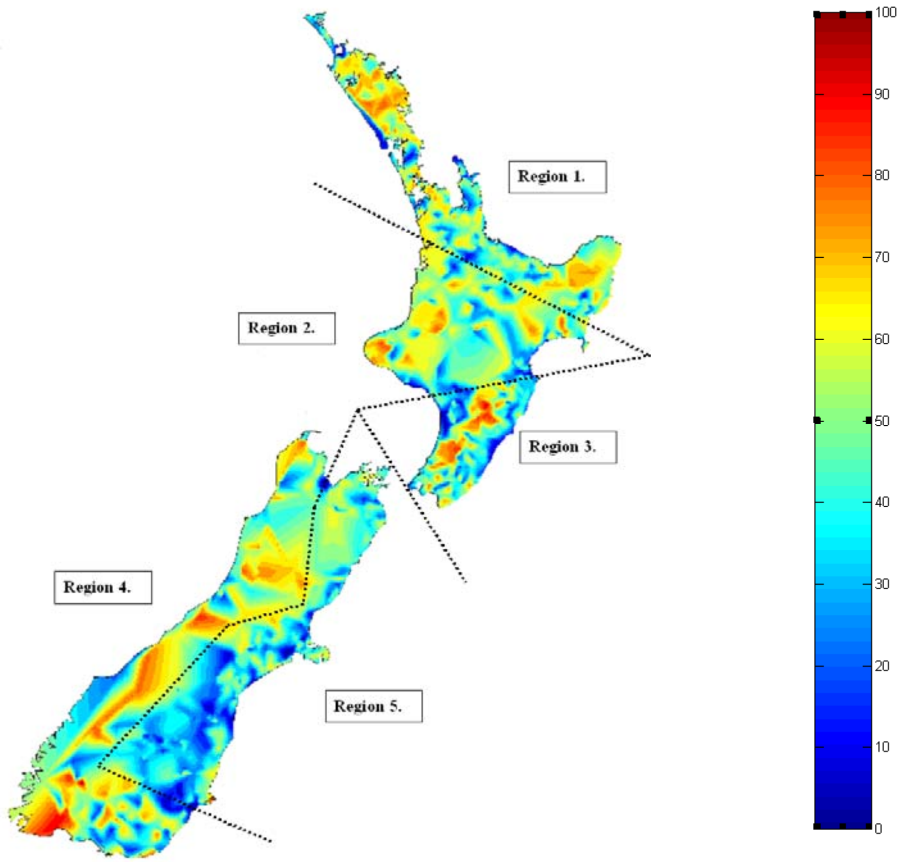
3.2.3 Developments in New Zealand

One of the limitations of the indices described in the previous section is that they only considered rainfall and evaporation factors. The ideal would be to have a combined index, which considers both geology and climate factors. By considering both these factors, a better estimate of material behaviour in pavements could be achieved. For example, a 'clayey' material would be stable under constant moisture conditions but with varying moisture, heaving and cracking of the clay would often have adverse effects on pavement structures founded on the material.

Cenek (2001) divided NZ into climatic regions for the purpose of model calibration across local authorities. He used a combination of geological and climatic factors to derive a ratio between the subgrade and moisture as seen in Equation 3.2.

$$\frac{\text{Subgrade Strength}}{\text{Moisture Indicator}} = \left[Log_{10}\left(\frac{\text{Wet Strength}^{3}}{\text{Moisture}^{0.5}} \right) \right]^{1.5} \qquad \textbf{Equation 3.2}$$

The ‘wet strength’ is a ranking from 1 to 100 of the soil wet strength in the NZ Soil Classification by Hewitt (1998). 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 the Thornthwaite Index. Wet strength is a good moderator of susceptibility of subgrade strength to moisture, since this reflects the worse case for pavement failure. Based on this ratio, New Zealand was divided into climatic regions as depicted in Figure 3.1.



NOTE: Blue shading represents high wet subgrade strength/low winter moisture whereas red shading represents low wet subgrade strength/high winter moisture. Green/yellow shading represents areas where compensatory factors are in play i.e. low wet subgrade strength/low winter moisture or high wet subgrade strength/high winter moisture.

Figure 3.1: Climatic Regions of NZ According to Subgrade Strength/ Moisture Ratio (Cenek, 2001)

3.2.4 Climatic Stratification used for the State Highway Network

This research has followed the general approach used by Cenek (2001). However, instead of using the regional distribution depicted in Figure 3.1, the outputs were more closely examined to establish regions with similar climatic and soil conditions, which were not necessarily located within the same geographical area.

The State Highway network was divided into sensitivity areas that reflected the combined effect of subgrade moisture and susceptibility of the subgrade to moisture. In theory, a high sensitivity area consists of sensitive soils within a wet climate. On the other end of the scale, low sensitivity areas are areas that are dryer and consist of more stable materials. A preliminary classification of the State Highway regions according to this scheme is presented in Table 3.1.

Table 3.1: Preliminary Regional Distribution of State Highway Calibration Sections

Sensitivity Area	Calibration Sections Within State Highway Regions
High	Northland, West Waikato, Gisborne, West Coast
Moderate	Coastal Otago, Auckland, Wanganui, Taranaki, Wellington
Low	Nelson, Marlborough, Napier, East Waikato
Limited	Canterbury

The preliminary regionalisation was used during the initial establishment of the sections and was confirmed later during the study when sufficient performance data became available.

3.3 Traffic/Loading

Traffic volumes on the State Highway network are not equally distributed across the country. Around the main centres such as Auckland and Wellington very high traffic volumes are recorded, where as most of the rest of the State Highways throughout New Zealand have medium to low traffic volumes.

In order to establish calibration sections over the full range of expected traffic volumes, it was necessary to use a traffic categorisation for the different regions as indicated in Table 3.2

Table 3.2 Traffic Classification System Used for Experimental Design

Traffic Volume Classification	Traffic Volume Range (ESA[1]) for Transit Regions with High Traffic Volumes (e.g. Auckland/Wellington)	Traffic Volume Range (ESA) for Transit Regions with Low Traffic Volumes (E.g. Canterbury)
Low	< 400	< 100
Moderate	400 – 1000	100 – 400
High	>1000	> 400

Notes: 1 ESA – Equivalent Standard Axle

3.4 Pavement Strength/Pavement Types

For most LTPP studies, it has been noted that it can be challenging to establish calibration sections that cover all the cells within a design matrix. For example, for the higher volume roads, stronger pavements can easily be located. However, very weak pavements are not found on high volume roads. The same concept applies to pavement/surface types. For example, granular pavements with chip seal surfacing will seldom if ever be used on traffic volumes above an AADT more that 10,000 per day. For these reasons, it was accepted that the State Highway LTPP sections would be limited by policy and current design principles and standards. A greater focus was placed on achieving a range of sections which represented both over and under designed pavements for a given traffic loading.

A further challenge faced when populating the State Highway LTPP design matrix with calibration sections was to estimate pavement strength without having actual strength measurements available during site establishment. In order to overcome this, a decision was made to estimate the strength classification based on the broad characteristics of the pavement.

The resulting pavement classification used is depicted in Table 3.3.

Table 3.3: Strength Classification Used for the Transit LTPP Study (Henning et al, 2004)

Weak Pavements	Strong Pavements
Unbound pavements with chip seals - total pavement shallower than 300mm or, Estimated SNP < 3	Unbound pavements with chip seals – total pavement deeper than 300mm or, (Asphaltic surfaced pavements) or, Estimated SNP ≥ 3

Note: SNP is the structural number as derived from Falling Weight Deflectometer measurements

Candidate LTPP sections were established using the criteria defined and subsequent to the final section selection and establishment, the measured/tested SNP values were used to confirm actual characteristics of the sections. The resulting statistical distribution of the final LTPP sections is presented in Section 3.8.

3.5 Condition / Age

In order to obtain sufficient data points for the calibration analyses, pavement age/deterioration was selected as an independent variable for the design matrix. Figure 3.2 illustrates three generally accepted phases of pavement deterioration that is the ‘new’, ‘aged’ and ‘end of life stages’. It would be ideal to have a representative sample of LTPP sites within each of the age distribution classes.

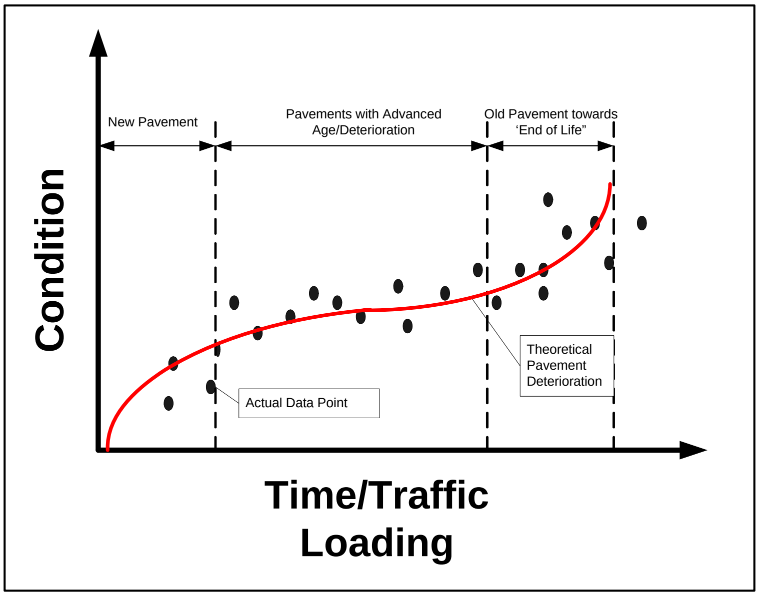


Figure 3.2: Illustration of Pavement Deterioration Stages

It is common to use the actual age classification in order to qualify the deterioration stage of pavements but this approach has some associated risk. If an experienced engineer is asked to provide an older section for the experiment, the first sections that are likely to come to mind are those with exceptionally good performance. A pure age base criteria may, therefore, not result in a truly representative selection approach.

Instead, in the State Highway LTPP programme, the stage of deterioration was also considered during the population of the design matrix. In following this approach, both under and over performing pavements were identified as part of the experiment.

Due to the roughness being an unrealistic indication of pavement deterioration (Henning and Riley 2000), cracking and rutting were used. It was found that roughness is dependent on the environment, topography and construction quality and therefore not a good indicator of age.

The parameters used to express the deterioration in the design matrix were (Henning et al., 2004 b):

- cracking (greater or less than 10% cracking);
- rutting progression (greater or less than 6 mm rutting);

- Age greater or less than 5 years and 15 years for the surface and pavement age respectively.

Based on the criteria above, a number of relatively new LTPP pavements were established.

3.6 Experimental Design for this research

Resulting from the methodology outlined in the previous paragraphs, the resulting design matrix is summarised in Table 3.4.

Table 3.4: Resulting Design Matrix Summary

Factor	Categories	Number of Cells
Environments (Sensitivity Areas)	Sensitivity areas: high, moderate, low and limited	4
Traffic Classes	High, Medium and low	3
Pavement Types/Strength	Strong and weak	2
Pavement Age/Condition	Old and new	2
Total Number of Cells (Sections established)		48 (63)

Note: the number in brackets represents the final number of sections established.

Table 3.4 shows that the total number of sections selected was 63 compared to the required 48 sections cells. The additional sections making provision for some sections to be sterilised and also providing some duplicate sections if any of the sections are ‘lost’ during the experiment (for example, some sections may be re-aligned). A sterilised site refers to a site that will have received minimum maintenance only to ensure safety for the travelling public, for example, only pothole patching will be allowed. This also satisfies the requirements for cell replication to test in-cell variation.

The outcome of the design matrix is shown in Appendix A. It presents each cell (LTPP site) according to the design matrix. The factors that describe each site can be traced on the decision tree.

3.7 Site Identification and Selection Criteria

3.7.1 Purpose of Selection Criteria

One of the challenges in establishing the LTPP sections was to ensure a representative sample of the network was included. However, in order to obtain meaningful data and ensure the survey process was practical, there were some constraints on the selection of sections for the study. These factors are discussed next.

3.7.2 Highway Alignment, Cross Section and Drainage Condition

The highway alignment/geometry was considered for the following reasons:

- Safety during surveys – since one half of the carriageway is closed during the surveys, sufficient sight distance has to be provided for the motorists approaching the LTPP sections;
- Accuracy of survey measurements – distance measurements around curves are less accurate and the location of a clearly defined wheel track is less obvious; and,
- The cross-section and drainage was creating meaningful calibration data from an analysis perspective. In order to avoid, as much as possible, ‘external factors’ on the pavement deterioration, geometric conditions have to be consistent. For example, the drainage conditions over the length of the section should be the same or very similar.

These and other factors, which were considered in the establishment of the sections are summarised in Table 3.5.

Table 3.5: Factors Considered During Site Establishment (Henning, et. al, 2004b)

Factor	Reason for Including Factor	Guideline
Horizontal Curves	Safety and data accuracy	Should be able to drive through the curve safely at 80 km / h
Gradient	Consistency in deterioration	Gradients should be less than 7%
Sag vertical curves	Variability in drainage conditions	No sag vertical curves allowed
Crest vertical curves	Safety	Sufficient sight distance for 80 km / h speed
Major drainage structures	Variability in drainage conditions and compaction	No major drainage structures allowed except if it has more than 2 m cover and does not cause a ‘jump’ on the roughness data.
Total surface thickness	Very thick chip seal layers ‘corrupts’ some models	Total chip seal surface thickness less than 70 mm

3.8 Statistical Summary of LTPP Sections Established

3.8.1 Purpose for Statistical Summary

In order to test the validity of the established LTPP sections as a function of the design matrix, a statistical analysis was undertaken. Table 3.6 depicts the descriptive statistics on completion of the second survey for all variables, which are further explained in subsequent sections.

Table 3.6: Descriptive Statistic for the State Highway LTPP Sections

Factor	Mean	Median	Minimum	Maximum	Standard Deviation
ESA	388	199	14	3301	693
Pavement Age	20.0	17.0	0.1	52.0	16
Surface Age	4.7	4.0	0.0	12.0	3
Mean Monthly Rainfall (mm)	108	98	44	247	43
SNC	3.1	2.9	0.1	7.6	1.6
ESA/SNC^{2*}	731.8	22.8	2.9	21070.0	3355.6

*Note: The ESA/SNC^2 ratio was used since it is used in this format with the HDM models.

3.8.2 Structural Number and Traffic Loading

Two methods of calculating the pavement strength (SNC) were available on the calibration sections as follows:

- Modified structural number using a regression function as described in Salt (1999). This function uses the peak deflection (D_0) and applies it to a regression function that yields the SNC;
- The back analysed method uses the stiffness from the pavement layers obtained from back analysing FWD deflections and calculates the SNC based on methods as described by Rohde (1994) and Rolt and Parkman (2000).

The distribution of the pavement strength using both these methods is illustrated in Figure 3.3. One would expect that the back analysed SNC would yield a more robust strength value since it uses all the deflection measurements from the FWD, not only the peak deflection. Furthermore, the SNC from the regression function is a function of the study area for the regression analysis, which is not always applicable to data outside the range of values it was developed for. As noted in Figure 3.3, the back calculated SNC also appeared follows a Normally Distribution. For the purpose of further analyses it was decided to use the back calculated SNC. This assumption will be tested using calibration analysis.

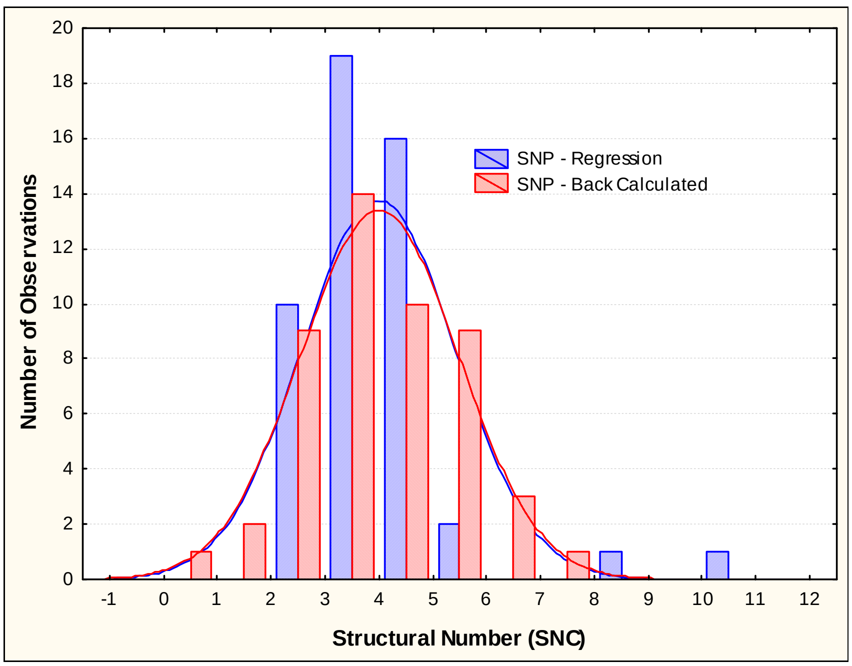


Figure 3.3: Distribution of Pavement Strength for LTPP Sections

Observations from the Figure 3.3 above include:

- The mean of the SNC is 3.1 (as given in Table 3.6) which suggests that the SNC is slightly skewed towards the stronger values; and,
- The SNC values range from 0.8 to over 7, which is a significant range.

Section 3.4 highlighted the intention of the LTPP study to investigate pavements with varying pavement strength and loading. This implies that there should be a range of sections from those classified as 'under-designed' to 'stronger designed' pavements. Table 3.6 depicts the statistical properties for the ratio between traffic loading and structural number (ESA/SNC^2). It is observed that this ratio is covering a wide range for the established LTPP sections.

The distribution is more clearly demonstrated in Figure 3.4. which illustrates the pavement strength as a function of the traffic loading (number of heavy vehicles). Some observations from Figure 3.4 include:

- There is no clear relationship between the pavement strength and the loading, suggesting that some pavements do not have sufficient capacity to carry the loads, while other pavements have much more strength capacity required for the loads;
- There is a wide range of pavement strength for most loading categories, especially for the lower volume roads with less than 200 heavy vehicles per day; and,
- As expected there are few weak pavements for the heavy traffic volumes (more than 400 heavy vehicles per day).

Based on the distribution of pavement strength and traffic loadings, it was concluded that the experimental design matrix sufficiently covered the expected ranges of traffic volumes and pavement strengths.

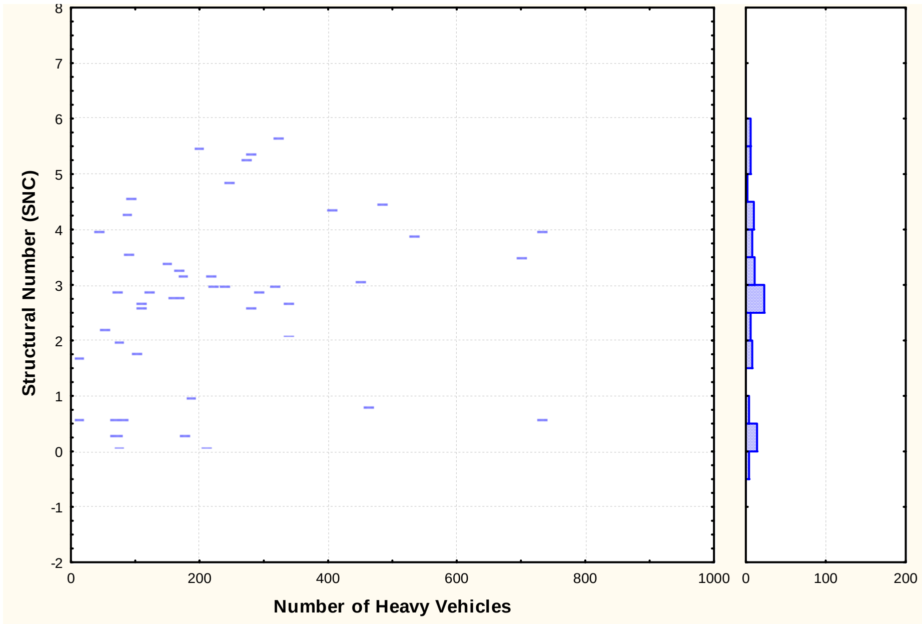


Figure 3.4: Plotting the Pavement Strength as a Function of Heavy Vehicles

3.8.3 Pavement and Surface Age Distribution

Figure 3.5 illustrates the distributions of the pavement age and the surface age. It is observed that 27% of the pavements are younger than 10 years while 46% are younger than 20 years. There are also a number of pavements older than 40 years but the validity of this data is questionable given that the database only existed for the past 15 years. Fortunately, sensitivity analysis on the HDM models suggests that the pavement models are not significantly sensitive for pavement ages older than 15 years. (Pradhan et al, 2001). The distribution of the pavement age seems to be satisfactory but will be confirmed in conjunction with the distribution of the pavement condition as discussed in Section 3.8.4.

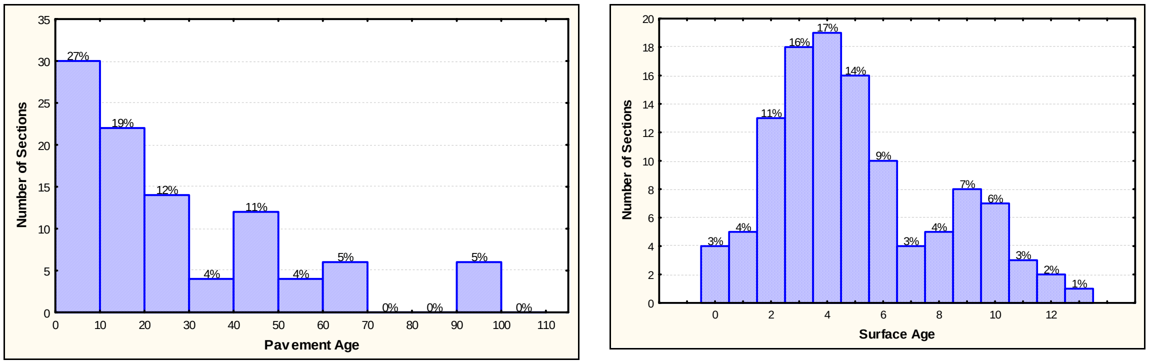


Figure 3.5: Distribution of LTPP Section Pavement and Surface Age (years)

The distribution of the surface age suggests a range of surface ages from being new to 12 years. There are a large percentage of the surface ages that are younger than 4 years. This trend is similar for the whole of the State Highway network which also has an average surface age of approximately 4 years (Transit 2000). It is therefore recognised that the results from this research may not be applicable surfaces of an older age. This factor is unavoidable given the safety requirements on State Highway network.

3.8.4 Pavement Condition Distribution

The distribution of the pavement condition was investigated and the mean rut depth was considered as being the most appropriate parameter for this purpose. Rutting is a good indicator of the pavement behaviour since it is less influenced by environmental deterioration which is common for parameters such as roughness. Figure 3.6 illustrates the distribution of the rutting as measured on the LTPP sections. It can be observed that relatively low rutting was measured. More than 90% of the sections have a mean rut depth less than 10mm. The distribution suggests that most pavements are relatively new and it does not correspond well with the observations made on the actual pavement age. Consequently, it was decided to consider the annual incremental change of rutting in order to confirm that there is a sufficient range of condition/age factors established for this research.

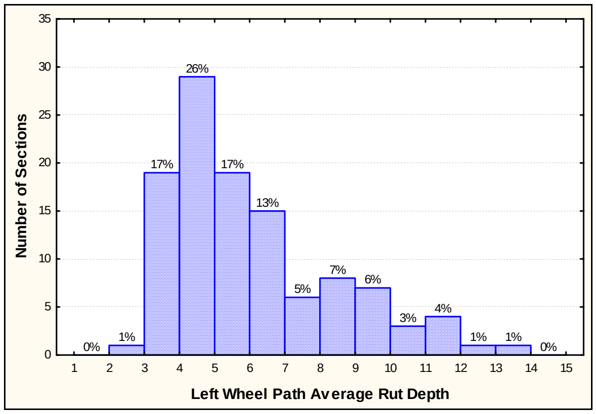


Figure 3.6: Distribution of Mean Rut Depth for LTPP Sections (Year 1 Survey)

Figure 3.7 illustrates the distribution of the annual rut change for the 2002/2003 survey rounds. This distribution suggests there is a sufficient range of condition change from one year to another and it was concluded that there is a sufficient distribution of weaker pavements with relative heavy vehicle loading and stronger pavements with relative low traffic loading. The distribution in incremental roughness changes is more important in the context of the study than the distribution of the absolute current rut depth. It was concluded therefore that the design matrix was sufficiently populated.

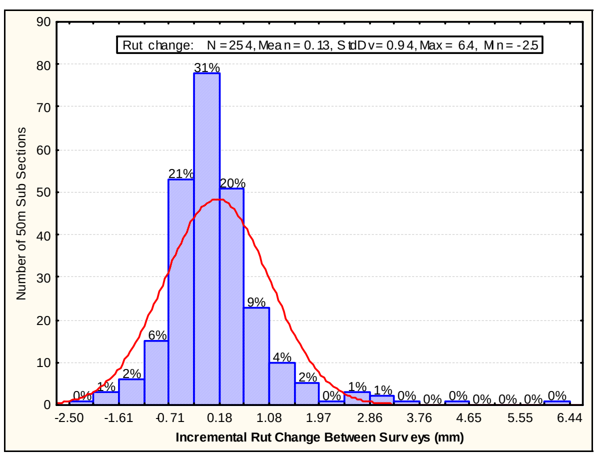


Figure 3.7: Distribution of Annual Change in Rutting (Henning, et al., 2004a)

CHAPTER 4 – LTPP DATA COLLECTION

4.1 Introduction

The purpose of this chapter is to discuss the data collection issues directly related to this research. A significant component of this research study was to develop the data collection methodology. Compared to other studies documented in Chapter 3, the data collection approach followed here was different in relation to the type of data collected and also the accuracy of the measurements.

The way in which data is collected is one of the most crucial aspects to consider, since it directly affects the results. This is even more pertinent with time-based deterioration model development. For example, if there is any bias in the data, or the scatter of the data is large due to measurement error, it becomes difficult to identify meaningful trends over time. The previous chapter has discussed the considerations taken into account for the establishment of the LTPP sections. This chapter discusses the data collection that, in essence, deals with two questions:

1. What should be measured?; and,
2. How and to what accuracy should the measurements be performed?

The first question refers to all the measurements that can be undertaken in order to qualify the condition and character of the pavement in a meaningful manner. The second question refers to the manner in which the data should be collected. Since there are various measurement techniques available to characterise pavements and its condition, it had to be ascertained which of these methods would be appropriate, given the objectives of the study and the analysis that had to be performed.

An additional challenge for this research was the fact that the data collection had to be undertaken according to a competitive tendering process. As a consequence the data collection process had to be specified as part of the tendering process. Also, given that specifications according to a method prescription may advantage certain tenderers, it was decided to develop the tender document according to an outcome based specified tender. As illustrated in Section 4.6, all the pavement condition parameters such as roughness, rutting, visual distress and location referencing had to be specified according to outcome based principles.

4.2 Theoretical Definitions and Considerations Related to the Data Collection

4.2.1 What Should be Measured?

With any study, it is important to collect data related to both the dependent and independent variables of the intended model development. However, sometimes the difficulty was, knowing which dependent variables were related to the objective of the study. Also, in most cases the actual independent variables that effect the outcome is unknown prior to the regression analysis.

With the State Highway LTPP study, the following base assumptions and study principles formulated the data collection regime (Henning et al, 2004b):

1. This research is primarily focused on the **pavement performance** of roads. It was realised at the onset of this research that there is also a significant need with regard to performance prediction related to the long-term performance of the surface types used in NZ. However, it was also realised that this long-term study will need to provide all the data required for modelling to be carried out on the performance of pavement surfaces at a later stage. For that reason most of the surface related defects were also measured, although they are beyond the scope of this research.

2. The data collection should reflect the main pavement **performance measures** commonly used in pavement management systems (i.e. roughness, rutting and visual distresses).

3. As many of the likely **independent variables** as possible should be captured. The data items considered were based on studies performed elsewhere, including the World Bank study in Brazil (GEIPOT et al, 1981), USA-SHRP

study (FHWA, 2000), South Africa (Rohde et. al. 2002) and Australia (Martin, 2003) – Chapter 2 gave a summary of the data obtained in these studies.

4. Only manual data collection methods were used for this research. However, in its analysed format, the resulting data provides measures similar to network surveys data such as HSD. This avoided the collection of specific data items that will only be appropriate for this research and no-one will able to use it for any other research applications.

The condition data collected for this research included:

- visual recording of surface distresses such as cracking, potholes, and surface damage;
- manual roughness, rutting and texture measurements; and
- routine maintenance recorded on sections including crack sealing, pothole patching and edge break repair.

Although not classified as condition data, inventory data such as pavement strength, material tests, traffic data and pavement composition were also collected.

4.2.2 Information Quality Level Adopted for this research

The data accuracy and precision for each data item is as important as the type of data collected. Adopting the wrong data information quality level may result in a specific data item being totally or partially useless for its purpose. The data Information Quality Level (IQL) concept was defined by Paterson and Scullion (1990). The IQL as explained by Bennett and Paterson (2000) specifies the accuracy and precision of data as a function of the end-use of the data as illustrated in Figure 4.1. It recognises that higher accuracy data collection costs much more and it is not warranted for application at a general planning level. However, for more sophisticated analysis and research, data collected at a lower accuracy level (IQL-3 and above) would not be appropriate.

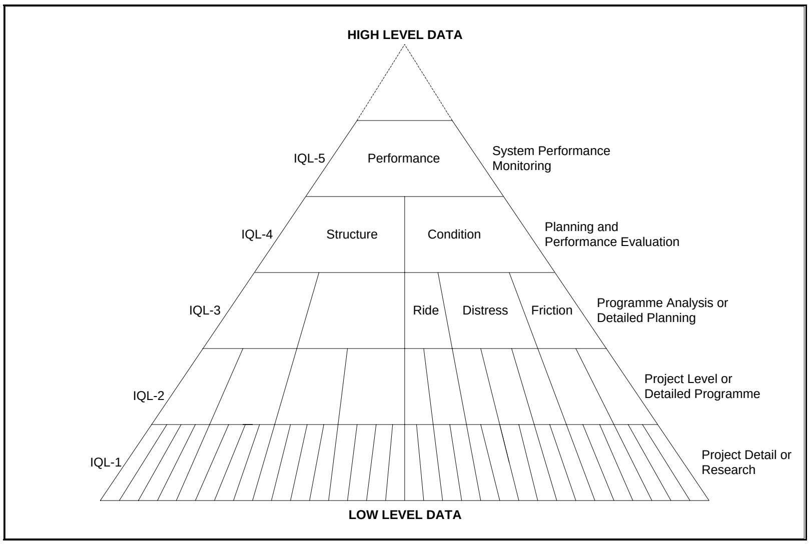


Figure 4.1: Information Quality Levels in Road Management (Bennett and Paterson, 2000)

Referring to the IQL pyramid, Table 4.1 lists the typical data collection techniques for each respective IQL. Also, Bennett and Paterson (2000) brought the IQL concept into the perspective of data needs for different pavement model calibration levels (also refer to Chapter 2).

Table 4.1: Classification of IQL Levels in Detail (Bennett and Paterson, 2000)

Level	Amount of detail
1	Most comprehensive level of detail, such as would be used as a reference benchmark for other measurement methods and in fundamental research. Would also be used in detailed field investigations for an in-depth diagnosis of problems, and for high-class project design. Normally used at project-level in special cases, and unlikely to be used for network monitoring. Requires high level of staff skills and institutional resources to support and utilise collection methods.
2	A level of detail sufficient for comprehensive programming models and for standard design methods. For planning, would be used only on sample coverage. Sufficient to distinguish the performance and economic returns of different technical options with practical differences in dimensions or materials. Standard acquisition methods for project-level data collection. Would usually require automated acquisition methods for network surveys by semi-automated methods or combined automated and manual methods for network surveys and use for network-level programming. Requires reliable institutional support and resources.
3	Sufficient detail for planning models and standard programming models for full network coverage. For project design, would suit elementary methods such as catalogue-type with meagre data needs, and low-volume road/bridge design methods. Able to be collected in network surveys by semi-automated methods or combined automated and manual methods.
4	The basic summary statistics of inventory, performance and utilisation, of interest to providers and users. Suitable for the simplest planning and programming models, but for projects is suitable only for standardised designed of very low-volume roads. The simplest, most basic collection methods,

	either entirely manual or entirely semi-automated, provide direct but approximate measures, and suit small or resource-poor agencies. Alternatively, the statistics may be computed from more detailed data.

Most other model development and calibration studies, such as the studies discussed in Chapter 2, used data collection methods that can be classified between IQL-1 and IQL-2. This suggests that most condition surveys were undertaken either by means of high speed devices (HSD) or manual measurements. Experience on the Gautrans calibration study suggested that condition data at an IQL-2 may not be appropriate for model development purposes (Rohde et al, 2002). Although the HSD equipment is capable of measuring to a high precision level, it is not capable of high accuracy in terms of repeatability and reproducibility This is discussed further in the following sections. The IQL alone does not necessarily specify what equipment type and measurement methodology should be used for a study.

4.2.3 Repeatability and Reproducibility

Condition survey equipment accuracy has improved remarkably over the years. However, the mere fact that equipment has a capability of measuring accurately does not necessary means that the equipment system will provide repeatable [1] and reproducible[2] results. For this reason, most network data contracts will include some specification that requires a validation/calibration process which the contractor must undertake prior to undertaking network surveys (Transit, 2002). It was therefore expected that the data collection for this research would be even higher than the repeatability and reproducibility criteria required for network surveys.

Given the incremental approach of existing HDM-4 models, it was imperative to conduct the surveys in both a repeatable and reproducible manner. At best, time

[1] Repeatability is an indication of a measuring system being able to measure a consistent value when the measurements are repeated in the same location without moving the measurement equipment.

[2] Reproducibility is an indication that a measurement in one location would be statistically the same as a measurement undertaken in the same location after some time has past, and the equipment was re-established in the same location.

Note: Conflicting definitions have been found in literature for these two terms. The definitions above are therefore applicable for the context of this research only.

series data for road condition is a challenge to interpret since it does not always demonstrate a gradual deterioration over time. With large scatter of the data due to poor repeatability, the actual condition change is over shadowed by the noise in data (apparent variability in the data that is attributed by data collection error). Although the level of repeatability between various equipment types differs, comparable results could be obtained through requirements such as number of measurements. For example, one measurement using equipment Type A may provides statistically the same result as the average of three measurements from equipment Type B. This assumes that type A is the more accurate device of the two. This technique is discussed further in Section 4.6.

Reproducibility however, is more difficult to achieve with more complex measuring devices such as High Speed Data (HSD) acquisitions systems. The reproducibility is a strong function of the ability to re-locate the instrument in the exact location as previous measurements. Naturally, this is easier to achieve with manual measurements, compared to HSD measurements which are conducted in a fast moving vehicle. This point has also highlighted the importance of location referencing for the data collection specifications.

4.2.4 Precision and Bias

Bennett and Paterson (2000) have illustrated in Figure 4.2, the different combinations of bias and precision levels achieved with condition measurements. Naturally, the ideal would be if all the measurements can be performed with high precision and no bias, but that is not a realistic expectation.

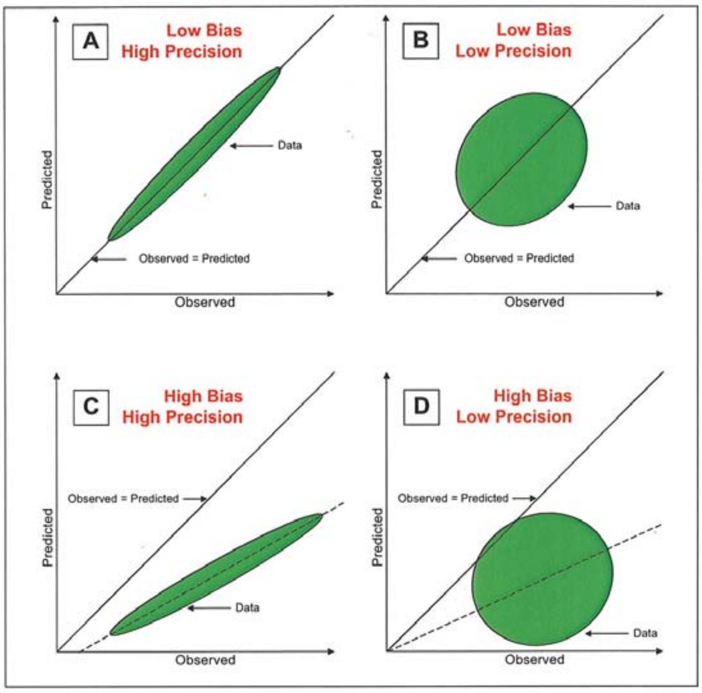


Figure 4.2: Bias and Precision (Bennett and Paterson, 2000)

For this research, the bias and precision considerations have been addressed through the specifications discussed later in Section 4.6. It is accepted that all equipment will have a varying degree of precision and bias that can be accounted for, through processes discussed in the following paragraphs.

Precision of measurements is a strong function of the equipment type. An instrument that can achieve the precision requirements with a limited number of measurements will produce similar values to an instrument with less accuracy/precision capabilities, but taking a number of repeated measurements. For example, Brown and Fong (2001) have established that comparable roughness measurement results can be obtained by one measurement using a FACE Dipstick and three measurements using the ARRB Walking Profilometer. However, using the FACE Dipstick measurements take approximately three times longer, compared to the ARRB Profilometer. For the purposes of this research, it was therefore important to know a) what level of precision would be required and b) it had to be specified in such a manner that it would be fair to all the equipment alternatives available to the potential contractor. Therefore, lower precision equipment was acceptable but in using it, more measurements were required in order to deliver a statistical comparable measurement.

Many harmonisation studies have demonstrated that there is always an expected bias associated with each condition survey instrument. Figure 4.3 illustrates a comparison between measurements of different devices. It is observed that the measurements ranged from an IRI of 160 to 220 in/mi. (approximately 2.4 to 3.5 m/km) for the first section and between an IRI of just over 60 to 140 in/mi for section 2, the smoother section of the two.

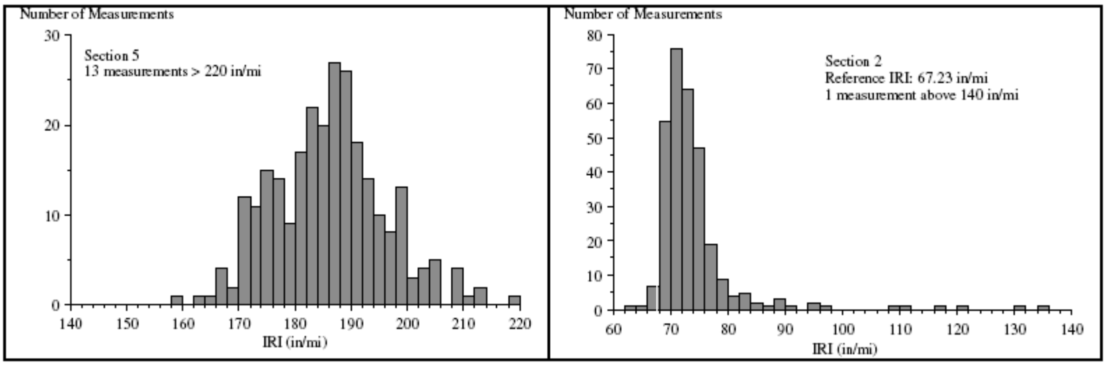


Figure 4.3: Example of Range of Profiler Measurements from Comparative Study (Karamihas, 2004)

Although precision was a major concern for the condition measurements during this research, the bias was of a lesser concern, as long as the same equipment was used for the duration of the study. As mentioned earlier, the data analysis for this research was to be undertaken on incremental data (i.e. change from one year to another). The change in condition from one year to another is relatively small. Also, any bias from a specific instrument is limited by a change in condition rather than an absolute condition value in time. Therefore, the change in condition measured with one instrument will be comparable to another instrument, despite an apparent bias between the two devices.

4.3 Roughness Measurements

4.3.1 Roughness Measurement Technology

Road roughness is one of the most commonly used road condition measurements. The development of the measurement and description of roughness is one of the most standardised road condition measurements. The popular use of the roughness measurement can be explained by its applications and significance including:

- The road users' comfort/road quality perception is directly influenced by the roughness. For this reason roughness has always been an important performance indicator of pavements;
- The road roughness has a direct impact on the vehicle's performance and maintenance costs. Many engineering and transport economic studies have led to the quantification of Vehicle Operating Costs (VOC) and Road User Costs (RUC). Road User Costs include the VOC, but also considers other user costs such as user time-costs. As a consequence, roughness has a direct input into economic evaluation of pavement design and rehabilitation options (Transfund, 2004); and,
- Roughness has also been related to the deterioration of road pavements, which resulted in the development of road roughness deterioration curves such as the HDM roughness model (Watanatada et al, 1987).

ASTM E867 (1987) defines roughness as "*the deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and pavement drainage*". The most commonly used unit of measurement for roughness is the International Roughness Index (IRI). The IRI is calculated from a quarter-car (see Figure 4.4) simulation.

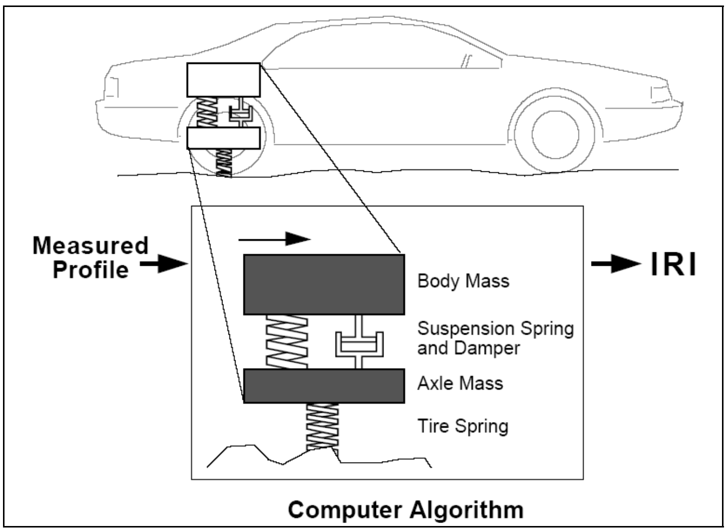


Figure 4.4: Quarter-car Computer Simulation (Sayers & Karamihas, 1998)

> "*The quarter-car filter calculates the suspension deflection of a simulated mechanical system with a response similar to a passenger car. The simulated suspension motion is accumulated and divided by the distance travelled to give an index with units of slope (m/km, in/mi, etc.).*" Sayers & Karamihas (1998)

It should be realised that the definition of roughness has undergone generations of development, similar to the technology of measuring the roughness. Early roughness measurements involved a subjective rating from the user's perspective. The first profiling technology was used in the 1960's with Elson Spangler and William Kelly developing the inertial profilometer at the General Motors Research Laboratory. Subsequent to this development, roughness measurements were performed by an array of technologies including:

- Response type measurements that rely on the reaction of the vehicle suspension to the unevenness of the pavement;
- Direct profile measurements such as rod and level, FACE Dipstick and ARRB Walking Profilometer; and,
- Modern High Speed Data measurement equipment that includes laser displacement measurements and accelerometers.

Regardless of the measurement equipment used, none can directly measure roughness in its defined unit of measurement, namely IRI in m/km. Most techniques establish a road profile as illustrated in Figure 4.5, from which the roughness is calculated using the quarter-car filtering process.

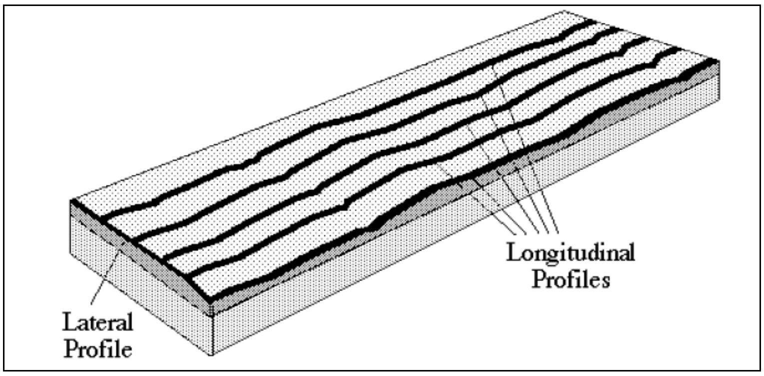


Figure 4.5: Road Profile (Sayers & Karamihas, 1998)

The accuracy of this roughness measurement was an important consideration for this research. For that reason, it was equally important to consider the possible sources of the measurement errors as explained in the following section.

4.3.2 Factors that Influence the Accuracy of Roughness Measurements

Everything involved with the measurement and analysis of roughness can cause errors, including the operator, equipment, the road characteristics and the way in which the profile is analysed. For research of this kind it was important to consider these sources of error in order to either eliminate, limit, or at least be aware of, potential areas of inaccuracy or bias.

Table 4.2: Potential Roughness Measurement Errors and Mitigation Adopted

Factor	Error Description	Mitigation Adopted for This research
Human/equipment	Location referencing – repeatable and reproducible measurements require an accurate reproduction of profile observation /measurements in exactly the same location as previous measurements. This is something which is very difficult to achieve using high-speed type equipment (refer to the paragraph below this table).	Manual measurements were specified in combination with very strict location positioning of LTPP sections.
	Accuracy of Measurements – all	Specifying accurate measurement

Factor	Error Description	Mitigation Adopted for This research
	equipment components plus the incorrect use thereof results in measurement errors. It is however accepted that varying accuracy expectations exist for different equipment types.	expectations plus ensuring a comprehensive equipment calibration process.
	Measurement Failure During Surveys – Something can go wrong during the surveys as a result of equipment failure or operator mistakes.	Repeated measurements are specified plus a real-time validation with prior surveys is requested.
Analysis Algorithm & System	The analysis algorithm may result in errors/ bias. e.g. the system and/or algorithm include some filtering mechanisms in order to interpret the appropriate profile wavelength and convert it into a roughness value.	Only one algorithm/system is used during all the surveys. Any change in the system have resulted in the re-analysing of all profile data
Equipment	Equipment Associated Bias -> The inherent characteristics of equipment results in a bias associated with a particular device. e.g. footprint of measurement area.	Only one equipment type is to be used for the duration of the survey period. Again, the absolute roughness is not as important as the roughness change from one year to another thus removing the impact of bias in the measurements.
	Known Equipment Limitations -> There are some known equipment associated limitations. Many of these are associated with the automated type measurements.	All automated type measurements were ruled out from the contract through the accuracy and repeatability specifications.
Physical Road Condition	Physical Road condition may influence measurements -> Sometimes there are secondary conditions that may influence results such as bleeding of the surface, standing water on the surface and or changes in the physical condition from one year to another.	Measurements are avoided in certain instances such as standing water or when some condition aspect influences the measurements. Any measurements that fall outside of reproducibility criteria must be supported by field notes, explaining the reason behind the “strange” measurements results.

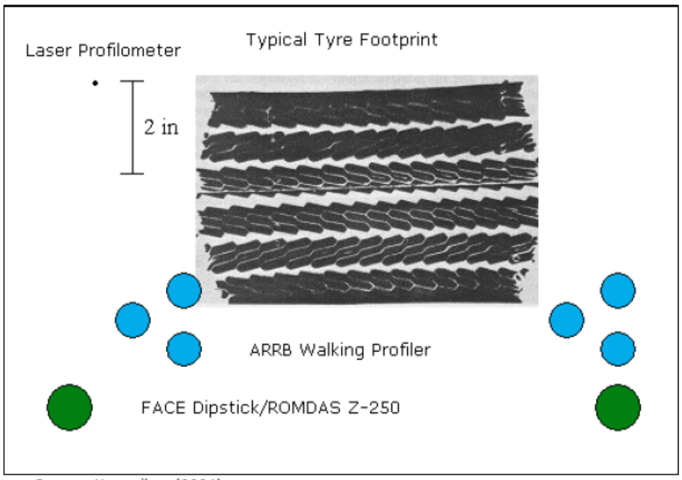
Sayers & Karamihas (1998) discuss how the longitudinal and the transversal positioning influence the measurement accuracy. Henning et. al. (2004b) also attributed this as one of the major factors explaining the difference in results between HSD and manual measurements.

Calibration procedures specified for the LTPP State Highway data collection contract were to achieve the following (Henning, 2001b):

- "*Ensure satisfactory repeatability and reproducibility of the measurements by traceability to international standards;*
- *Provide evidence of continuing measurement stability;*
- *Define any limitations of the equipment;*
- *Define factors influencing the results and how the correction procedures of the factors are applied*."

Figure 4.6 illustrates the relative difference in footprint for the different measurement instruments. One of the major effects resulting from these differences is observed by the way these instruments are affected by the surface texture. For example, on a large chip seal surface, a very small footprint, such as the footprint from a laser, would appear to have a higher roughness compared to a much larger footprint such as that from a vehicle tyre. During the computer data processing, there is a filter which will remove short wave-length measurements such as the surface texture effect. However, this filter is instrument specific and does not completely remove these effects recorded by different measurement instruments.

During the HSD network surveys on the LTPP sections, it was observed that those which were resurfaced had an increased roughness which 'smoothed out' within the first two years after resurfacing. However, this trend was much more visible on the LTPP data (which used the Walking Profilometer) than the laser measurements used during network HSD surveys of the State Highways. This is illustrated in Figure 4.7. It is suspected that the filters on the laser roughness calculations removed most of the short length frequencies, which cannot be isolated from the Walking Profilometer measurements.



Source: Karamihas (2004)

Figure 4.6: Comparison of Measurement Footprint from Different Instruments

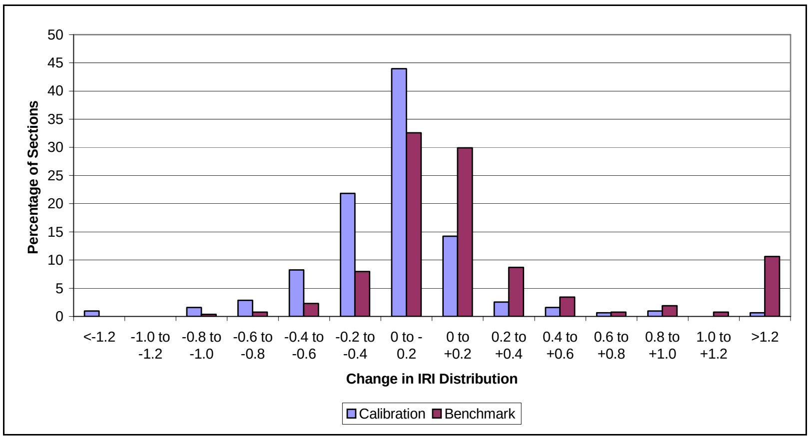


Figure 4.7 Comparing Incremental Roughness Change Measured for Different Equipment Types (Henning and Furlong, 2005)

Figure 4.7 shows incremental roughness changes for measurements taken by the laser profilometer (Benchmark) and the Walking Profilometer (Calibration). The negative change means a roughness improvement which was observed on newly resealed and reconstructed resurfacing. Subsequent investigations confirmed a strong relationship between surface texture and roughness when measured with the Walking Profilometer.

4.4 Rutting Measurements

4.4.1 Rutting Measurement Technology

Rutting has a dual importance in most pavement management systems. Firstly, the physical rut depth is an important parameter from a safety perspective. Aquaplaning may become an issue if the combined factors of cross-fall, longitudinal alignment and rut depth cause water ponding. Furthermore, the rut progression is an important performance measure for roads as it is a strong indicator of the structural behaviour of the pavement under traffic loading. Normally, it is accepted that ruts start to develop as a result of densification within the subgrade once the pavement has reached the design traffic loading. It can also indicate some early failures originating from either the densification of the subgrade or even shear failure in the base or subbase layers (Visser, 1999).

The rutting measurement has experienced rapid technical development over the last decade. The original and simplest method of measurement is the straight edge and wedge method, which is still used today. However, this method is very slow and impractical when large numbers of measurements have to be performed. More recent technologies include an automated measurement of a transverse profile. The height measurements are taken at different offsets and are converted to an equivalent rut depth through computer simulations which are discussed later. Some measurement methodologies include (Mallela and Wang, 2006):

- **Lasers** – different configurations of lasers measure the cross profile of the road and a rut depth is calculated from computer simulations. The most commonly used laser systems consist of point lasers that measure the transverse profile heights at different offsets. More modern lasers scan the road profile continuously with a high speed laser capable of scanning the cross section of the road at high speed;

- **Optical Imaging** – the rut depth is electronically determined from optical images of the transverse road profile. With some systems the optical images may be analysed jointly with some laser profiles;

- **Ultrasonics** – this low cost option uses the measurement of sound waves to determine the cross profile and rut depths.

4.4.2 Factors Influencing the Accuracy of Rutting Measurements

Similar to roughness, the factors influencing the accuracy of rut measurements includes all aspects of the measurement process. The recognised factors which influence the accuracy of the measurements plus the mitigating measures adopted for this research, are summarised in Table 4.3.

Table 4.3: Potential Rutting Measurement Errors and Mitigation Adopted

Factor	Error Description	Mitigation Adopted for This research
Human/equipment	Location referencing – repeatable and reproducible measurements require an accurate reproduction of profile observation /measurements in exactly the same location as previous measurements.	Manual measurements were specified together with very strict location positioning of LTPP sections.
	Accuracy of Measurements – all equipment components plus the incorrect use thereof results in measurement errors.	Specifying accurate measurement expectations plus ensuring a comprehensive equipment calibration process.
	Measurement Failure During Surveys – Something can go wrong during the surveys as a result of equipment failure or operator mistakes.	Repeated measurements are specified plus a real-time validation with prior surveys is requested.
Analysis Algorithm & System	The analysis algorithm may result in errors/ bias. (See Section 4.2.4)	Only one algorithm/system is used during all the surveys. Any change in the system would result in the re-analysing of all profile data. In addition to this, a 2-m straight edge simulation method was prescribed.
Equipment	Equipment Associated Bias -> The inherent characteristics of equipment results in a bias associated with a particular device. Spacing of transversal measurements.	Only one equipment type is to be used for the duration of the survey period. Measurements taken in subsequent year are compared with previous measurements.
	Known Equipment Limitations ->	All automated type measurements were

Factor	Error Description	Mitigation Adopted for This research
	There are some known equipment associated limitations. Many of these are associated with the automated type instruments. Refer to the paragraph below this table.	ruled out from the contract through the accuracy and repeatability specifications.
Physical Road Condition	Physical Road condition may influence measurements -> Sometimes there are secondary conditions that may influence results such as bleeding of the surface, standing water on the surface and or changes in the physical condition from one year to another.	Measurements are avoided in certain instances such as standing water or when some condition aspect influences the measurements. Any measurements that fall outside of reproducibility criteria must be supported by field notes, explaining the reason behind "strange" measurements results.

The most significant consideration of the rut measurements for this research was the equipment limitations and the influences these factors have on the accuracy of the rut measurement. These factors include the spacing of transverse measurements, the transverse placing of the measurement device and the profile analysis method. The latter factor is discussed later in Section 4.4.3.

It is well known that the spacing of the transverse measurement affects the accuracy of the analysed rut depth (Simpson, 2001). In their rutting harmonisation project, Mallela and Wang (2006) investigated the impact of the measurement spacing. Figure 4.8. compares the resulting rut depth from varying measurement configurations. The actual rut depths from a full profile were 8.0 and 5.3 mm for left and right rut depths respectively. In comparison, the rut depths from different measurement spacings varied from 6.8 to 7.4 mm for the left wheel path and 2.6 to 3.5 mm in the corresponding right wheel path. From these observations, it is clear that there may not be a direct relationship between the number of sensors versus the ability to cover the true low point of the rut. However, it can be assumed that more sensors would be more likely to identify the deepest rut point than a limited number of sensors.

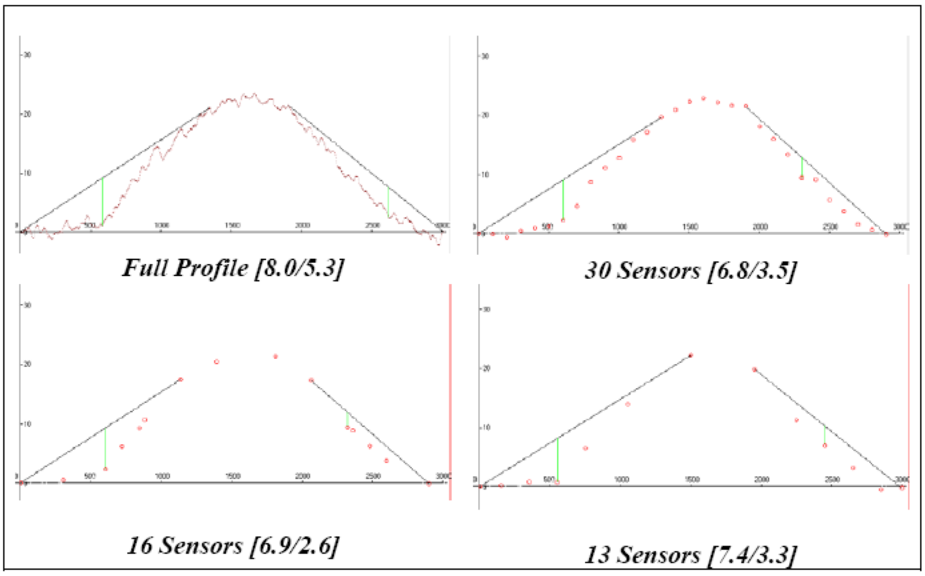


Figure 4.8: Effects of Sampling from Three Different Instruments (Mallela and Wang, 2006)

The transverse positioning of the rut measurement also influences the outcome of the measurements with the following considerations pertinent:

- The measurements have to be consistent between years in the case of non-continuous profiles. For example, if a fixed measuring interval is used (say 100mm) the lateral positioning must remain unchanged between years. Varying the lateral positioning may result in inconsistencies, thereby increasing or decreasing rut depths because of a “moving” deepest rut point;
- The measurements have to determine the full rut depth;
- The measurements have to take account of the analysis process. For example, if the first measurement is on an elevated point (in case of kerbs) an unrealistic rut may be generated during the analysis process.

In order to address the issue of sampling interval and lateral positioning in this research, a simplistic approach was adopted based on the following assumptions:

- Firstly, it was assumed that most ruts have an average width of 500mm; and,
- The actual spacing of the rut depth varies for different lane configurations and also for prevailing travelling patterns for a given alignment and road profile width.

As a result, it was decided to require at least 10 measurements within a standard rut. This equates to a measurement interval of 50mm. The contract specified a profile for the full width of the pavement. Laterally, all measurements started at the edge line. The referencing of the edge line was also recorded. This was undertaken to ensure a consistent reference in the case of resurfacing when the edge line may shift over time.

In terms of the objectives of this research, the expected accuracy requirements for the rutting measurements had to exceed the expected incremental annual change in rutting, which was less than 0.5mm per year. Knowing the lack of repeatability with HSD type equipment (Rohde et al, 2002), it was decided that a manual type measurement would be most appropriate for this research. No specific instrument was recommended in the survey specifications (refer to LTPP data collection presented in Table 4.4). Ultimately, the successful tenderer proposed an automated transverse profile beam (TPB) which is illustrated in Figure 4.9. This device records the relative height displacement at 30mm offsets (transversally).

Figure 4.9: Transverse Profile Beam

4.4.3 Transverse Profile Analysis

Various profile analysis methods are currently in use in order to determine the rut depth. Some examples are listed in Figure 4.10 below. All of these methods describe a basis or benchmark reference for calculating the rut depth from a transverse profile.

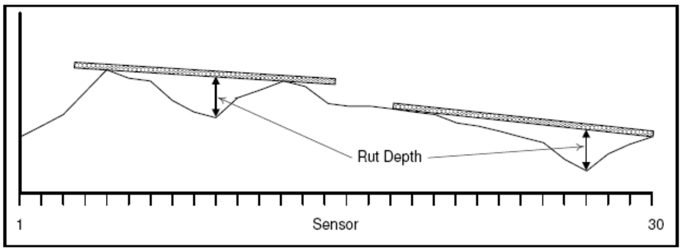
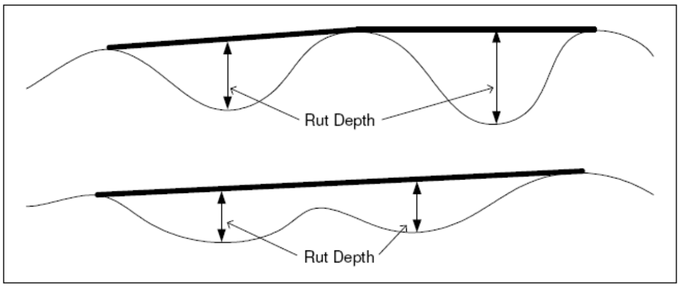
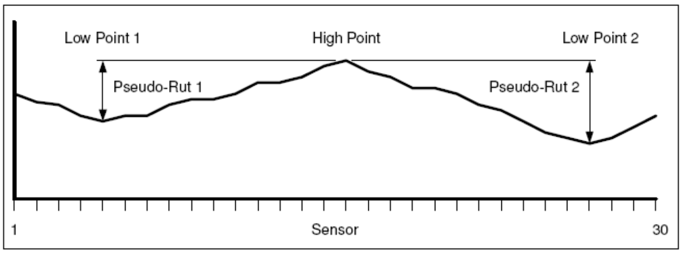
Profile Analysis Method	Graphical Illustration of Analysis Principle
Straight Edge Method	
Wire Method	
Pseudo Rut	

Figure 4.10: Transverse Profile Analysis Methods (Bennett et al, 2006)

No preference was placed on the actual analysis process for the LTPP data collection, since the raw profile data was also required. The selected analysis method adopted by the contractor was the straight edge method, using a 3m virtual straight edge. This was accepted given that the HDM rut depth models are based on the same approach.

4.4.4 Accuracy of Rutting Measurements Achieved

The rut measurement accuracy and repeatability obtained during this research was acceptable for the given objectives and assumptions made during the onset of the research (refer to Section 4.6). It was also observed that the annual rut change was relatively small but correlated with the expected values during the onset of the research. Figure 4.11 illustrates the distribution of rut changes for the first three survey periods of this research. All results are plotted in these figures, and the negative values represent sections which have been maintained, and some which showed a minor rut improvement which was within the measurement tolerance. It can be concluded from the plots that the majority of sections had an incremental rut change between -1 and 1 mm.

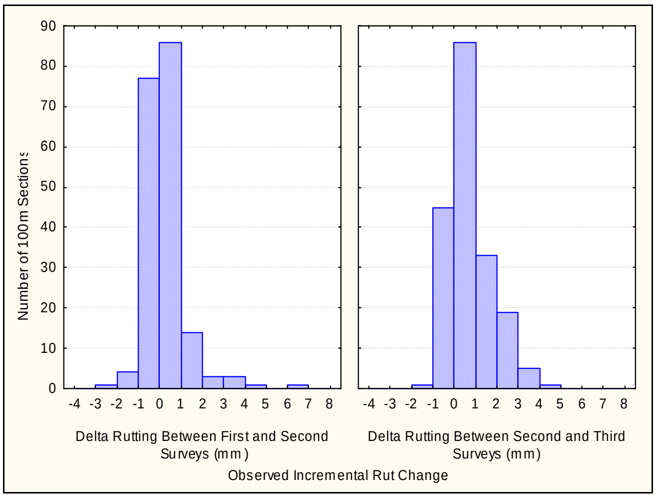


Figure 4.11: Comparing Rut Change for the Different Survey Periods

4.5 Visual Surveys

The visual distress modes were recorded manually during the survey period. Various survey methods have been considered, with the most appropriate method being developed by Van Zyl (1994). The main benefits offered through this method include the level of detail in recording the distresses, and the relatively good match with current HDM defect definitions (GEIPOT, 1981).

For the LTPP visual data collection development, the methodology of Van Zyl (1994) was further expanded to cater for some specific needs including:

- Tracking the history of defects is vital for modelling purposes. For example, even though cracks have been sealed, it is important to record them as such. During the analysis one can then get an overall view of how intensive certain defects are, which includes existing cracks, plus cracks which have been resurfaced;
- The location of defects is important since interrelationships between defects are investigated. For this research, all defects were recorded for individual 50m subsections; and,
- There was a strong emphasis on supporting the data with additional information such as site notes and photographs. Any variation in condition changes from one

year to another can be explained. This assists validation of any routine maintenance which has been performed on sections without being recorded. Also, any unexplainable data trends can be explained from a practitioner's point of view. For example, some sections may indicate "disappearing cracks" – simple site notes such as "extensive bleeding of the chip seal surface covered prior cracking" can explain the situation.

All distresses were to be recorded in metres (linear metre for linear items and length by width for areas affected distress types). Instruments for these measurements were not a requirement, and as the measurements could be performed by pacing or estimating the lengths to an accuracy of +/-50mm.

All measurements were recorded in metres to the second decimal (e.g. 1.85m). The recording of the distresses was undertaken according to the rating sheet depicted in Figure 4.12.

Subsequent to the rating, each distress was expressed as a percentage of the total carriageway (area = 50m x surfaced width - one direction). The area of linear cracks was taken as the linear length multiplied by a width of 0.5m, similar to the HDM definition (GEIPOT, 1981).

TRANSIT NEW ZEALAND
ARARAU AOTEAROA

CALIBRATION VISUAL RATING FORM

SECTION ID: ____________ RATOR: ______________________________ SH: ________________ RP
SEGMENT: Displacement from ____________ to ________________ DIRECTION: I/D ______ DATE:

LANE WIDTH ________________ TERRAIN: Mount

SURFACING

	None	Length	Width	Length	Width	Length	Width	Length	Width	Length	Width	% AREA	Total AVE DIA
SURFACE DISINTEGRATION													
Active Loss													
Stable													
Delamination													
Mech Damage													
BLEEDING/FLUSHING													
Flushing Seriousness	L	M	H										

STRUCTURAL CRACKS

	Narrow < 3mm						Wide >3mm						
	None	Length	Length	Length	Length	Length	Length	Length	Length	Length	Length	NARROW %	WIDE %
LONGITUDINAL													
Edge Cracking													
Wheel Track													
Irregular													
TRANSVERSE CRACKS													
ALIGATOR CRACKS													
General													
Wheel Track													
PARABOLIC (shift of seal)													

	None	Length	Width	Length	Width	Length	Width	Length	Width	Length	Width	% AREA	AVE DIA
PATCHING													
Surface Patch													
Structural													
POTHOLING													
Potholes													
Edge Breaks													
FAILURES													
Shoving													

Figure 4.12: Typical Visual Rating Form (Henning, 2001)

4.6 Survey Specifications

4.6.1 Performance Based Specifications

For the data collection of the State Highway LTPP sections, as noted in Chapter 3, Transit decided to appoint a private contractor. Hence, the data collection contract had to comply with Competitive Pricing Procedure (CPP) requirements (Transfund, 1997).

The contractual requirements had an additional benefit for this LTPP research, in the sense that precision, repeatability and reproducibility requirements had to be specified in detail. This approach is different to many other situations where researchers are often more focused on prescribing the method of data collection. The additional benefits were realised in that the tender had to specify exactly what the desired outcome had to be, whilst this aspect is not always well thought through in other research projects. Therefore, research projects are also constrained with regards to the availability and/or the affordability of equipment. However, in this LTPP data collection, the risk of delivering to the specifications was placed on the contractor, and it was up to the contractor to decide what equipment would be appropriate for the contract. As a result, a large part of the tender required the contractor to demonstrate that the proposed equipment could meet the specifications.

Table 4.4 lists the data collection specifications adopted for the LTPP survey contract on the State Highway LTPP sections. The rationale behind every condition measurement is discussed in more detail in Table 4.4.

Table 4.4: Calibration Survey Contract Specification (Henning, 2001b & Henning, et. al, 2004b)

Item	Measurement Specification	Measurement Interval and Reporting
Roughness	Demonstrate the number of repeated measurements required providing the same accuracy as specified for the following instruments: Walking Profilometer. Each wheel path must be measured with three repeated runs, which are within 5% (± 2.5%) repeatability according to the student t-statistical method (on 100m lengths). Face Corporation's Dipstick. Each wheel path only has to be measured only once if the closing height is within the following limits (start left wheel path, measure right wheel path in opposite direction). The readings had to close at the start point within a relative height difference of less than 60 mm. The accuracy of the distance measurements within the section (survey measurements) had to be less than a 3% error.	The roughness data had to be collected in both wheel paths and sampled and stored at intervals not greater than 250 mm and reported for 10m and 100m intervals.
Rutting	The transverse profile equipment must be capable of measuring to an accuracy of 0.5mm. The number of repeated measurements required on all sections would be established during the validation. It will be determined by taking the number of measurement runs required to achieve a standard error less than 0.3mm. The contractor is requested to demonstrate that the accuracy specified could be achieved with the nominated instrument. The contractor also had to nominate the algorithm used for calculating the rut depth from the transverse profile and document any filtering that is used on the electronic data.	The transverse profile has to be measured and reported at 10m intervals. The individual transverse profiles consist of transverse measurements spaced at less than 50mm. The rut depth is defined as the maximum rut depth resulting from the measurements using the 2m straight edge method.

Visual Condition Rating	All measurements of distresses have to be undertaken to an accuracy of ± 0.05m.	All measurements must be reported in metres to the second decimal (e.g. 1.85m). The recording of the distresses can be undertaken electronically or according to the rating sheet method.

Measurements performed in subsequent LTPP surveys have to comply with the following requirements[3]:

> *"The repeatability between consecutive years shall satisfy the following (these clauses are only valid for surveys in years 2, and 3, and subsequent years, if appropriate):*
>
> ***Roughness***
>
> *i The R^2 correlation is at least 0.85 for the 12 (six/wheel path) individual roughness values for each 50 m sub-section regressed against the roughness values of the previous year; and,*
>
> *ii The roughness change (CH) % is not greater than 15% within each sub-section.*
>
> ***Rutting***
>
> *i The R^2 correlation is at least 0.85 for the 12 (six/wheel path) individual mean rutting values for each 50 m sub-section regressed against the rutting values of the previous year; and,*
>
> *ii The mean rutting change (CH) % is not greater than 20% within each sub-section.*
>
> *where:*
>
> *Xn1 is the mean rutting on the 50m section for the previous survey.*
>
> *Xn2 is the mean rutting on the 50 m section for the current survey"*

[3] Extract from Transit (2001)

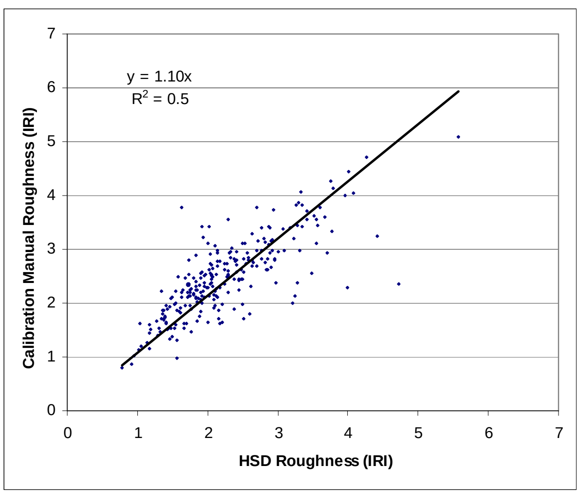
4.7 Discussion on Appropriateness of Data Collection Regime

4.7.1 Information Quality Level

The data collection IQL for this research is much higher than the other LTPP studies reviewed in Chapter 2. During the third survey year (2003), the data was reviewed to establish the appropriateness of the data collection regime. This was established by comparing the LTPP manually measured data to HSD network data. The HSD data for the LTPP sections consisted of four repeated measurements in each direction on all the LTPP sections.

4.7.2 Comparing LTPP with HSD Roughness

A total of 251 sections, each of 100m length has been used for the roughness comparison between HSD and manual Walking Profilometer measurements. Figure 4.13 illustrates the outcome of the regression analysis, which shows a regression coefficient of 1.1 with a zero intercept and a R^2 of 0.5. The results show also that for roughness levels higher than about 3 IRI, the HSD measurements over-estimate the roughness.



Note: N = 251

Figure 4.13 Comparing HSD Roughness and Calibration Walking Profilometer Measurements (Henning et al, 2004)

This is in contrast to roughness levels lower than about 3 IRI where typically the manual measurements were higher than the HSD readings. As expected, more variability between the measurement types was observed for higher roughness readings. Overall, it seems that the correlation between the HSD and manual roughness measurements was reasonable and no significant bias between the measurements is apparent.

The difference between the measurement types for individual 100m sections is depicted in Figure 4.14. The difference between the measurements was established by subtracting the HSD measurement from the manual measurement. For only 15% of the sections, the difference between manual and HSD readings was less than 0.05 IRI. This difference is significant since the expected annual roughness change on New Zealand roads is between 0.05 and 0.1 IRI. Therefore, the required accuracy cannot be achieved with HSD equipment.

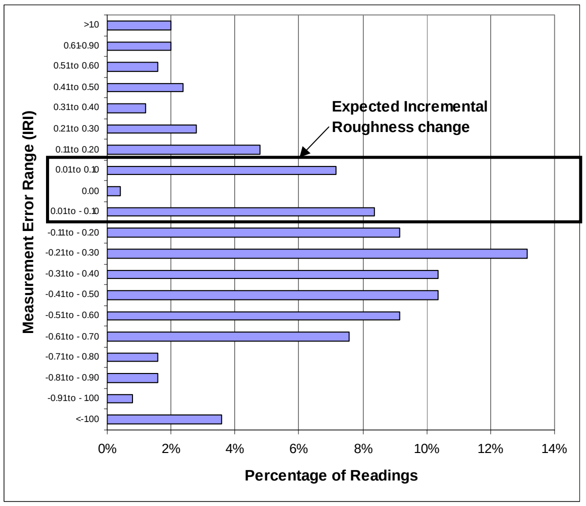
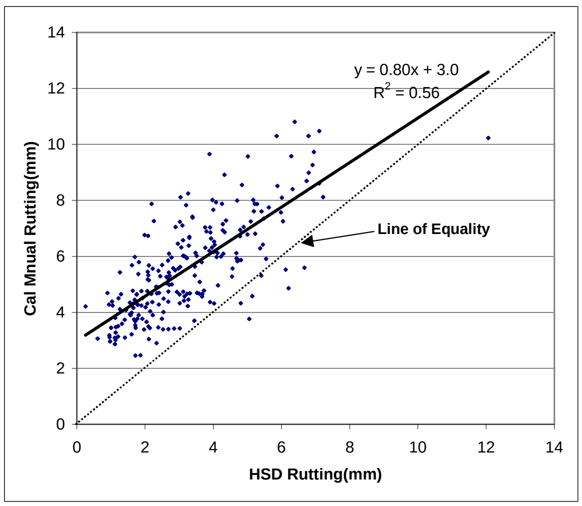


Figure 4.14: Comparing HSD Roughness Measurements with Manual LTPP Data (Henning et al, 2004)

4.7.3 Comparing LTPP with HSD Rutting

For rutting, the same comparisons were undertaken and this resulted in a regression coefficient of 0.8 and R^2 value of 0.6. For the manual and HSD rutting, only the outside wheel track was used for the comparison. A total of 229 section averages, each section 100m in length, were included in the comparison.

Figure 4.15 shows a bias of approximately 3mm for the HSD rut depths (i.e. suggesting the HSD underestimates the rutting by an average of 3mm). A visual comparison of the regression line and the line of equality clearly illustrate the bias in the HSD measurements. The transverse profile beam manual rutting measurements performed on the calibration sections were confirmed with samples of straight edge and wedge readings on all sections. The comparison between individual sections was not undertaken since the regression clearly illustrates that the variation and bias recorded for HSD rut measurements exceeds the expected 0.5mm annual rut change typical for New Zealand. For this reason, no direct comparisons were undertaken for the individual sub-sections.



Note: N = 229

Figure 4.15 Comparing HSD Rutting and Calibration Transverse Profilometer Measurements (Henning et al, 2004)

Conclusions from these comparisons included:

- The pavement condition measurement specifications defined for the LTPP data collection are appropriate. In particular, it is possible to yield data to an accuracy required to investigate condition changes from one year to another;
- The same conclusion cannot be drawn for network HSD measurements. Although these measurements are appropriate and cost effective for network surveys they should and cannot be used for the development of pavement prediction models to the required accuracy adopted for this research. It has

been demonstrated that the variances of these measurements are too great for gaining an understanding of the factors that influence pavement deterioration.

On the basis of these early results from this research, the State Highway LTPP data collection has continued on the same basis.

CHAPTER 5 PREDICTING CRACK INITIATION

5.1 Introduction

Various forms of cracking can appear on roads including:

- Load associated cracking, which normally appears within the wheel tracks and is an indication that the induced traffic loading is starting to cause damage to the pavement. These cracks are often referred to as Alligator or Crocodile cracks;
- Transversal cracks which could be a result of settlement, freeze-thaw or an early form of shrinkage cracking due to cementation of the pavement layers;
- Longitudinal cracks will occur as a result of construction joints (for example, when shoulders are constructed during a later stage of the pavement's life), settlement or active clay in the sub layers; and,
- Block or cementation cracks, observed on highly cemented layers.

This chapter only deals with load associated cracking. This cracking mechanism is one of the most important pavement performance indicators for two reasons:

1. It is one of the pavement design aspects indicating the various stages of pavement decay/deterioration. For example, in mechanistic pavement design, early cracking in cemented pavements indicates the first stage of reduced stiffness/strength pavement behaviour (refer to Figure 5.1). Likewise, intensive cracking on the same pavements indicate when the lightly cemented pavement would start behaving as a normal granular pavement.

2. Engineers will combat pavement cracking as soon as possible, as it exposes the base layer to water ingress, increasing the risk of secondary defects appearing, such as potholes and/or rutting.

Therefore, it is common for most pavement management systems to include cracking as a performance measure and/or trigger point for maintenance intervention. Likewise, in New Zealand, cracking has a prominent role as a practical maintenance decision driver in the field, and in the pavement management system.

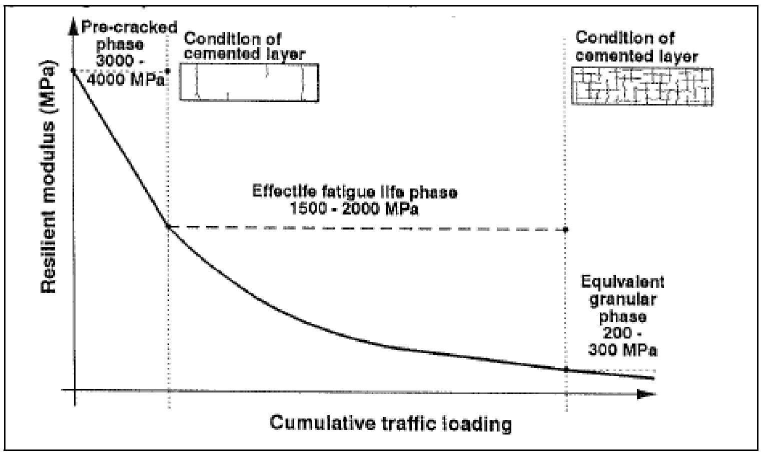
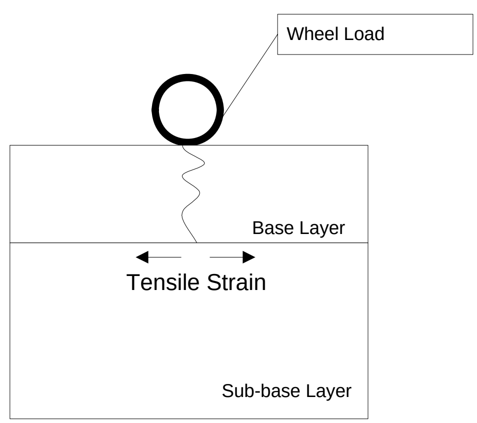


Figure 5.1: Long-Term Behaviour of Lightly Cemented material (Theyse. et al, 1996)

The actual crack mechanism can occur as a result of two possible mechanisms, as illustrated in Figure 5.2. In the modelling of cracking, both mechanisms are viewed together as load associated cracks, since it is very difficult to tell which mechanism will take place for a given pavement.

Cracking due to base layer performance (lightly cemented)

This form of cracking will appear when the traffic loading exceeds the fatigue life of the base layer (tensile strain at the bottom of the base layer)



Cracking due to deformation or densification of sub layers

This form of cracking normally involves shear failure of the base layer due to lack of support from sub layers of the pavement. The deformation of sub layers may result from over-loading or subsoil movement such as active clay deformation.

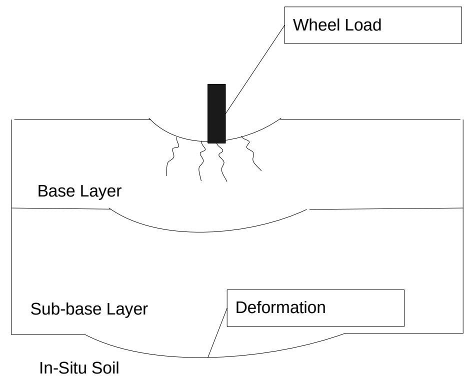


Figure 5.2: Mechanisms of Cracking Due to Traffic Loading

The rest of this chapter describes the research process which resulted in the development of a crack initiation prediction model. This development took place in three stages:

1. The default HDM-4 models were calibrated by adjusting the climatic calibration coefficients;
2. The HDM-4 model format was kept unchanged, but by changing model coefficients, a better fit with the data was attempted; and,
3. The last step involved the development of a new model from basic principles. As part of this process the data was further analysed in order to establish the significant factors that contribute towards the time of crack initiation. This was followed by statistical regression techniques such as Anova analysis, and step-wise regression.

Each of the above stages are discussed in following sections. These sections each start with some theoretical background relevant to the process. Then the data and data requirements are discussed followed by the results. At the end of this chapter, all these methods are discussed in terms of its appropriateness to the New Zealand conditions. The last section summarises the findings and recommends the appropriate implementation process.

5.2 Calibration of the HDM-4 Model

5.2.1 Existing Model Format

According to the HDM definition, crack initiation occurs when a surface displays cracks on more than 0.5% of its area (Watanatada, et al. 1987). The cracked area is calculated by multiplying the length of the crack by the width of affected area (for line cracks the effected width is assumed to be 0.5m). Separate crack initiation model forms were developed in HDM-4 for stabilised and granular bases, and for original surfaces and resurfaced surfaces. The majority of New Zealand roads fall in the granular base category, as most New Zealand pavements are only lightly stabilised in contrast to global practice. The crack initiation for these types of pavements can be predicted from (NDLI, 1995):

Original Surfaces:

$$ICA = K_{cia}\left\{CDS^2 a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\} \qquad \textbf{Equation 5.1}$$

Resurfaced Surfaces

$$ICA = K_{cia}\left\{CDS^2\left[MAX\begin{pmatrix} a_0 \exp\left[a_1 SNP + a_2\left(\dfrac{YE4}{SNP^2}\right)\right]* \\ MAX\left(1 - \dfrac{PCRW}{a_3}, 0\right) a_4 HSNEW \end{pmatrix}\right] + CRT\right\} \qquad \textbf{Equation 5.2}$$

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	areas 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 due to 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, for resurfaced sections, the thickness of the new surface.

5.2.2 Calibration Methodology of the World Bank HDM-4 Models

The main objective of Level 2 calibration is to adjust the calibration coefficients so that the predicted performance compares well with the observed performance. For example, in an area with high rainfall it would be expected that pavements would deteriorate faster than similar pavements in a dry area. In such a scenario, the wet area would have a crack initiation calibration coefficient of less than 1.0, which means that pavements would crack faster than pavements modelled in the original study area.

The HDM-4 proposed method to calibrate crack initiation models is (Bennett and Paterson, 2000):

$$K_{ci} = \frac{\text{mean OTCI}}{\text{mean PTCI}} \qquad \textbf{Equation 5.3}$$

The error function to minimise is given by (Bennett and Paterson, 2000):

$$RMSE = SQRT\left\{ mean\left[(OTCI_j - PTCI_j)^2 \right]_{j=1,n} \right\} \qquad \textbf{Equation 5.4}$$

Where:

RMSE is the Root Mean Square Error

OTCI observed time to crack initiation

PTCI predicted time to crack initiation

The disadvantage of the HDM-4 approach is that it only takes account of sections that have already cracked, thereby ignoring sections which outlast expected performance. This method is therefore biased towards early cracked sections. In order to take account of sections that are un-cracked beyond the point of predicted cracking, Rohde et al. (1998) proposed an alternative method.

According to this method, K_{ci} is determined from 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):

$$Err = \sum w_i \left(TYCR - SAGE_2\right)^2 \qquad \textbf{Equation 5.5}$$

where:

Err is the error function to be minimised over the number of sections

$SAGE_2$ is the seal age at the time when crack initiation took place (first observation of cracking) or the current age if the section is still uncracked;

TYCR is the predicted time to crack initiation

w_i is the weighting factor:

- 0.0 if TYCR > SAGE2 and the pavement is uncracked;
- 1.0 if TYCR < SAGE2 and the pavement is uncracked;
- 1.5 if TYCR < SAGE2 and the pavement is cracked; and
- 1.0 if TYCR > SAGE2 and the pavement is cracked.

The above weightings were subjectively derived and tested by comparing the model prediction outcome with expected and observed values (Rohde et al., 1998). Initial calibration results have suggested that the weightings are appropriate for New Zealand conditions (Henning and Tapper, 1994).

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

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

It is not expected that these differences would result in major changes in the predicted outcome between the two calibration approaches. However, it is expected that the HDM-4 approach would yield a more conservative result. In other words, it will usually predict earlier cracking.

5.2.3 Data Requirements for Calibration

During the initial analysis of this research, the LTPP survey data had only been collected for three years. Therefore, the cracking data was limited and not statistically robust for crack initiation calibration. There was, however, an opportunity to utilise network survey data collected on the same LTPP sections for this purpose. Most of the State Highway LTPP sections are contained within the State Highway benchmark sections which are one km long that undergo repetitive High Speed Data (HSD) surveys annually. In addition, a visual rating is undertaken at these sites according to the RAMM survey methods. More importantly, since comprehensive inventory and cracking data (RAMM ratings) were available on these sections dating back as far as 1999, it was possible to perform the crack calibration using the benchmark section data.

The RAMM rating consists of assessing the length of wheel-path cracked. This length of cracking is subsequently converted to percentage cracking, according to conversion factors documented in HTC (1999):

$$\text{Percentage Cracking} = 0.0004\left(\text{Alligator} \times \frac{50}{\text{insp\_length}}\right)^2 + \left(0.28\ \text{Alligator} \times \frac{50}{\text{insp\_length}}\right) \quad \textbf{Equation 5.6}$$

Where: Percentage Cracking is the percentage of the total lane area cracked

Alligator	the length (m) of the wheel path showing alligator cracking
insp_length	inspection length in (m)

The accuracy of this conversion is not a major concern, since the crack initiation is identified at a point when the cracking exceeds 0.5% (or an equivalent of approximately 2m of cracking on a 50m 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 50m rating section was assessed in successive years.

5.2.4 Calibration Results for Adjusting Climatic Coefficients

The resulting regional calibration factors are presented in Table 5.1. The State Highway LTPP sections are located in four climatic regions as described in Section 3.2.4. According to this method, the climatic regions are classified according to the ratio of rainfall to wet strength properties of the soil. High and moderate risk areas include wetter areas combined with more sensitive soil types (e.g. Northland), whereas low and limited risk areas are the drier areas with more stable soil types such as Canterbury. Statistically, more than 15 sections in each sub-category is required in order to have sufficient data for meaningful results (Van As and Joubert, 1991). Sufficient data were only available to group the results into two climatic regions in order to obtain statistically significant results. Consequently, high and moderate risk areas where combined, as were low and limited risk areas.

Table 5.1 indicates a smaller crack initiation factor (Kci) 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 research. The New Zealand values are consistent with other international calibration results (Rohde et al., 2002). Furthermore, it is to be expected with New Zealand climate and soil types to have a calibration coefficient which is less than 1.

Table 5.1: Summary of Calibration Result for Different Climatic Regions (Transit, 2004)

	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

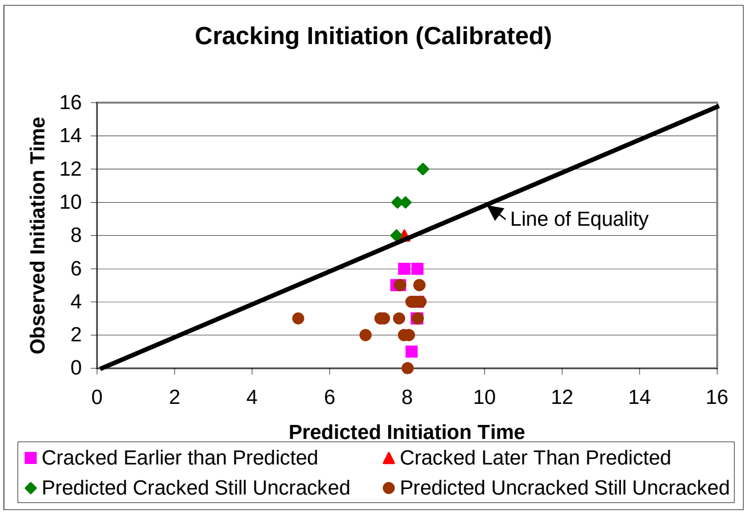
Notes: The values in brackets are the calibration coefficients obtained from LTPP Section data alone.

Figure 5.3 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. Figure 5.3 shows the range of predicted crack initiation is much narrower compared with the range in actual crack initiation time (i.e. 4.5 - 8.2 years versus 0 - 12 years). A similar observation has been reported by Henning (1998). The wider distribution in actual crack initiation time compared to the model can be explained as follows:

- There will always be a wider range in observed over predicted values due to the natural variation of the data and influences from external factors which are not incorporated into the model;

- The model calibration outcome as presented in this section only involved the adjustment of the climatic calibration coefficient. A closer fit between the actual and the predicted crack initiation can be obtained by adjusting the model coefficients; and

- It is also possible that different crack mechanisms exist, and by aggregating them into one single analysis, a poorer fit between the actual and predicted observations is obtained. For example, in Figure 5.3 all sections are included regardless of whether it has cracked before resurfacing or not.



Notes: Data included for all Benchmark Sections that correspond with the LTPP sections (i.e. 40 sections across the country)

Figure 5.3: Comparing Actual Cracking with Predicted Cracking (Transit, 2004)

5.3 Adjustment of HDM-4 Default Model Coefficients

5.3.1 The Default Model Coefficients

The default model coefficients were included in the HDM-4 models in order to have one model format applied to various pavement and surface types. For example, consider the crack initiation expression for new surfaced pavements indicated below (NDLI, 1995):

$$ICA = K_{cia}\left\{CDS^2 a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\}$$ **Equation 5.7**

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

In this expression above there are three default model coefficients a_0, a_1 and a_2. The default values of these coefficients for different pavement and surface type combinations are listed in Table 5.6.

Table 5.2: Default Coefficients forHDM-4 Cracking Models (NDLI, 1995)

Pavement Type	Surface Material	a_0	a_1	a_2	a_3	a_4
Asphalt Mix on Granular Base	All	4.21	0.14	-17.1		
	All except concrete	4.21	0.14	-17.1	30	0.025
	concrete	13.2	0	-20.7	20	1.4
Asphalt Mix on Asphalt Base	All	4.21	0.14	-17.1		
		4.21	0.14	-17.1	30	0.025
Asphalt Mix on Asphalt Pavement	All	4.21	0.14	-17.1	30	0.025
Asphalt Mix on Stabilised Base	All	1.12	0.035	0.371	-0.418	-2.87
		1.12	0.035	0.371	-0.418	-2.87
Surface Treatment on Granular Base	All	13.2	0	-20.7		
	All except single layer, CAPE	13.2	0	-20.7	20	0.22
	single layer, CAPE	13.2	0	-20.7	20	1.4
Surface Treatment on Asphalt Base	All	13.2	0	-20.7		
	All except single layer, CAPE	4.21	0.14	-17.1	20	0.12
	single layer, CAPE	4.21	0.14	-17.1	30	0.025
Surface Treatment on Asphalt Pavement	All	4.21	0.14	-17.1	20	0.12
Surface Treatment on Stabilised Base	All	1.12	0.035	0.371	-0.418	-2.87

Therefore, by changing the model coefficient, each pavement and surface type combination will have a unique model for predicting the crack initiation, while the overall model format remains unchanged. For example, the pavement strength (SNP) is a significant factor in Asphalt type pavements (a_1 = 0.14), while it is not significant in granular pavements (a_1 = 0).

5.3.2 Methodology for Adjusting the Model Coefficients

The process of establishing model coefficients for a given model format is relatively simple and most statistical software are capable of performing this type of analysis. Given that statistical software perform this process without any regard for the validity of the result, this type of analysis should be complemented with a thorough exploration of statistical graphs in order to interpret the validity of the statistical outcome. The full process of determining the appropriate model coefficients is as follows:

- Prepare a dataset that includes all the independent variables plus the dependent variable (time to crack initiation in this instance);
- Perform the exploratory statistics in order to determine the significance of the variables included in the default model;
- Undertake statistical linear regression analysis by entering the existing model format and allowing the statistical software to determine the model coefficients. A model estimation process using a maximum likelihood statistical estimation (MLE) approach can be used in instances where there is a concern about the bias of the dataset (Bennett and Paterson, 2000). For example, under normal circumstances, it is difficult to take account of pavements which have not cracked yet. In order to determine the average crack initiation on a network, this limitation can be partly overcome by using the MLE approach; and,
- Validate the model outcome in order to ensure that the statistical analysis did not yield any impractical results.

5.3.3 Data Requirements for Model Adjustment

Given the limited LTPP and benchmark section cracking data, network RAMM survey data were considered and found to be appropriate for the analysis purposes. Data from two regions (East Wanganui and Coastal Otago) were selected for the crack analysis. These two regions are classified as medium and low climatic sensitivity areas respectively (refer to Henning et al. 2004b). Furthermore, the data availability and researcher's 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 50m rating sections had not changed over time;
- Each section had a minimum of four rating years; and
- All historical performance data were extracted in order to compare the performance of surfaces prior and after resurfacing.

Only chip seal pavements were analysed. The distribution of time to crack initiation from the cracking data for the respective regions is presented in Figure 5.4.

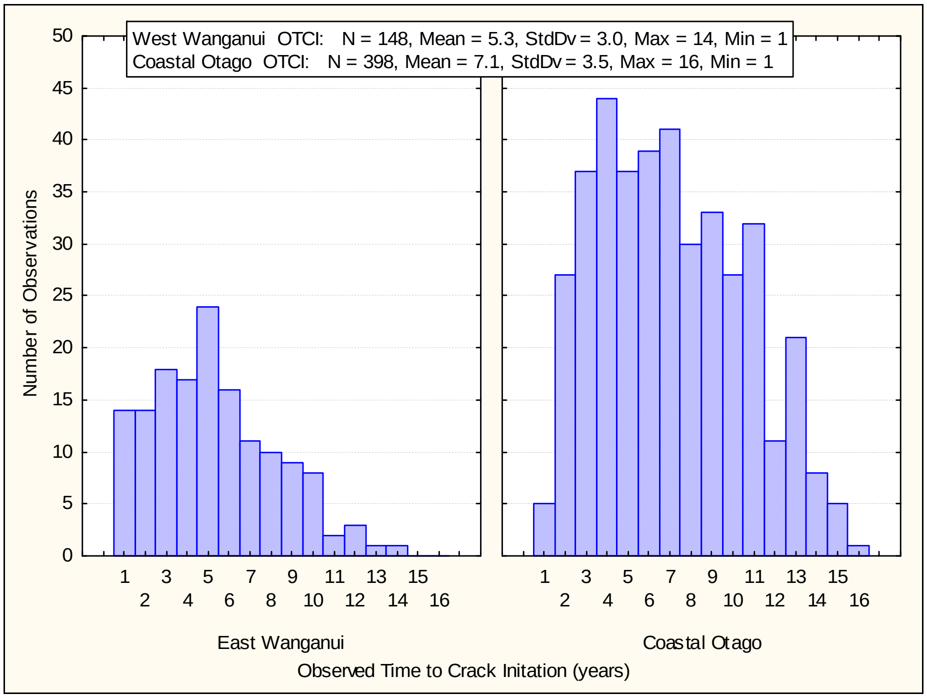


Figure 5.4: Distribution of Crack Initiation for the Two Regions

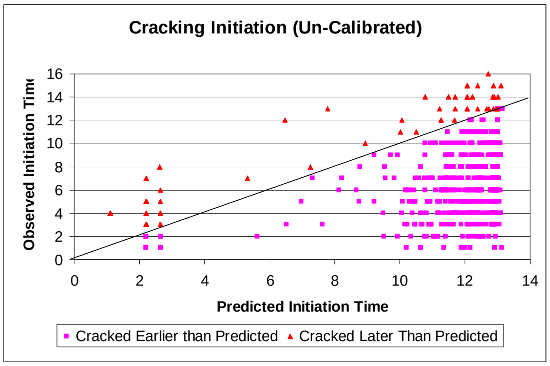
As expected, the average time to crack initiation was longer on the Coastal Otago region (7 years vs. 5 years), thus confirming the appropriateness of the climatic classification. Coastal Otago consists of more stable soils and climatic conditions (i.e. lower risk area according to Section 3.2.4). It 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.

5.3.4 Resulting Model Coefficients

Based on the methodology described in Section 5.3.2, the model coefficients were adjusted using a goal-seek regression method. The results from the analysis are summarised in Figure 5.5.

Default HDM Crack Initiation Model

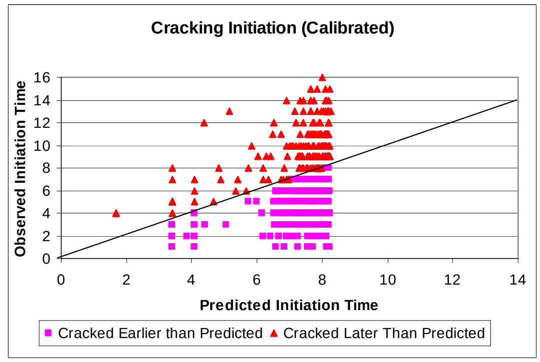
a0	a1	a2	a3	a4
13.2	0	20.7	20	0.22



Error = 17597.7

HDM Crack Initiation Model with Adjusted Coefficients

a0	a1	a2	a3	a4
8.3	0	18.54	0.01	0.34



Error = 6697.9

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

Figure 5.5: Resulting Model Coefficients for Existing HDM Model Format

Observations from Figure 5.5 include:

- using the new model coefficients has reduced the error by more than half from 17,597 to 6,697;
- un-calibrated, the predicted crack initiation varied from 1 to 13 years compared to the actual 1 to 16 years. The corresponding range for the predicted cracking initiation using the adjusted model coefficients ranged between 1 to just over 8 years. Clearly, the current model and/or calibration process has a limitation 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 majority of the predicted crack initiation periods are between 10 and 13 years and 6 and 8 years, for the default and adjusted model respectively. In contrast the actual crack occurrences are more evenly distributed; and,
- All the model coefficients have changed with the a_3 coefficient showing the most significant change from the default to the adjusted model.

Changes to the model coefficients suggest that the influence of loading/strength relationship ($YE4/SNP^2$) on crack initiation 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 significantly reduced (a_3). The coefficient a_3 has changed from a value of 20 to 0.01. For the default model, the ratio between the previous cracking and the coefficient was a continuous variable, whereas the calibrated model suggests that it is a binary variable (either 0 or 1). This suggests that crack initiation is a function of whether the old surface was cracked or not. The actual value of the previous cracking is not significant. This phenomenon has been confirmed in both the exploratory and the regression analysis as described in Section 5.4.

Although the calibrated model has a significantly better fit to the actual crack initiation (lower error), it is observed that the scatter between predicted and observed crack initiation time is still significant. Based on the results shown in Figure 5.5, it can be concluded that the model has little 'predictive power'. In addition, the tail-end of the prediction (maximum values) does not correspond with reality. This outcome can suggest that the model format is wrong and/or that some transformation is required. For example, one of the variables should have been log transformed or exponential. The model form and transformation of variables are further discussed in the following section.

5.4 Development of an Alternative Crack Initiation Model

5.4.1 Applicability of Data Driven Models

The work completed in the previous sections was aimed at adjusting the existing HDM crack initiation models. The results have suggested that a new model form should be considered. However, it should be highlighted that any deviation from the HDM modelling approach to a more data driven type model would result in a less transferable model. Therefore, the models discussed in the following sections can only be applied for the areas/networks on which they are developed.

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

- Exploratory statistics aimed at understanding the relationships between the possible variables and the predicted crack initiation better. During this stage it is also important to search for any inter-variable relationships. General trends and possible relationship forms are noted during this phase, since it could simplify the regression analysis which follows later;
- Multivariate analysis is then undertaken to determine the significant variables that influence the independent variable; and
- Lastly, the regression is undertaken to define in which format the variables are combined, in order to predict the outcome of the independent variables.

5.4.2 Exploratory Statistics

5.4.2.1 Investigated Variables

The variables considered during the analysis in the following sections are included in Table 5.3:

Table 5.3: Variables Considered for Predicting Crack Initiation

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
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)	Binary
CS_PCA	cracked status prior to resurfacing (0 or 1 for not cracked or cracked)	Binary
HTOT	total surface thickness (in mm) of all the layers	Continuous
HNEW	surface thickness (in mm) of the latest surfaced layer	Continuous

The binary factors were adopted since there are two different crack stages – the first generation where cracking occurs on newly constructed pavements, and the second generation where cracks occur on a resurfaced section which has been cracked before. The latter mechanism is also referred to as reflective cracking. The first objective of the analysis is to establish whether the actual data supports these two crack stages.

5.4.2.2 Influence of Condition of Surface prior to Resurfacing on the Crack Initiation Time

Figure 5.6 presents the distribution of crack initiation for first/second generation surfaces (left plot) and for different cracked status prior to resurfacing (right plot). This clearly illustrates the distinct difference in crack initiation time for new surfaces and resurfaced seals. It was therefore expected that this variable would have a significant influence on the final model, and has therefore been included in the multi-variable analysis.

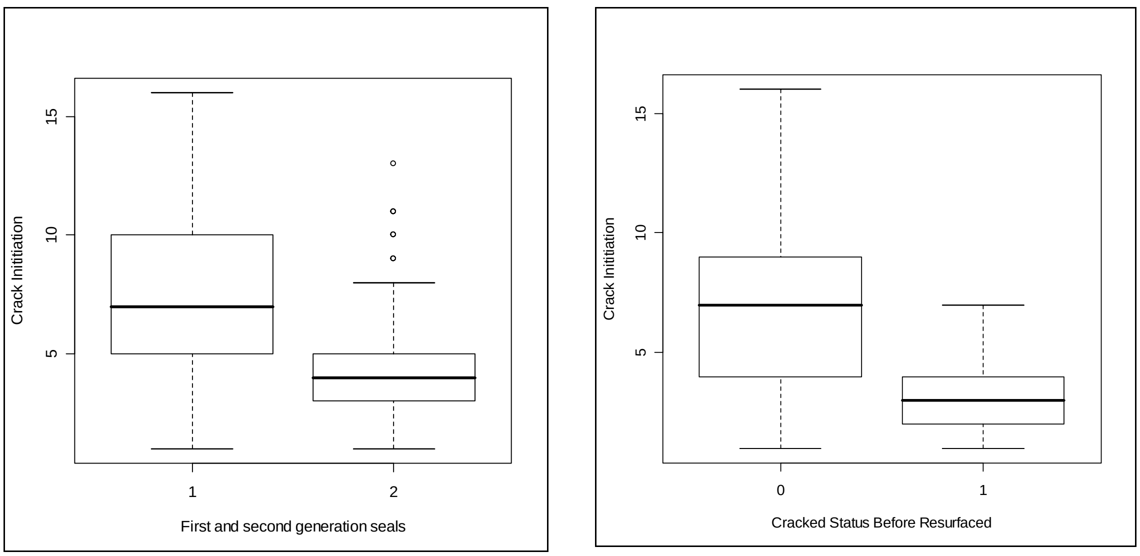


Figure 5.6: Crack Initiation for Different Resurfacing Cycles and Status Prior to Resurfacing

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 seal;
	Cracked Status (0) –	new surfaces or resurfaced sections that have not been cracked before (note that in NZ skid resistance or texture depth may be a driver for resurfacing)
	Cracked Status (1) –	resurfaced sections where the previous surfaced was cracked prior to resurfacing.

The relationship between the crack initiation and the percentage cracking prior to resurfacing was further investigated, and no conclusive relationship was established.

This result is consistent with the regression findings in Section 5.3.4. which resulted in the model coefficient for previous cracking in a binary form rather than a continuous variable. This means it is not important how much the surface has cracked before resurfacing. Therefore, it can be safely concluded that whether or not a section was cracked prior to resurfacing, will have a significant influence on the crack initiation period. However, the actual crack percentage prior to resurfacing is not significant.

It remains questionable whether both the parameters should be included into the model. If the aim of this process is to keep the final model as simple as possible as it may result in only one of these two variables being used.

5.4.2.3 Thickness of the New Surface and the Total Surface Thickness

Figure 5.7 shows the relationship between surface thickness and crack initiation. Two thicknesses were considered, firstly the new surface thickness and secondly, the total surface thickness.

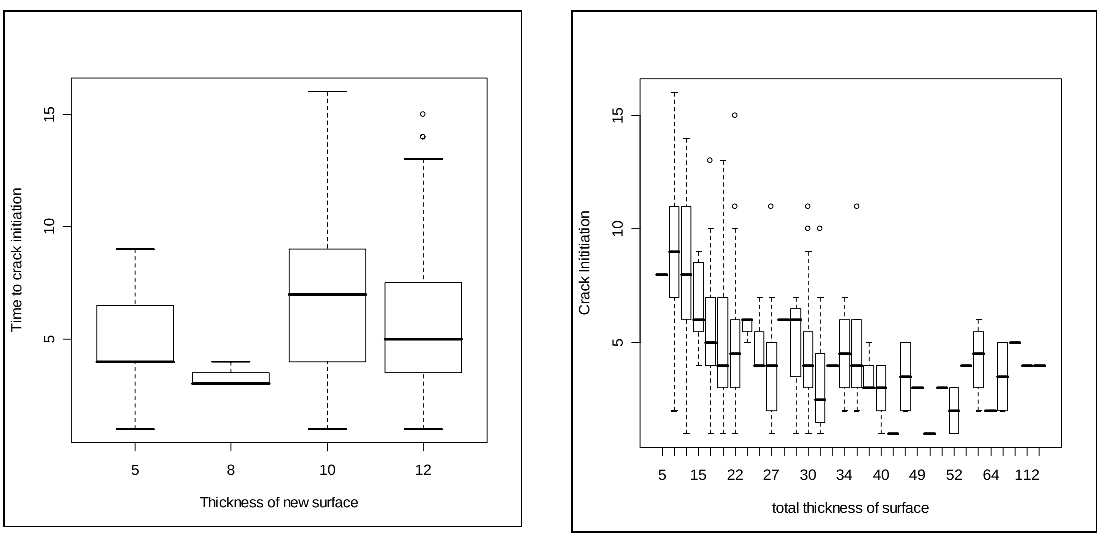


Figure 5.7: Relationship between Thickness of New Surface and Total Surface Thickness with the Crack Initiation Period

There is no apparent relationship between the new surface thickness and the crack initiation time (Figure 5.7 left hand plot). It should be noted however, that the new surface thickness was derived from an assumed thickness given the surface code in RAMM. From experience, this data is known not to be 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 5.7 (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 prior to the last resurfacing. Cracking observed on these sections are therefore reflective cracking of third, fourth or even later generation seals; and
- It is well known that multiple surface layers are more unstable (HTC, 1999). It can therefore be assumed that there is significant movement and flexing of the surface layers, thus resulting in more strains and subsequent cracking of the newly surfaced layer.

5.4.2.4 Pavement Strength (SNP)

Figure 5.8 illustrates the crack initiation as a function of structural number for different surface thicknesses and cracked status. No significant relationship is shown in any of the graphs depicted.

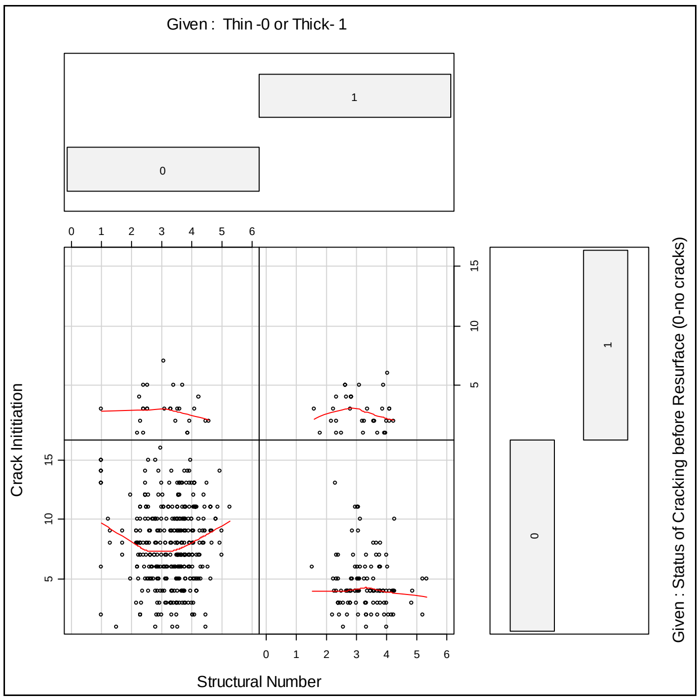


Figure 5.8: Crack Initiation as a Function of Structural Number for Different Combinations of Surface Thickness and Cracked Status

The lack of any relationship between SNP and the crack initiation is not completely unexpected, because it does not consider the traffic effects. Most pavements are constructed according to the expected traffic loading. For example, it is expected that a stronger pavement would 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 further discussed in Section 5.4.2.6.

5.4.2.5 Traffic Loading

Figure 5.9 illustrates a possible relationship between the log of average annual daily traffic (AADT), and the crack initiation. It appears that there is a consistent relationship between traffic loading and cracking, regardless of cracked status prior to resurfacing. The log format was used for the traffic loading 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.

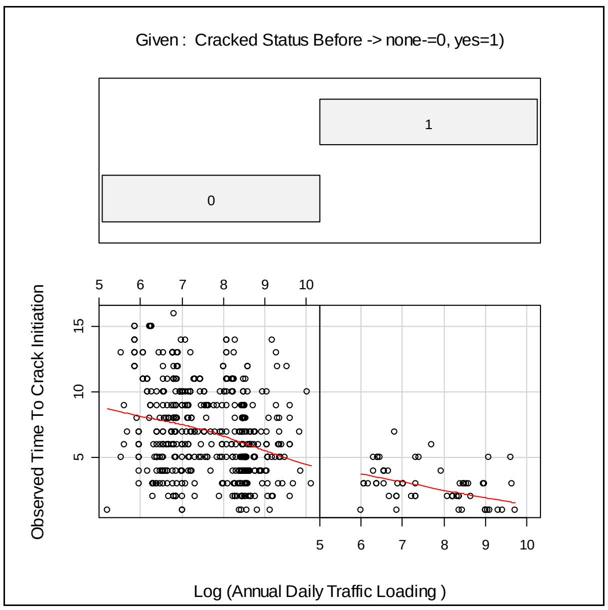


Figure 5.9: Observed Crack Initiation Period as a Function of Traffic Loading and Annual Daily Traffic (AADT)

5.4.2.6 Traffic Loading and Structural Number Relationship

The previous sections investigated the relationship between the traffic and SNP separately with the time to crack initiation. These suggest that 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 crack initiation. Of specific interest was whether the (YE4/SNP^2) relationship differs for new surfaces and previously cracked surfaces. Note that (YE4/SNP^2) was investigated since this is one of the factors in the HDM-4 model (discussed in Section 5.2.1). This relationship is illustrated in Figure 5.10. There appears to be a strong relationship between the YE4/SNP^2 and the crack initiation. It further appears that this relationship could be of an exponential form.

Figure 5.10 further shows that the relationship has a similar format regardless of whether sections have been un-cracked or cracked prior to resurfacing. It does appear though, that the relationship is more distinct for previously un-cracked sections.

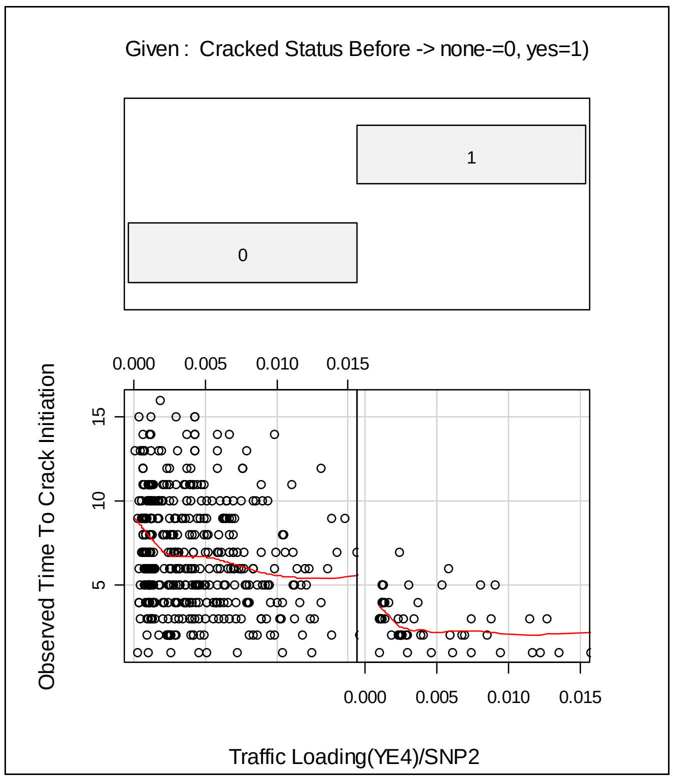


Figure 5.10: Crack Initiation as a Function of Traffic Loading and SNP

5.4.3 Correlation Analysis

The previous sections considered variables in isolation. The next step considers the inter effects of the variables. It is important not only to get an understanding of how the factors relate to crack initiation, but also to clarify how these factors relate to each other. One of the aspects considered in this approach is to test for co-linearity. Co-linearity is when two independent variables are both significant factors in a model, but they are dependent on each other.

As a first step, all the factors considered for the model were plotted against each other, and these are presented in Figure 5.11. Although the apparent relationships did not reveal any unexpected trends, it is still worth mentioning some issues to address.

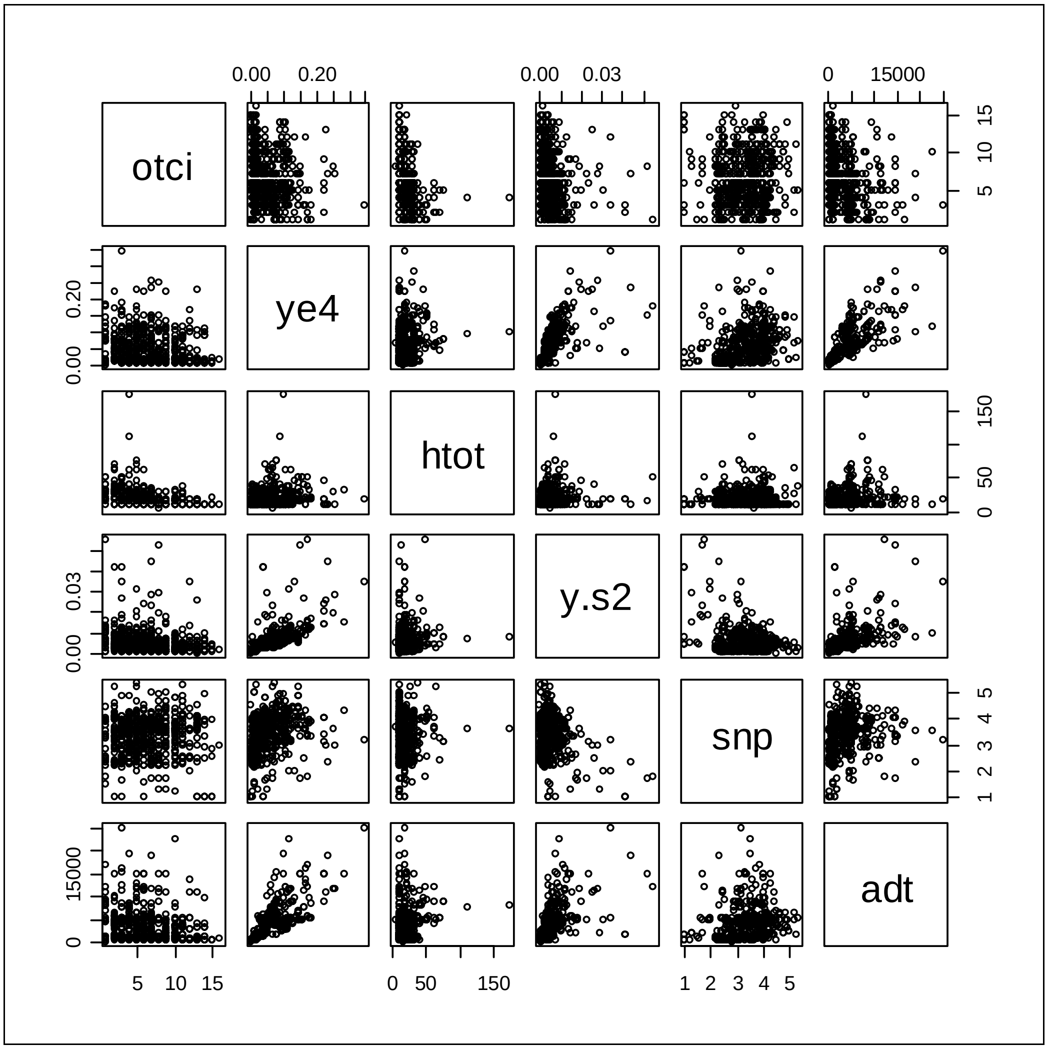


Figure 5.11: Inter-Relationships of Crack Initiation Variables

The traffic Loading (YE4) is derived from taking certain percentage heavies of the annual average daily traffic (AADT). It is, therefore, expected that these two factors are 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, it may be that YE4 should be used for the first occurrence of cracking, and AADT for the reflective cracking.

Obviously, there should also be a relationship between YE4 and SNP as individual factors compared with the $YE4/SNP^2$ variable. It should be further investigated what the optimal relationship of these factors are as a single predictor.

5.4.4 Linear Regression Model

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 improves the model. With the backward step method, the process starts with all the possible variables included to 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:

$$ICA = f\begin{pmatrix} Surf\_Gen + CS\_PCA + HTOT + \\ AADT + (HTOT : AADT) \end{pmatrix} \qquad \textbf{Equation 5.8}$$

Where	ICA	is the crack initiation time in years after the surface is constructed
	Surf_Gen	is the 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)
	CS_PCA	is the cracked status prior to resurfacing (0 or 1 for not cracked or cracked)
	HTOT	is the total surface thickness (in mm) of all the layers

AADT is the annual average daily traffic

The model coefficients and respective model statistics are presented in Table 5.4. The model outcome has confirmed the observations made in the previous sections regarding the significance of the model variables including:

- The status of the overall surface is the prominent predictor of cracking, this includes how thick the total surface is and whether it has cracked prior to resurfacing or not;
- The only other significant variable is the traffic loading;
- The pavement strength (SNP) and traffic loading (YE4) were not significant factors. Although earlier results contradicted this finding (as discussed in Section 5.4.2.6), it should be realised that the contribution of these factors are valued relative to all the other factors in this regression; and
- There is also an inter-relationship between the HTOT and AADT, which suggests that the influence of total surface thickness differs for various traffic ranges. This trend was also confirmed with the base data plots (shown in Figure 5.12).

Table 5.4: Results of Regression Analysis for Predicted Crack Initiation

	Estimate	**Std. Error**	**t value**	**Pr(>|t|)**	**Significance[1]**
Intercept	$1.088e^{1}$	$4.051e^{-1}$	26.845	$< 2e^{-16}$	***
Surf_Gen	$-7.659e\text{-}^{1}$	$3.783e^{-1}$	-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

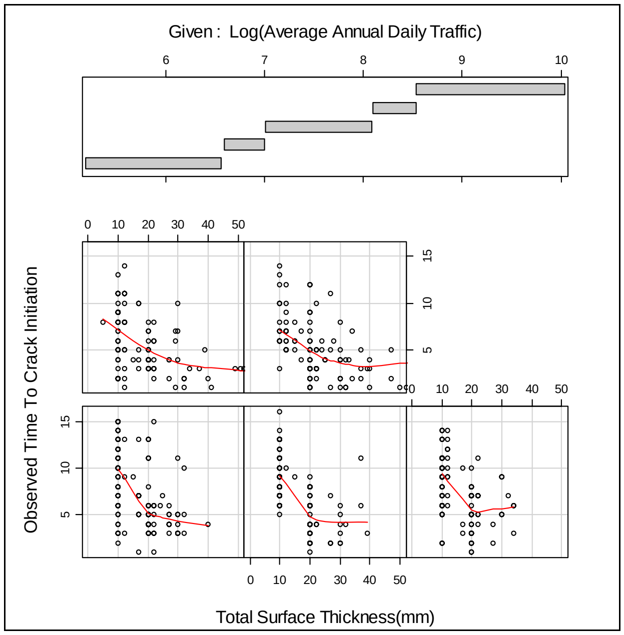


Figure 5.12: Inter-relationship between Total Surface Thickness and Log Traffic (AADT)

Figure 5.12 illustrates the crack initiation for various traffic classes and total surface thickness. A possible logarithmic trend was observed for all the plots. This trend appears to be more sensitive for the lower traffic classes (bottom three plots). However, a general faster crack initiation was reported on the higher volume classes (top two plots).

The model was further improved by considering other model formats. For example, by transforming the observed crack initiation to a logarithmic scale slightly raised the correlation. A simplified and recommended form of the model can be given by:

For PCA = 0 (sections not cracked prior to resurface)

$$\text{ICA} = \text{Kci} * \exp\begin{bmatrix} 5.7 - 1.25\log(\text{HTOT}) - \\ 0.3\log(\text{AADT}) + 0.08\log(\text{HTOT}) * \log(\text{AADT}) \end{bmatrix}$$ **Equation 5.9**

For PCA > 0 (sections were cracked prior to resurface)

$$\mathrm{ICA} = \mathrm{Kci} * \exp\begin{bmatrix} 4.6 - 0.68\log(\mathrm{HTOT}) - 0.47\log(\mathrm{AADT}) + \\ 0.08 * \log(\mathrm{HTOT}) * \log(\mathrm{AADT}) \end{bmatrix}$$ **Equation 5.10**

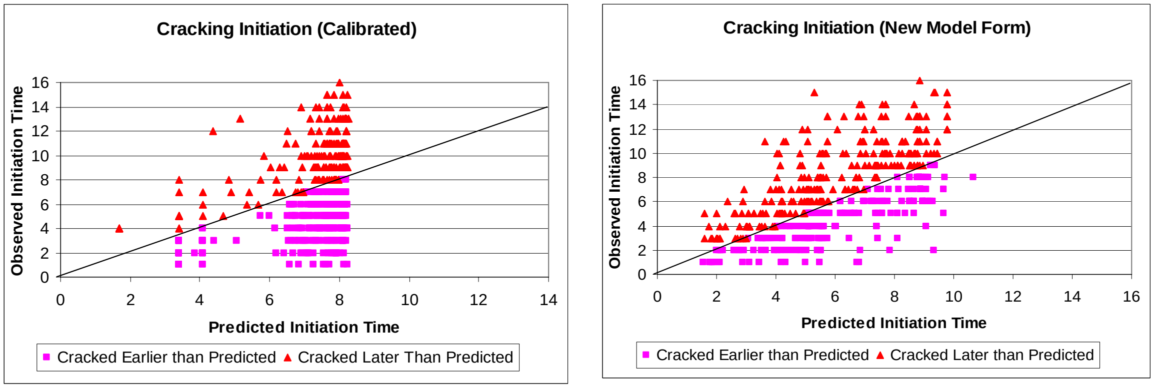
Where ICA is the crack initiation time in years after the surface is constructed

K_{ci} is the crack initiation calibration coefficient

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 5.13. This compares the resulting predicted crack initiation for the adjusted HDM-4 model (left plot – refer to Section 5.2.4.), and the new model format (right plot), resulting from the linear regression and both expressions indicated above.



Error = 6,697.9

Error = 5,157

Figure 5.13: Comparing Predicted Versus Actual Crack Initiation for New Model Format

Comparing the results in Figure 5.13 illustrates the overall better fit of the new model format. It is also noted 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-4 model. The error of the new model (5,157) format is lower than the adjusted HDM model and improved the accuracy by a factor of three, compared to the default HDM-4 model (17,597).

However, there is still a large scatter observed between the predicted and actual values. It is observed that the best correlation coefficient obtained from the data are $R^2 = 0.45$ 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. It can therefore be assumed that there are many other factors that influence the model outcome that are not included in the expression. For most of these factors there is no data. Some of these 'missing variables' are likely to include:

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

Generally, it is possible to improve the robustness of the model by including some of these factors such as construction quality to the model, as discussed in Paterson et al (1997) and Henning (1998). The difficulty with these factors is that this information is rarely available for networks, and it therefore does not make the model more applicable to the network under consideration.

Secondly, no model will ever be able to predict pavement behaviour 100% accurately, because there will always be some random effects that may influence behaviour outside the scope of the prediction model. For this reason, R^2 of less than 0.5 are common in pavement performance prediction.

The question is whether or not such a low predictive power is really acceptable within the pavement management system.

5.4.5 Generalised Linear Model (GLM)

An alternative method would be to consider the actual statistical distribution of cracking initiation. Therefore, by presenting the prediction model in a different way, it incorporates 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. Also, instead of predicting an actual initiation time it considers the full life of the surface with an associated probability for cracking in every year. This probability to crack not only considers the surface age but all other factors that significantly influence the crack behaviour.

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, un-cracked surfaces will remain cracked status = 'false' at the given surface age. Similar to the previous section, a stepwise regression was performed. The resulting model from this analysis was (refer to Table 5.5 for the variable significance):

$$\mathrm{STAT.ACA} = f\begin{pmatrix} \mathrm{AGE2} + \mathrm{FACTOR(stat.PCA)} + \\ \mathrm{Log(AADT)} + \mathrm{Log(HTOT)} + \mathrm{SNP} \end{pmatrix} \qquad \textbf{Equation 5.11}$$

Where STAT.ACA Cracked status within a given year;

AGE2	is the surface age in years, since construction
stat.PCA	is the cracked status prior to 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

Table 5.5: Results of Regression Analysis for Predicted Crack Initiation

	Estimate	Std. Error	t value	Pr(>|t|)	Significance[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: Signif. 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 (Akaike's Information Criterion (AIC) is like a fault term, with the lower values indicating better fit with the observed data. The best model (i.e. most significant variables) is determined by finding the best combination of variables in order to minimise the AIC)

Again, the results obtained from the GLM analysis have been consistent with the data observations presented in earlier sections. For example, Table 5.5 shows the significant factors being the cracked status prior to resurfacing, traffic (AADT), total surface thickness and structural number. Both the traffic and total surface thickness are included in the model in a logarithmic format. The only factor that appears to differ with the exploratory plots presented earlier is the structural number. However, during the stepwise regression it was inconclusive whether the SNP was significant or not. Given the value the SNP as a factor can contribute towards a more robust model, it was decided to include it into the final model. For example, it will cater for cases such as weak pavements being overloaded with traffic.

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

$$p = 1 / \left[1 + \exp\left(-a - Bx\right)\right]$$ **Equation 5.12**

Where p is the probability that a specific event occurs, (p(Y=1)

a is the coefficient for the constant term (a would be the intercept which was not included in this model),

B is the coefficient on the independent variables

x is the independent variable (s)

Therefore, the recommended crack initiation model is:

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

Equation 5.13

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 prior to 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 5.14 illustrates an example of the output from this Logit model. It shows two probability plots of cracked status for sections being cracked or un-cracked prior to resurfacing. It suggests that for the given data one can expect sections to crack between 3 to 15 years, depending on the crack status prior to resurfacing.

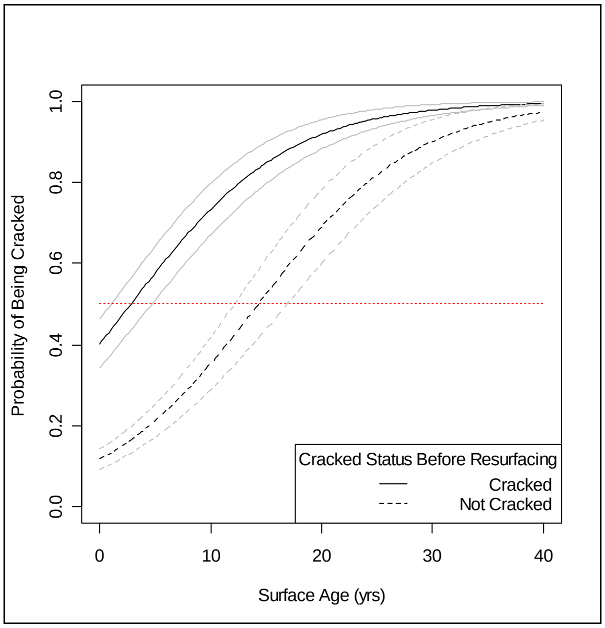


Figure 5.14: Output from the Logit Model - Probability of Cracking for a Given Year

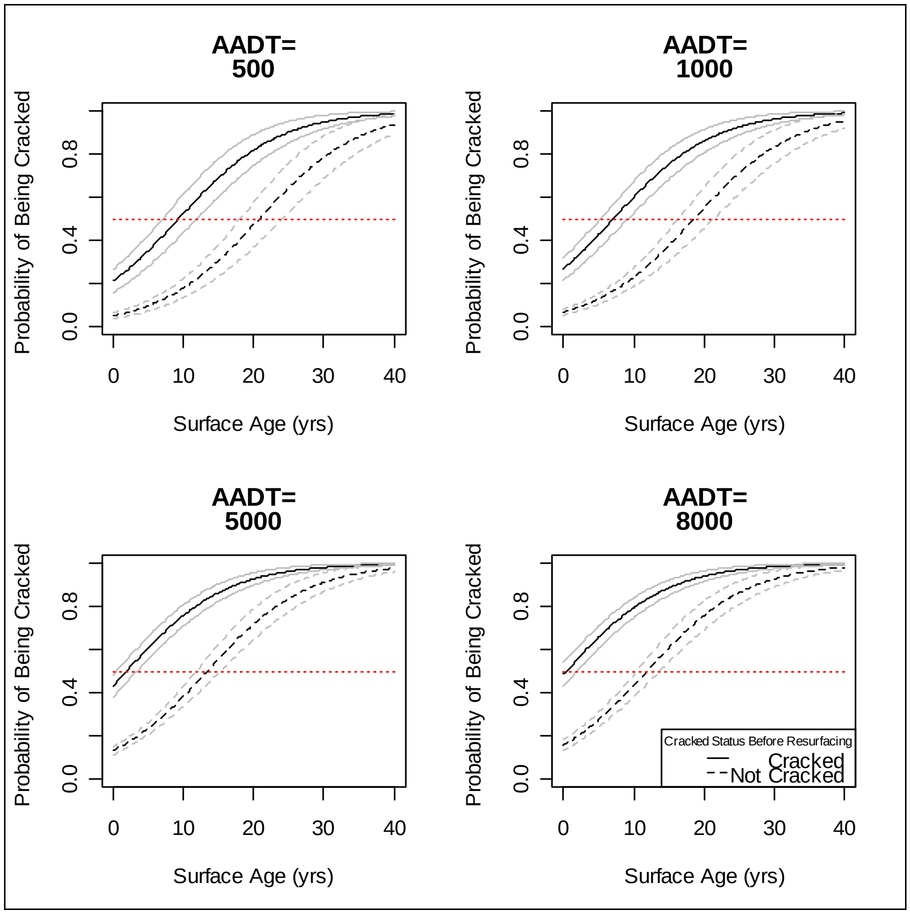
Note: Data plotted for AADT = 2500, HTOT = 60mm, SNP = 2.5

Confidence interval plotted for two standard deviation

Expected crack initiation where p = 0.5

Further outputs from the Logit model are presented in Note: SNP = 2.5, Total Surface Thickness = 30mm

Figure 5.15. With these outputs, the sensitivity of the model was tested for varying traffic levels.



Note: SNP = 2.5, Total Surface Thickness = 30mm

Figure 5.15:Probability of Crack Initiation Times for Different Traffic Levels

It is observed that the probability of cracking changes significantly for varying levels of traffic. For example, at the given structural number, cracked status and surface thickness a section would have a 50% probability of cracking at 9, 7, 2 and 1 years for 500, 1000, 5000 and 8000 vehicles per day respectively. Intuitively this range of crack initiation fits well with experience where low volume roads would starts showing cracks at 9-13 years (refer to Figure 5.13) while high volume roads with such a low structural number will show very early crack initiation, especially if it has cracked prior to resurfacing. This example also shows that:

- The model format is more flexible in terms of predicting extreme cases than the HDM-4 model. Figure 5.13 has demonstrated that the earlier model formats were unable to match the extreme long crack initiation time of some pavements; and,

- The probabilistic format of the model further provides much flexibility in the application of the model. For example, at low traffic volumes maintenance intervention may be scheduled at a higher risk profile, while lower risk profiles may be more appropriate at higher traffic volumes. For this example, maintenance intervention at low traffic volumes may be planned at say 60% probability while the corresponding level would be say 40% on higher traffic volumes.

5.5 Discussion

Table 5.6 summarises the results from the modelling process for the different calibration methodologies covered in this chapter.

Table 5.6: 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 colspan="2">
<table>
<tr><td></td><td colspan="2">Regional Classification</td></tr>
<tr><td></td><td>High and Moderate</td><td>Low and Limited</td></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></tr>
<tr><td>Level 3 – Adjusting Model Coefficients</td><td>Default HDM-4 Model Coefficients
<table>
<tr><td>a0</td><td>a1</td><td>a2</td><td>a3</td><td>a4</td></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><td>a0</td><td>a1</td><td>a2</td><td>a3</td><td>a4</td></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</td><td>For PCA = 0 (sections not cracked prior to resurface)</td><td></td></tr>
</table>

Calibration Level/Method	**Results**	**Error/ Accuracy**
Model Format (Linear Model)	$ICA = Kci * \exp\begin{bmatrix} 5.7 - 1.25\log(HTOT) - 0.3\log(AADT) + \\ 0.08\log(HTOT) * \log(AADT) \end{bmatrix}$ For PCA > 0 (sections were cracked prior to 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	5157
Level 3 – Logit Model	$p(stat.aca) = 1 \Big/ \left[1 + \exp\begin{pmatrix} -0.141AGE2 + \{(5.062, 3.440)\ \mathit{for}\ stat.pca = (0,1)\} \\ -0.455Log(AADT) - 0.275Log(HTOT) + 0.655SNP \end{pmatrix} \right]$ Where p(stat.aca) is the probability of a section being cracked AGE2 is the surface age in years, since construction stat.PCAis the cracked status prior to 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	

It has been demonstrated that the detailed analysis 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 versus actual behaviour.

In particular, it has been demonstrated that the Logit model provides the most promising results. Various factors contribute towards this model being recommended for adoption in New Zealand including:

- More explanatory variables are included in 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 is available (e.g. oxidation of bitumen). The surface age of crack initiation is also an independent variable for all the other model formats;

- The model format is relatively simple with most factors included to the model in an additive method;
- With the logistic model, all data on the network is considered as a basis for the analysis and as a result it takes account of both under-performing and over-performing pavements. Despite all the best intentions, this is not achieved with the HDM type model which does not take full account of un-cracked/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 more flexibility in implementing the model into a pavement management system. For example, triggers can be set according to varying risk/criticality considerations. Also, this same flexibility could also be considered in the interaction with other models such as rutting and roughness. For example, engineers 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%; and,
- The model responded well to 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 NZ system compared to the current approach.

Despite the advantages mentioned, it is accepted that the current model is depends on the data it was derived from. For example, it is noticed that it contains the average annual daily traffic (AADT) instead of the expected traffic loading (YE4). However, from experience more confidence exists in the AADT data compared to the traffic loading[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 strong data driven model, it must be tested for more networks before it is adopted into a national modelling system.

5.6 Crack Initiation Summary

This chapter presented the results from a calibration process that was aimed at yielding the most appropriate model to predict load associated crack initiation on chip seal pavements. This cracking mechanism is one of the most important pavement performance indicators for two reasons:

1. It is one of the pavement design aspects indicating the various stages of pavement decay/deterioration. For example, in mechanistic pavement design, early cracking in cemented pavements indicates the first stage of reduced stiffness/strength pavement behaviour. Likewise, intensive cracking on the same pavements indicates when the lightly cemented pavement would start behaving as a normal granular pavement.
2. Engineers combat pavement cracking, as it exposes the base layer to water ingress, increasing the risk of secondary defects appearing, such as potholes and/or rutting.

Most pavement management systems include cracking as a performance measure and/or trigger point for maintenance intervention. Likewise, in New Zealand, load associated cracking has a prominent role as a practical maintenance decision driver in the field, and in the pavement management system.

[4] Data reviews in NZ 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) do not always reflect reality

For this model development a range of methodologies were tested including:

- An HDM Level 2 calibration resulting in environmental calibration coefficients;
- Model review of the existing HDM-4 model format resulting in new proposed model coefficients;
- New proposed model format that included the development of a simplified linear model; and,
- Newly proposed concept in predicting the cracked status using the Logit model.

The results have indicated that a probabilistic model has achieved the best correlation between the predicted and the actual crack initiation time. The reason for this is because the model incorporated more explanatory variables and it further recognises the randomness of defects appearing on roads. This model format does not predict a definite timing of cracking. Instead a probability of cracking for a given year is predicted for the full life cycle of the surface. This model format not only provides a better fit with the actual behaviour but it also provides a number of benefits for the application of the model including:

- It allows for more flexibility in implementing the model into a pavement management system. For example, triggers can be set according to varying risk/criticality considerations;
- This same flexibility could also be considered in the interaction with other models such as rutting and roughness. For example, crack sealing may be considered 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%; and,
- 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 NZ system compared to the current approach.

With the benefits, there are also some limitations with the Logit model. Most significant is that is not as transferable from one road network to another. This will require the model to be tested in all the areas where it is applied. It is therefore recommended that the cracking model is tested on all the four climatic areas identified in Chapter 3.

CHAPTER 6 PREDICTING RUT PROGRESSION

6.1 Introduction

Rutting is one of the most useful and widely used performance indicators on flexible road pavements. The most prominent of these are listed below:-

- It is used as a performance measure in defining the design criteria of pavements in mechanistic design procedures. For example, according to the AUSTROADS (1992) flexible pavement design method, critical vertical compressive strain on the subgrade is defined for an assumed failure mechanism of a 15mm rut depth.
- Rutting is an important performance measure for pavement management systems (PMS). Most international PMS use rutting as an intervention criteria to trigger maintenance work (RIMS, 2007 and Robertson et al., 1998). As a result, rutting is often reported as a performance measure for reporting overall network condition trends (Transit, 2005).
- From a safety perspective, rutting is one of the measures closely monitored by authorities (Transit, 2005). With an increase of rutting depth, there is an increased risk of water ponding on the road surface, which may lead to hydroplaning. Lay (1998) recommends that rut depths should not exceed 11 to 14mm depending on the road class, geometrical aspects and surface type.

As noted above, rutting is used in many facets of pavement design and management in New Zealand. In addition to this, experience in New Zealand has suggested a relatively poor correlation with the existing World Bank HDM-4 rutting models. For this reason, the testing and development of a rut prediction model is one of the priorities of the New Zealand Long-Term Pavement Performance (LTPP) programmes.

Excessive rut progression is a visual manifestation of pavement failure due to various causes. For flexible unbound pavements, it is often assumed that rutting occurs mainly as deformation of the subgrade due to the vertical compressive strains induced by traffic loading over the design life of the pavement. However, rutting can also be indicative of deformation or failure within the base and subbase layers. As a consequence, designers use rutting in conjunction with other parameters, such as pavement deflection, in order to establish the cause of pavement failure in some pavement rehabilitation design methods. For example, Jordaan (1986) in Figure 6.1 illustrates the relationship between rut depth and deflection. The shaded area represents high rutting that corresponds with low deflections thus indicating poor base/subbase performance.

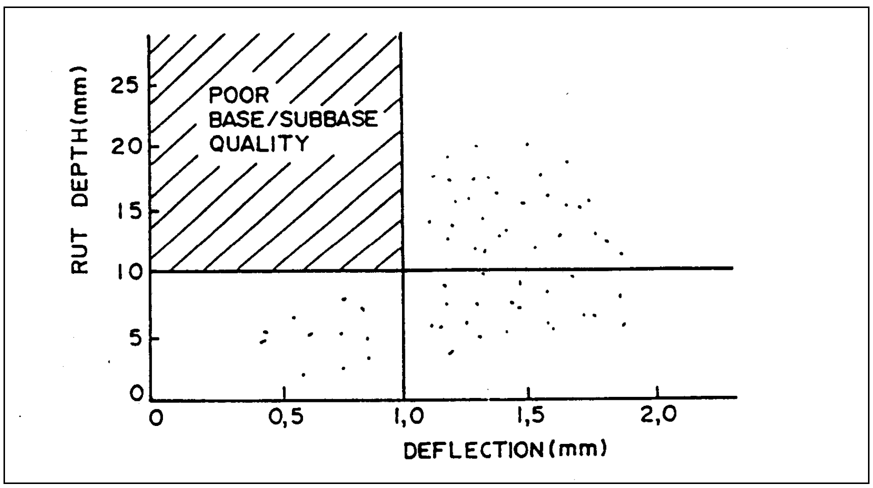


Figure 6.1: Plot of Deflection and Rut Depth Indicating the Cause of Pavement Failure (Jordaan, 1984)

Rut progression is also an important performance indicator of asphalt surfaced pavements since they are prone to plastic deformation of the asphalt layer. Asphalt pavements fall outside the scope of this research and will therefore not be discussed further.

This chapter describes the process used to develop the rut progression models applicable to New Zealand flexible, unbound pavements. Given that rutting develops in distinct stages, each stage is discussed individually. The development associated with each stage was undertaken according to the following steps:

- The existing HDM-4 expressions were calibrated using existing data;
- Subsequently, new model formats were investigated by reviewing exploratory statistics and regression analysis; and,
- Once a satisfactory model format was developed, it was tested and calibrated using the LTPP data.

6.2 Analysis Objectives and Data Use

6.2.1 Objectives

The main objective of the rut model development was to develop an accurate model for use on NZ State Highways. In all instances, the HDM-4 models were used as a base case for the model development. Given that originally the HDM-III and later the HDM-4 models were adopted during the PMS implementation in New Zealand (Henning et al, 2004b), it was important to demonstrate the full capabilities of these models to predict rut progression. Where alternative models were proposed, it had to be evident that these models would give more robust predictions of the actual pavement behaviour.

Specific objectives for the model development are discussed in detail in subsequent sections.

6.2.2 Rut Progression Stages

It is widely accepted that pavements deteriorate in three stages, which are particularly evidenced in rut progression. These stages can be seen in Figure 6.2 (Martin, 2003) and are described below:

- an initial densification stage that lasts for a short time after construction;
- a stable progression stage during which the progression occurs at a relatively constant rate; and
- an accelerated progression stage that represents rapid failure of the pavement towards the end of its design life.

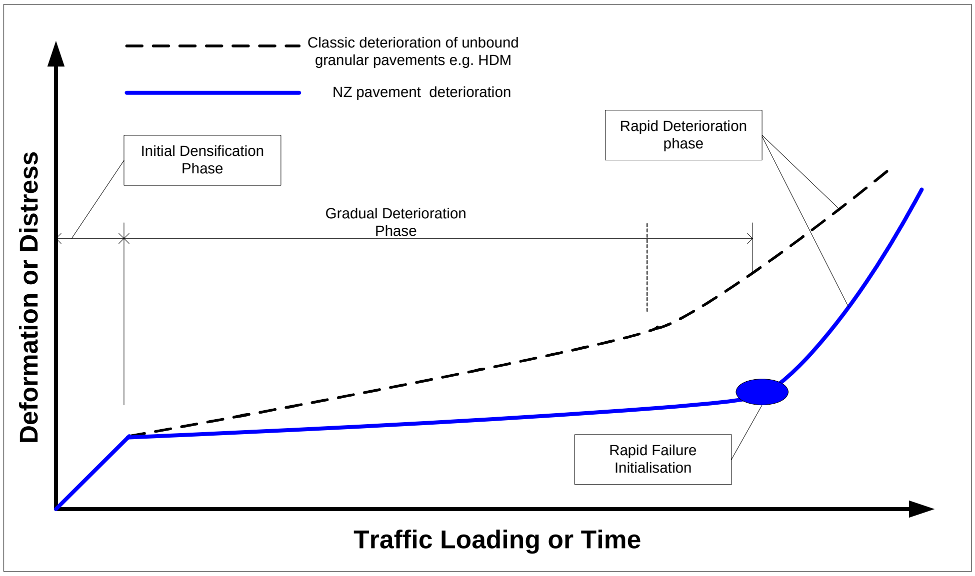


Figure 6.2: Deterioration Phases for Sealed Granular Pavements (based on Martin, 2003)

There are two methods that simulate the above three stages. In the HDM-4 models two models are provided, one for the initial densification and another for the progression model (NDLI, 1995). An exponential model format is provided for the rut progression, suggesting an increased rut rate for the later years of the pavement design life.

Martin (2003) presents another approach that considers the three progression stages separately. Rut prediction models are proposed for the initial densification and stable progression stages. However, no predictive model is presented for the accelerated stage as this was beyond the acceptable level of service for Australian roads.

In New Zealand, three-stage progression was also observed in the LTPP data. In particular, observations on low volume/low strength roads suggest a very low rut progression during the stable stage, and a rapid accelerated rut rate towards the end of the pavement life. These observations resulted in the first aim for this analysis:

Aim 1: To investigate the most appropriate model format that represents the three-stage progression behaviour of flexible unbound pavements.

6.2.3 Incorporating Accelerated Pavement Data in the Experiment

The New Zealand LTPP sections include a range of pavement conditions and ages. However, given the relatively young age of the programme, few pavements contain accelerated deterioration data. For this reason, additional data was sourced from the CAPTIF (Canterbury Accelerated Pavement Testing Indoor Facility) experiment that tests all pavements to a failure point. This is described further in Section 6.2.5. The combination of Long-Term performance data and accelerated pavement testing data has been used with success by Martin (2004). Consequently, a similar approach was adopted in order to develop models for New Zealand.

Aim 2: To develop the most appropriate methodology in order to utilise both the LTPP and CAPTIF data in model development.

6.2.4 Simplify Model Formats

A common view of the HDM-4 models is that they are relatively complex, and in some cases difficult to calibrate. In addition, the HDM-4 models sometimes use variables that are not normally collected at network level on New Zealand networks. For example, construction quality increases the robustness of deterioration models, however data related to construction quality is seldom collected at a network level.

Aim 3: To simplify the models where appropriate, and to use only variables that are commonly collected on New Zealand networks.

6.2.5 Data Use - Combining the LTPP Rut Data with the CAPTIF Data

CAPTIF is located in Christchurch, New Zealand. It consists of a circular track, 58 m long (on the centreline) contained within a concrete tank 1.5 m deep and 4 m wide, so that the moisture content of the pavement materials can be controlled and the boundary conditions are known. A central platform holds the machinery and electronics needed to drive the system. A sliding frame is mounted on this platform, which can move horizontally by 1 m. This radial movement enables the wheelpaths to be varied laterally, and can be used to have the two 'vehicles' operating in independent wheelpaths. An elevation view is shown in Figure 6.3.

At the end of this frame, two radial arms connect to the Simulated Loading and Vehicle Emulator (SLAVE) units shown in Figure 6.4. These arms are hinged in the vertical plane so that the SLAVEs can be removed from the track during pavement construction, profile measurement, etc., and in the horizontal plane to allow for vehicle bounce.

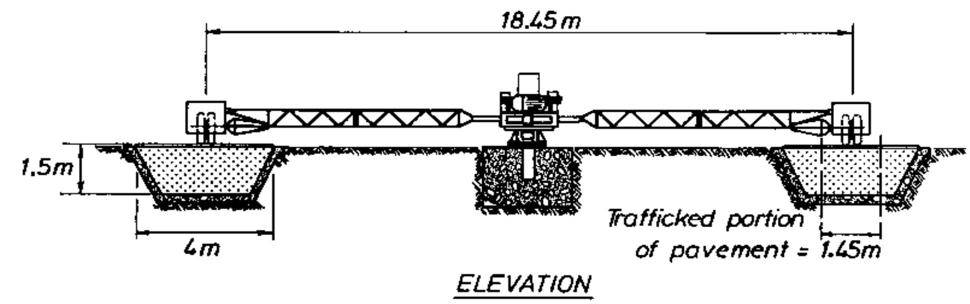


Figure 6.3: Elevation view of the CAPTIF testing equipment (Alabaster and Fussell, 2006)

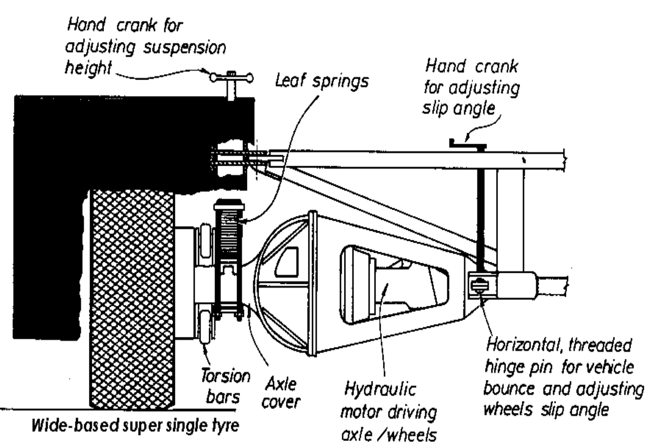


Figure 6.4: Diagram of the key components of the CAPTIF SLAVE unit (Alabaster and Fussell, 2006)

The CAPTIF data is ideal for investigations into the rut progression of pavements, given the controlled conditions for construction and monitoring of the pavement wear. Therefore, as opposed to in-service pavements, the researchers have a clear understanding of the pavement make-up and the construction quality. By removing the uncertainty associated with these factors, a much better understanding can be obtained of the long-term behaviour of the pavement.

The data used for the initial densification model development was sourced from the PR3-0810 Fatigue CAPTIF experiment (Alabaster, et al. 2006). Five pavement types were investigated and are listed in Table 6.1. Note that all the sections were surfaced with either Asphalt Concrete (AC) or Open Graded Porous Asphalt (OGPA). Earlier work at CAPTIF suggested poor performance of chip seal surfaces under the loading conditions, while the pavement behaviour underneath was similar regardless of the surface type used.

Table 6.1: Pavement Sections Tested with the CAPTIF Experiment (based on Alabaster et al, 2006)

Section	Surface Material	Subgrade Material	Avg CBR	Min CBR	Max CBR	Base Layer Thickness	Pavement Classification
A	AC	Tod[1] OMC[2]	7	7	8	150	thin + strong
B	OGPA	Tod OMC	9	9	10	150	thin + strong
C	OGPA	Tod OMC + 10%	2	2	2	150	thin + weak
D	OGPA	Tod OMC + 10%	3	2	4	300	thick + weak
E	OGPA	Tod OMC	8	8	9	300	thick + strong

Notes:

1 - “This material has been named Tod Clay after the owner of the pit from which it was excavated. The soil has a workable consistency due to the mica content and has a relatively low susceptibility to shrinkage and swelling due to the predominant kaolin mineral.” (Steven et al, 1999)

2 - OMC- Optimum Moisture Content

The original idea of combining the LTPP experiment with an accelerated pavement testing programme was taken from Martin (2003) and Martin et al. (2004). This research used Accelerated Load Testing (ALF) in order to estimate the relative performance factors for all the maintenance treatments for rutting and roughness in Australia. Given that this research gave reasonable results, it provided confidence that this project could include the CAPTIF data for developing pavement deterioration models.

Other benefits arising from using the LTPP data in conjunction with the CAPTIF data include:

- **Gaining a better understanding of the environmental impact on pavements:** The LTPP sections were 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
- **This research was also of benefit to the CAPTIF studies** by confirming results observed for the PR3-0810 Fatigue CAPTIF experiment (Alabaster, et al. 2006) on the basis of the LTPP data.

6.3 HDM Rut Models

6.3.1 HDM-4 Initial Densification Model

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

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

Only the first two components of the rut progression are relevant to New Zealand conditions. Plastic deformation is particularly relevant to asphalt basecourse pavements, which are not part of this research. Also, studded tyres are not used in New Zealand. This section gives the detail of the initial densification model, while structural deformation is presented in Section 6.5.

6.4 Predicting Initial Densification

6.4.1 HDM Initial Densification Model

The HDM-4 initial densification model is given by (NDLI 1995):

$$RDO = K_{rid}\left[a_0\left(YE4x10^6\right)^{a_1+a_2DEF}\right]SNP^{a_3}COMP^{a_4}$$ **Equation 6.1**

Where RDO = the rutting due to initial densification (mm)

Krid = calibration coefficient for initial densification

YE4 = annual number of Equivalent Standard Axles (ESA)(millions/lane)

DEF = Maximum Benkelman Beam deflection (mm)

SNP = adjusted structural number of the pavement

COMP = relative compaction (%)

a_i = model coefficients

6.4.2 Testing the HDM-4 Initial Rut Depth Model on LTPP Data

There were only three State Highway LTPP sections that were new or reconstructed since 2001. A full calibration of the HDM-4 model was therefore not possible since initial rut depth values were not available. However, the HDM-4 model predictions were compared with the recorded initial rut depth results for these three LTPP sections.

Figure 6.5 illustrates the output from one of these comparisons. It shows that some of the actual rutting varies significantly with the predicted initial rutting (estimated to be just less than 3mm). Some of the actual initial rutting was as high as 7mm.

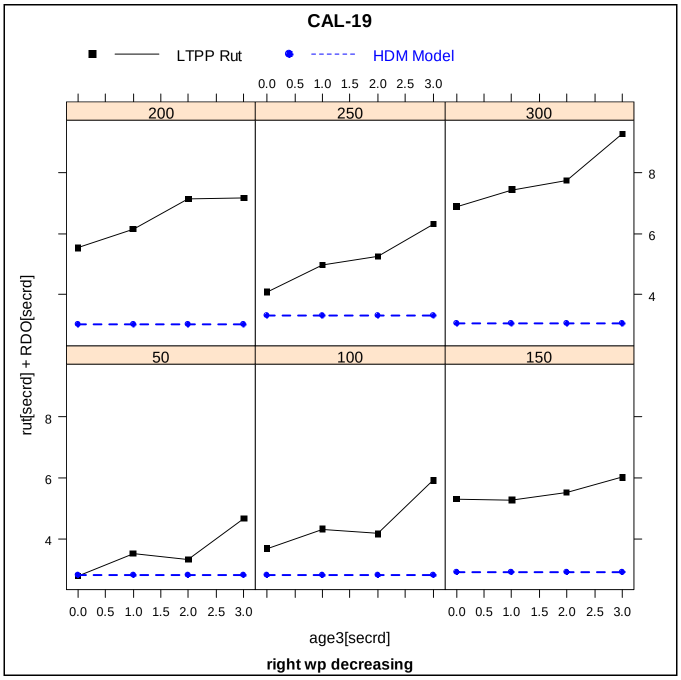


Figure 6.5: Comparing Predicted versus Actual Initial Rut Depths on LTPP Section – CAL-19 (decreasing chainage)

In Figure 6.5,

- Each block represents the average 50 m rut depth at different pavement ages (age3); 50, 100 and 150 indicated the end point reference for each section.
- The vertical axes indicate the rut depth (predicted and actual in mm).

Some observations from these output comparisons include:

- The difference between the predicted and the actual rut depths did not show any distinct pattern. There were no trends observed in relation to the assumed strong/weak pavements, and no trends observed in relation to the left and right wheel-paths. It was somewhat unexpected to record relatively high initial densification rut depths in the right wheel paths;
- A reduction in rut depth compared to the initial densification rut depth for some sections was not uncommon; and
- Some sections demonstrated a significant rut progression within the first three years of the section age.

Based on this limited data, it was inconclusive regarding the appropriateness of using the HDM-4 model on the State Highways.

6.4.3 Initial Densification Model Form Development Based on CAPTIF Data

6.4.3.1 Defining Initial Densification for CAPTIF Data

The challenge in using the CAPTIF accelerated pavement testing data was to establish the point of initial densification cessation. For this, the cumulative variation from the mean of the rut values was used (CUSUM). According to this method, any slope change in CUSUM plot would indicate a significant change in the behaviour of the pavement (Bennett, 2004).

As shown in Figure 6.6, it was observed that there was an unstable rut development for the initial phase up to approximately 50,000 load cycles. After 50,000 cycles, most rut development becomes stable for longer periods. This equates to approximately 100,000 Equivalent Standard Axles, which approximates to 12 months of traffic with an AADT of 2,500 and a 15% heavy vehicles.

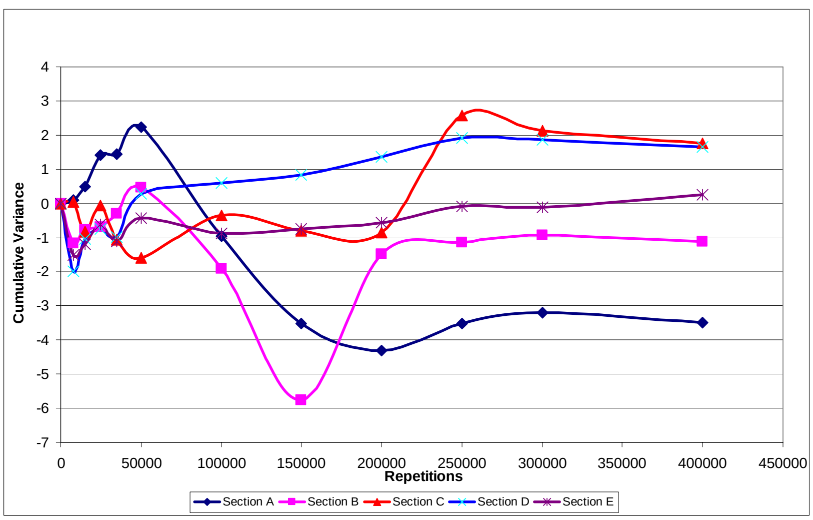


Figure 6.6: CUSUM Plot for the Rut Development on CAPTIF Data

6.4.3.2 Exploratory Plots

Exploratory plots were reviewed for 12 independent variables for the test pavements listed in Table 6.1. These variables ranged from pavement type, material properties, deflection data and loading information and are listed in Table 6.2

Table 6.2: CAPTIF Data Variables Used in Model Development

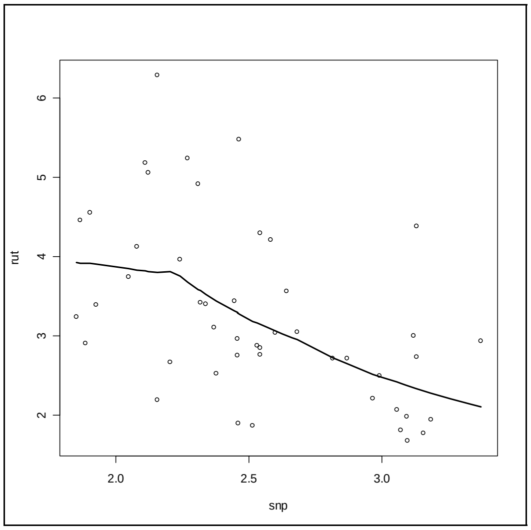
Variable	Description	Variable Type
type	Pavement Type (Refer to Table 6.1)	Text
DEF	Peak deflection in mm	Number
SNP	Pavement structural number	Number
COMP	Relative compaction in %	Number
Layer	Layer type	Text
DD	Dry density	Number
WD	Wet Density	Number
MC	Moisture content (in %)	Number
PR	Density	Number
%SAT	Per cent saturation	Number
CBR	Californian Bearing Ratio	Number
REPS	Wheel load repetitions	Number

The possible significant variables identified for predicting initial rut depth included:

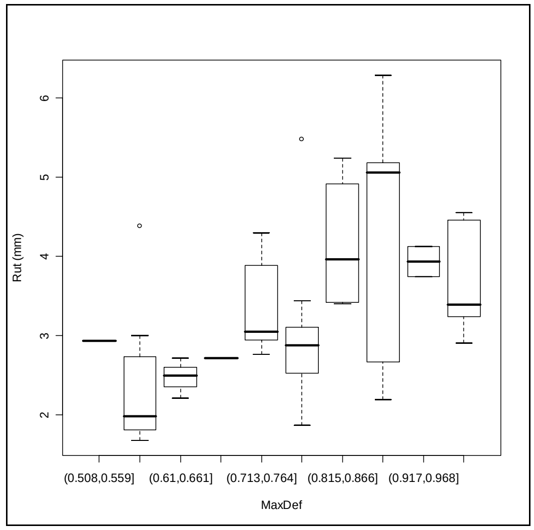
- Subgrade strength measured as CBR (Californian Bearing Ratio);

- Moisture content;
- Modified Structural Number (SNP); and
- Maximum Peak Deflection measured with the Falling Weight Deflectometer.

Examples of the exploratory plots for these variables are presented in Figure 6.7 and Figure 6.8.



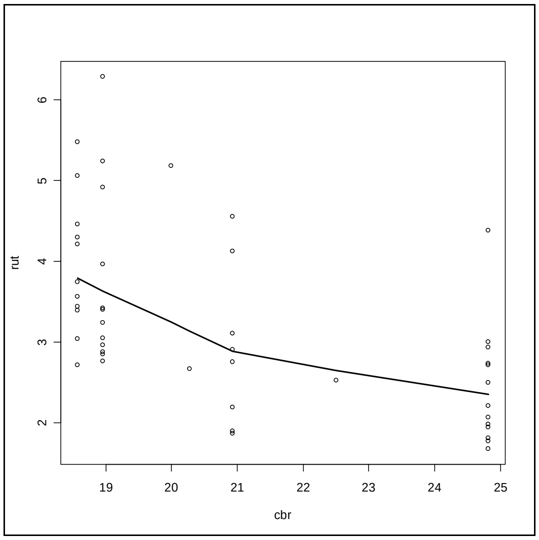
Initial Rut Depth (mm) as a Function of Modified Structural Number (SNP)



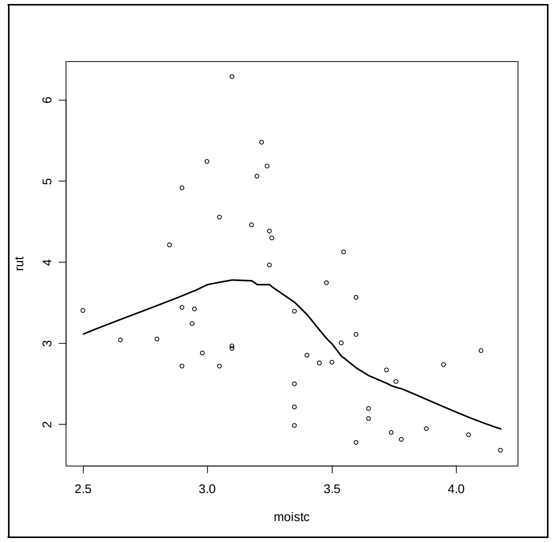
Initial rut Depth (mm) as a Function of Maximum Peak Deflection

Figure 6.7: Plots of Significant Factors Identified for Initial Rut Depth

Figure 6.7 illustrates the initial rut depth as a function of the Modified Structural Number (SNP) and the maximum Peak Deflection (from FWD measurements). The SNP is derived from the deflection measurements, pavement layer thickness and the subgrade CBR. For that reason, it was expected that a similar trend between the initial rut depth and these two variables would be noticed. With an increase in structural number (decrease in deflection), there is a significant reduction in the initial rut depth. There is a significant scatter of the data for the relationship between these factors. Depending on the outcome of the regression analysis, which is discussed in Section 6.4.3.3, either the SNP or the maximum deflection, can be used in the prediction of initial rut depth. However, because of their co-linearity, both of these factors should not be used as independent variables in the expression.



Initial Rut Depth (mm) as a Function of CBR



Initial Rut Depth (mm) as a Function of Moisture Content (%)

Figure 6.8: Initial Rut Depth as a Function of CBR and Moisture Content.

Figure 6.8 indicates the CBR relationship with the initial densification/rutting, with higher CBR values yielding a lower initial rut depth. CBR is on of the factors represented in the SNP. Therefore, combining these factors together with the SNP should be avoided in order to avoid co-linearity.

All the CAPTIF test sections were compacted to 95% of their maximum dry density at their respective optimum moisture content. For that reason, the level of compaction did not indicate a trend with the initial rut depth. Hence, it was not possible to include a compaction factor or moisture content in the final model.

Compaction during construction does, however, have an impact on the resulting initial densification. If sub-standard compaction is performed during construction, there is a greater likelihood of high initial densification. However, on the New Zealand State Highways, the compaction requirements for base courses are similar, plus the achieved density is seldom recorded for the complete network. For that reason, it was decided to include only those variables that are recorded on a network basis. In addition, the deflection measurements are an effective indicator of the layer stiffness, which is a strong function of the level of compaction achieved.

6.4.3.3 Regression Analysis Result for Initial Densification

The regression analysis performed on the CAPTIF data yielded results which are consistent with observations made in the previous section. Although it was not appropriate to use certain factors in the final model, they were still included in the model during the initial analysis. Table 6.3 lists the results from the step-wise model regression. The significant factors in the model predicting rutting initial densification were moisture content, maximum peak deflection and structural number. The regression resulted in a relatively low Akaike's Information Criterion (AIC)[5] of 124. A similar regression was also completed with SNP instead of the maximum peak deflection and yielded similar results.

Table 6.3: Linear Model Regression for Rutting Initial Densification.

	Estimate	**Std. Error**	**t value**	**Pr(>|t|)**	**Significance Level**
(Intercept)	1.30	0.451	2.89	0.006	**
moisture content	-0.38	0.093	-4.12	0.000	***
max deflection	1.35	0.313	4.33	$8.70*10^{5}$	***
thickness	0.13	0.085	1.50	0.14	

Note: Significance codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1 – **AIC: 124.65**

The next stage of the analysis tested whether the factors had to be used in linear or logarithmic transformed format. This analysis resulted in the maximum deflection, in logarithmic format having a t value of 3.86 and a significance Pr(>|t|) of 0.004. Therefore, there was no increase in the accuracy of the model by using a log-transformed maximum deflection.

[5] The AIC is like a fault term, with the lower values indicating better fit with the observed data. The best model (i.e. most significant variables) is determined by finding the best combination of variables in order to minimise the AIC.

Figure 6.9 illustrates the residual plots for the linear model given in Table 6.3 that predicts the initial rut densification. With these residual plots, the ideal is to have a uniform distribution of the residuals, that is the difference between predicted and actual values. If a non-uniform distribution is observed, it may be possible that another model format could improve the overall model. As shown in Figure 6.9, top right plot, a slightly non-uniform distribution of the residuals was recorded.

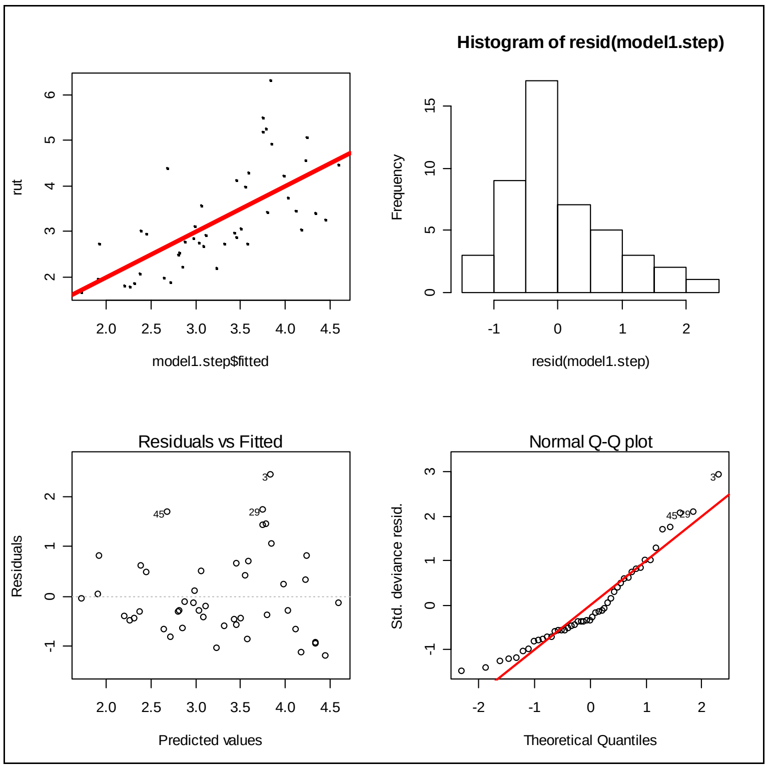


Figure 6.9: Residual Plots for the Linear Model Predicting Initial Rut Densification.

In order to improve this fit, all the data was transformed into a logarithmic function and a significantly better residual outcome was achieved as indicated in Figure 6.10. Also, the overall model had an improved fit with an AIC of 4.9 compared to the 124.65 of the linear model, suggesting a much more appropriate model format for the prediction.

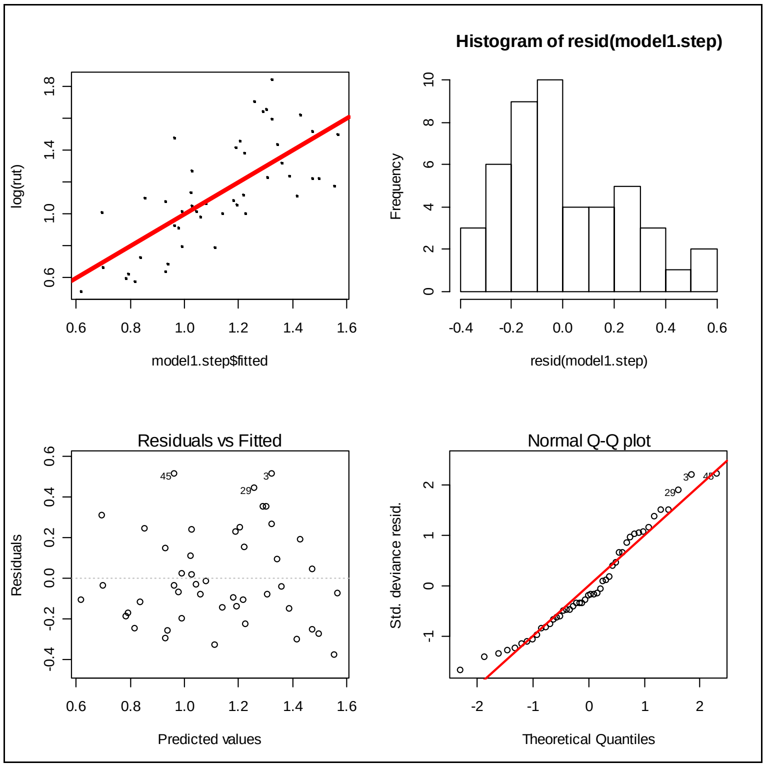


Figure 6.10: Residual Plots for the Linear Model Predicting Initial Rut Densification (Logarithmic Transformed Data).

Based on the results presented in Table 6.3, Figure 6.9 and Figure 6.10, two more aspects needed further investigation, namely:

- The inclusion of the moisture content is of concern given the reasons explained in section 6.4.3.2. It is acknowledged that the moisture content affects the rut progression. However, the moisture content is assumed to be close to optimal during construction and is excluded from the initial densification model; and,
- The original HDM-4 model includes both the maximum deflection and the SNP. Both these factors and the inter-relationship between them should be investigated in the initial rut densification model. However, according to this research, these two variables have a strong co-linear relationship and therefore both should not be used.

The model regression was repeated with testing most of the possible combinations of SNP and base layer thickness included, and the moisture content being excluded. The resulting model coefficients are listed in Table 6.4 and the residual plots are presented in Figure 6.11. The results indicated a satisfactory model outcome.

Table 6.4: Linear Model Regression for Initial Rut Densification (based on CAPTIF Data).

	Estimate	Std. Error	t value	Pr(>|t|)	Significance
(Intercept)	2.44	0.279	8.766	3.30E-11	***
snp	-0.551	0.119	-4.635	3.19E-05	***
thickness.f300	0.161	0.099	1.633	0.11	

Note: Significance codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1 – **AIC: 18.04 R^2 = 0.31**

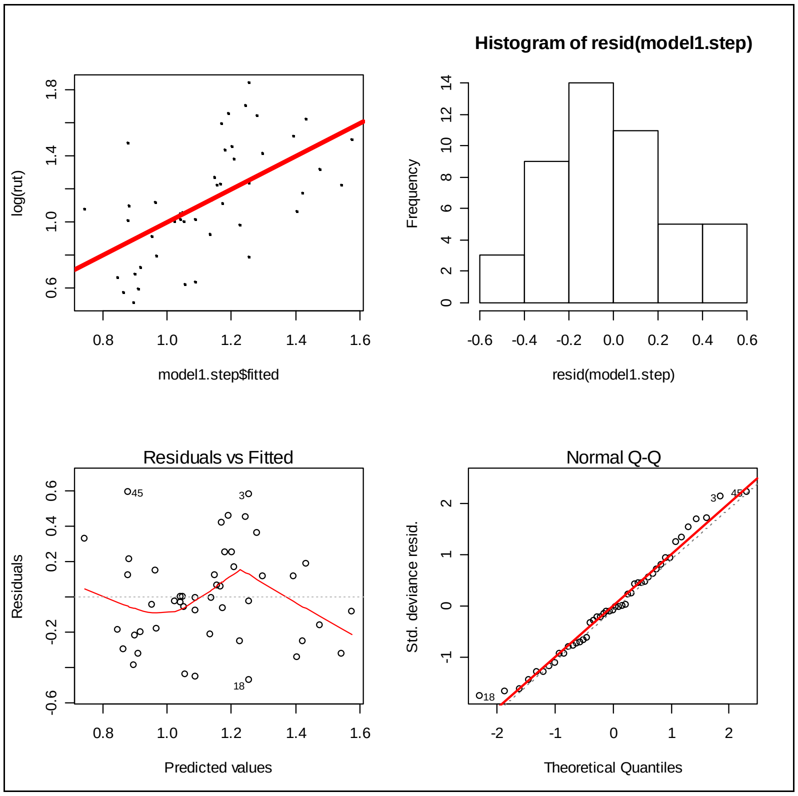


Figure 6.11: Residual Plot for the Predicted Initial Rut Depth.

Figure 6.12 shows the model outcome for the HDM and the CAPTIF Data derived model. Note that the model as reported in Table 6.4 had to be calibrated in order to fit with the actual LTPP data.

The calibrated and final recommended model is:

$$Initial\_Rut = 3.5 + e^{(2.44 - 0.55SNP)} \qquad \textbf{Equation 6.2}$$

Where Initial Rut = the rutting due to initial densification (mm) that will occur within the first year after construction.

SNP = is the Modified Structural Number, calculated from FWD measurements.

Figure 6.12 illustrates that the HDM-4 model predictions have a larger variance in predicted initial rut depth, since it considered more factors than just the SNP. However, the general trend is similar to the model developed on the CAPTIF data. It should also be noted that a significant variance between the LTPP observed and the predicted initial rut depth remains.

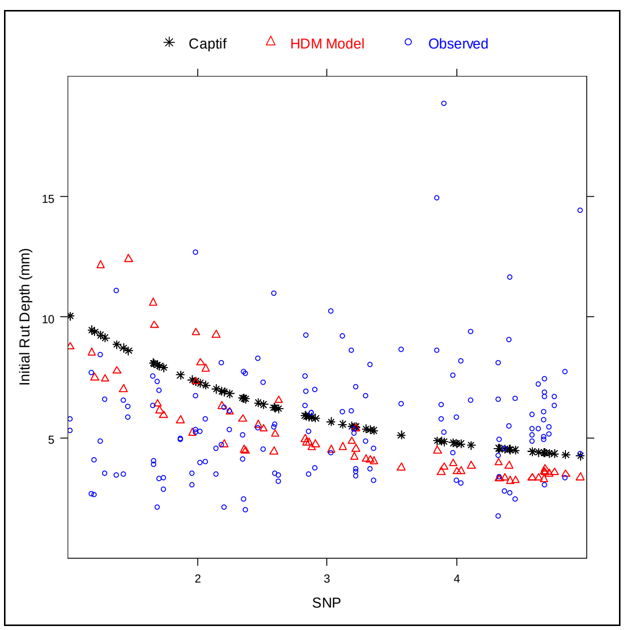


Figure 6.12: Plot of the Initial Rut Model Developed on the CAPTIF Data (Plotted Against LTPP Observed Data).

6.5 Rut Progression

6.5.1 HDM-4 Rut Progression Model

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

Structural deformation for un-cracked sections

$$\Delta RDST_{uc} = K_{rst}\left(a_0 SNP^{a_1} YE4^{a_2}\, COMP^{a_3}\right)$$ **Equation 6.3**

Structural deformation after cracking

$$\Delta RDST_{crk} = K_{rst}\left(a_0 SNP^{a_1} YE4^{a_2}\, MMP^{a_3}\, ACX^{a_4}\right)$$ **Equation 6.4**

Where: $\Delta RDST_{uc}$ = incremental increase in structural deformation for uncracked sections in the analysis year (mm)

$\Delta RDST_{ckr}$ = incremental increase in structural deformation for cracked sections in the analysis year (mm)

K_{rst} = calibration coefficient for structural deformation

YE4 = annual number of ESA (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

Default model coefficients are provided for chipseal pavements and also for asphalt pavements. As previously mentioned, asphalt pavements are not part of this research.

6.5.2 Calibrating the HDM-4 Rut Progression Model

The HDM-4 rut progression model was calibrated according to the methodology described in Bennett and Paterson (2000). The results of this calibration process are listed in Table 6.5.

Table 6.5: Calibration Result of the HDM-4 Rut Progression Model

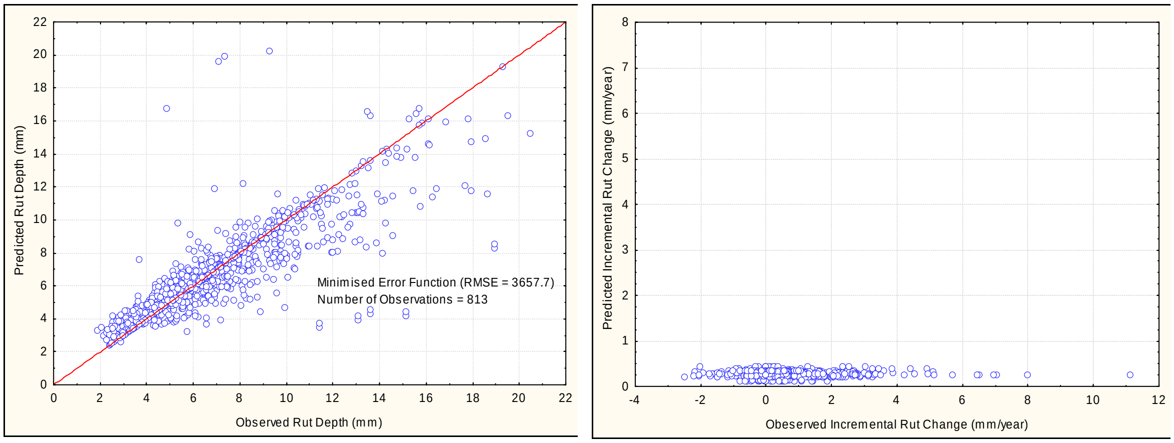
Sensitivity Risk Area[1]	Rut Progression Calibration Coefficient (Krp)	Error Function (RSME-square root of the difference between predicted and actual)[2]
Low and Limited	1.03	2,719 (2,729)
Medium and High	0.98	931 (933)
All Data	1.01	3,658 (3662)

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)

The calibration results suggest 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. Therefore, first impressions indicate that the rut model would closely reflect the actual behaviour of the pavements. A closer investigation of the graphical outputs suggests otherwise. Figure 6.13 depicts two plots that compare the predicted rut depth with the actual observations on the LTPP sections.



Comparing the observed and predicted rut depth for the calibrated HDM-4 model

Comparing the observed and predicted rut depth incremental change for the calibrated HDM-4 model

Figure 6.13: Comparing the Calibrated Rut Progression Model with Observed Data

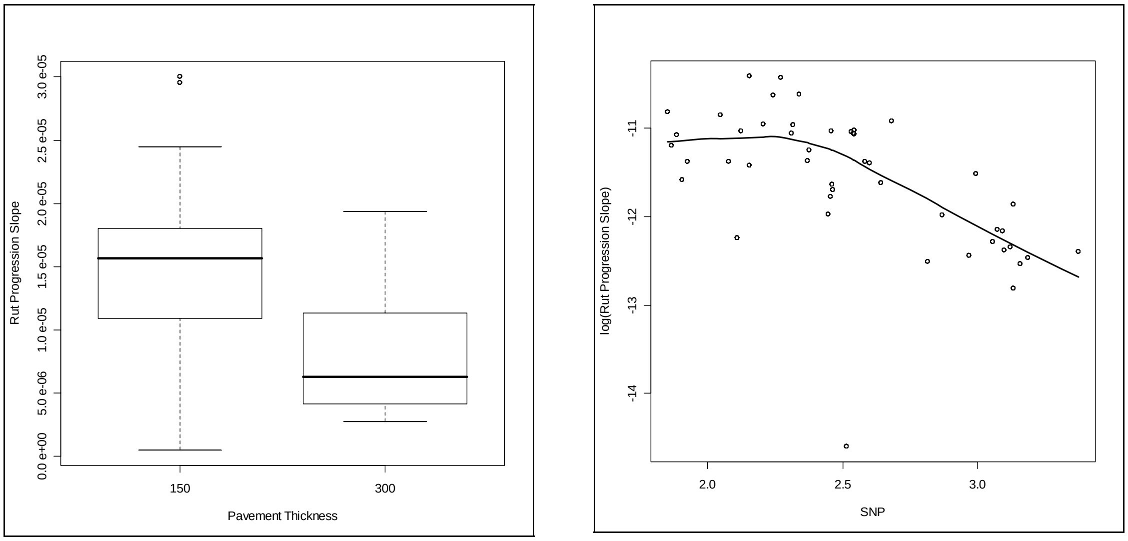
Again, the results seem to be promising when considering the absolute predicted rut depth with the observed rut depth (left hand plot). Although a significant variability exists, the predicted rut depth values are centred uniformly around the line of equality. However, the predicted and actual incremental change of rutting reveals a different trend (right hand plot). It is observed that the predicted change in rut depth centres around an average value of 0.3 mm per year, and has no relation to the actual observations. Where the predicted values vary little from the average, the observed incremental rut change varied between 0 to 8 mm per year.

Based on these results from HDM-4, it was decided to attempt the development of a completely new model form. The most significant change suggested is to include the separation of the rut progression into two phases, the stable rut progression and the accelerated rut progression as discussed earlier in Section 6.2.2.

6.5.3 Predicting Rut Progression Based on CAPTIF Data

6.5.3.1 Exploratory Plots

All the available factors given in Table 6.2 were tested for significance in predicting the rut rate. Of these, the most significant factors were found to be the pavement thickness and the structural number. Figure 6.14 depicts the relationship between rut progression slope and pavement thickness (left hand plot) and structural number (right hand plot).



Rut Progression Slope for Different Thicknesses

Rut Progression Slope as a Function of Structural Number (SNP)

Figure 6.14: Example of Exploratory Plots Investigating Trends with Stable Rut Progression Slope

The results from Figure 6.14 suggest:

- There is a significant difference in rut progression slope for the different pavement thicknesses. Thin pavements (thinner than 150mm), have a higher rut progression slope compared to the thick pavements;
- There is in an decrease in rut progression rate for an increase in pavement strength (SNP).

The inter-effects of these two variables were also investigated and are illustrated in Figure 6.15.

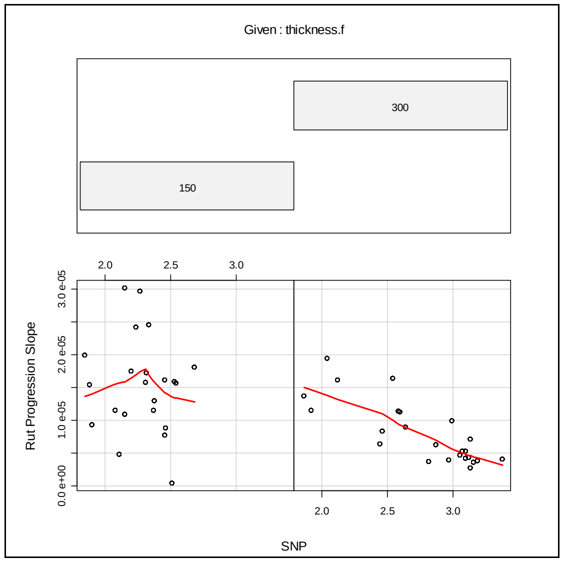


Figure 6.15: Rut Progression Slope for Different Thicknesses and Structural Number (SNP)

The results from Figure 6.15 suggest:

- There appears to be a different trend between the rut rate and SNP for the two thickness categories;
- The relationship between the rut rate and SNP has a larger scatter of data for the thinner pavements (left hand plot) when compared with the thick pavements (right hand plot);
- A stronger linear relationship between rut rate and SNP exists for thicker pavements than for thin pavements in the next section.

The relationship between the rut progression rate and the SNP is not necessarily linear and consequently different model forms were investigated during the regression analysis in the next section.

6.5.3.2 Regression Analysis

As a first attempt, the regression on the rut rate slope was performed to include all possible variables. Most of the variables resulted in being significant only in an additive format. Therefore, none of the variables could be considered as significant independent variables in their own right.

Since it was not possible to identify significant independent variables from the regression, the next step was to undertake a regression on the variables identified in the exploratory statistics. Table 6.6 lists the results from this analysis and Figure 6.17 depicts the residual plots.

Table 6.6: Regression Results Obtained for the Linear Model on the CAPTIF Rut Rate data.

	Estimate	Std. Error	t value	Pr(>|t|)	Significance Level
(Intercept)	2.26E-05	2.71E-06	8.31	3.75E-16	***
snp	-3.14E-06	1.18E-06	-2.66	0.008	**
thickness.f300	9.66E-06	3.18E-06	3.03	0.003	**
snp:thickness.f300	-5.63E-06	1.32E-06	-4.26	2.25E-05	***

Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

Residual standard error: 5.286e-06 on 842 degrees of freedom

Multiple R-Squared: 0.43,   Adjusted R-squared: 0.43

F-statistic: 211.1 on 3 and 842 DF, p-value: < 2.2e-16

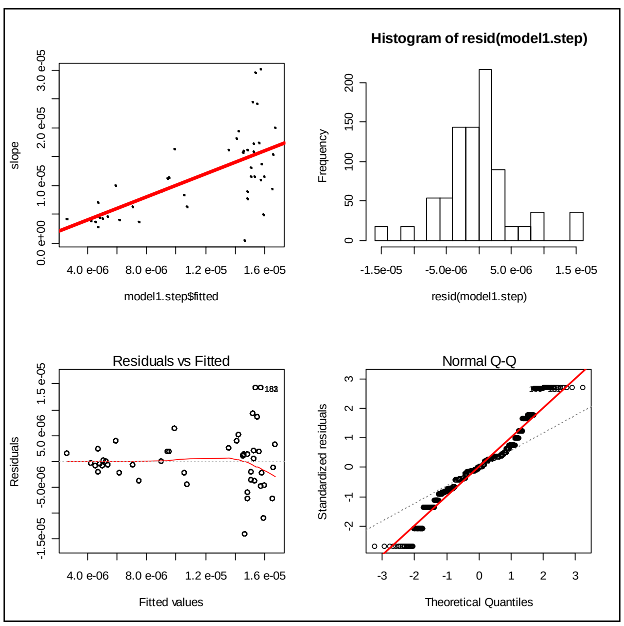


Figure 6.16: Residual Plots for the Rut Progression Slope Linear Regression.

The main conclusions from the regression results were:

- Although a R^2 of 0.43 is acceptable for the modelling of road pavements, it should be noted that with the CAPTIF experiment all data was obtained under controlled conditions. However, the model fit was significantly better than the HDM model presented in Figure 6.13; and,

- The residual plots indicated the model to be unstable at the extreme of the data. This is indicated by both the residual vs. fitted plot and the Q-Q plot. Normally, this phenomenon indicates the potential for a better model format.

Given the above, a logarithmic model format was attempted. However, poorer results were obtained. The resulting R^2 from this analysis was 0.35 and as illustrated in Figure 6.17, the residuals are not 'Normally' distributed, again suggesting the model to be of the wrong format. Also noticeable in Figure 6.17, the extreme points of the data are more prominent in the Q-Q plot.

However, as no other model format tested gave any better results, it was decided to proceed with the linear model format. It should also be noted that the analysis in this section did not include any pavements with structural numbers higher that 3.5. Hence, any extrapolation of this model above an SNP of 3.5 is not recommended and could give inappropriate results.

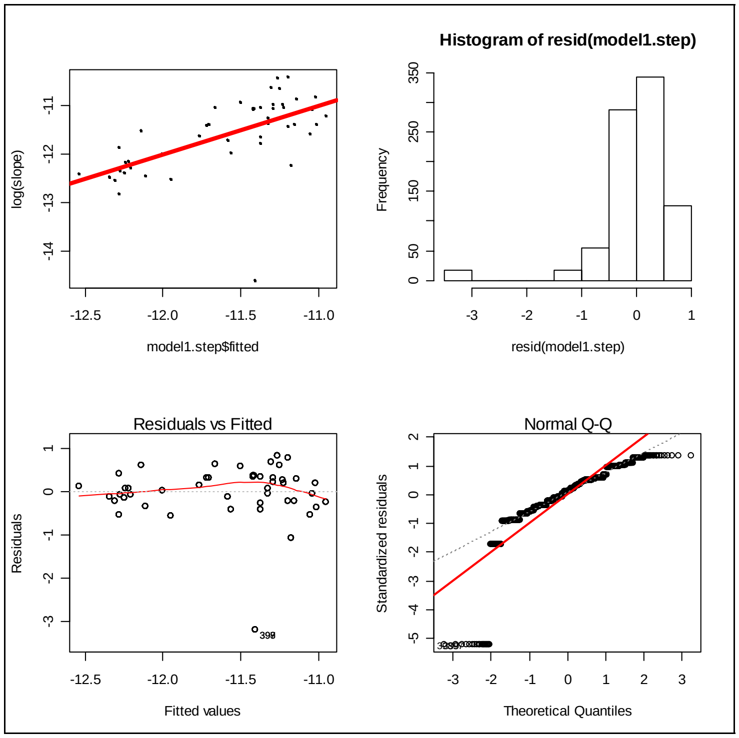


Figure 6.17: Residual Plots for the Rut Progression Slope Logarithmic Regression.

Based on the above results, it was decided to use the following models in linear format given by:

Thin pavements:

$$RPR = 9.94 - 1.38 \times a_1 SNP$$ **Equation 6.5**

Thick pavements (>150mm):

$$RPR = 14.2 - 3.86 \times a_2 SNP$$ **Equation 6.6**

Where: RPR = Stable rut progression rate in mm/million ESA

SNP = Modified structural number;

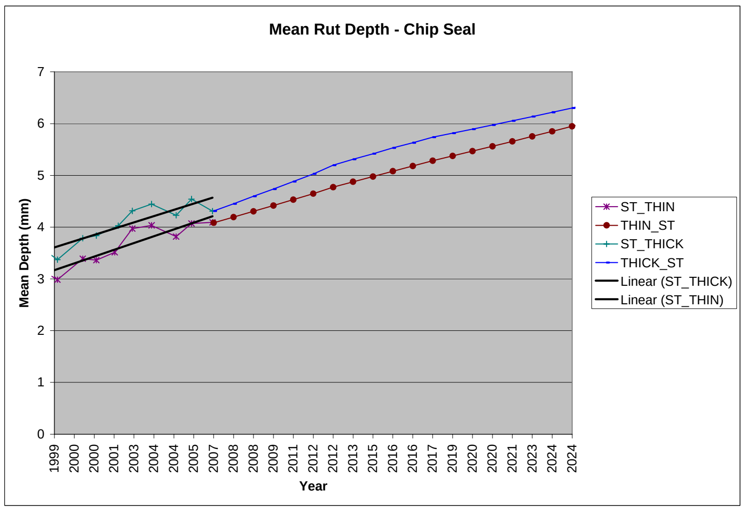
a_1, a_2 = Model/Calibration coefficients

Note:

The above equation was converted from values presented in Table 6.6 using the conversion of 1 ESA = 0.44. Repetitions

6.5.4 Testing the Rut Progression Model on Network Data

It was decided to test the rut progression models on network data in order to confirm its validity. The stable rut progression model was tested on a complete network data-set for the State Highway Gisborne and Hawke's Bay network. The outcome of this test is graphically represented in Figure 6.18 which illustrates both the historical and predicted rut progression for thin and thick chip seal pavement.



Note: ST_THIN is thin chip seal pavements (historical rutting data)

THIN_ST is thin chip seal pavements (predicted rutting data)

ST_THICK is thick chip seal pavements (historical rutting data)

THICK_ST is thick chip seal pavements (predicted rutting data)

Figure 6.18 Testing the Stable Rut Progression on a Complete Network Dataset (Hatcher, 2007)

Both the thick and thin predicted trends correlate well with the average performance observed for the past eight years. There is a varying slope for the predicted rut progression of thick chip seal pavements. This trend resulted from temporary variation in traffic loading on this portion of the network for a number of years (logging traffic). The model therefore successfully indicates the impact of traffic loading variances on the rut progression rates.

Based on the findings of this research plus the independent tests, linear rutting model is recommended for use within the NZdTIMS system.

6.6 Accelerated Rutting

6.6.1 Defining Accelerated Rutting

Accelerated or rapid failure of road pavements is a well understood phase of pavement performance. Most prominent literature describing this phenomena, refer to this pavement behaviour when observed from heavy vehicle simulator programmes such as the New Zealand CAPTIF Programme (Alabaster and Fussell, 2006), the Australian ALF project (Martin, 2003) and the South African Heavy Vehicle Simulator (HVS) programme (Thyese et al, 1996).

Figure 6.19 gives an example of pavement deterioration over time explained graphically. It shows pavement deterioration in the three phases as adopted for this research. Figure 6.19 also shows not all pavements will reach an accelerated distress phase before the rapid failure occurs. Some pavements can be classified as failing "prematurely" before their design or their expected service life is reached.

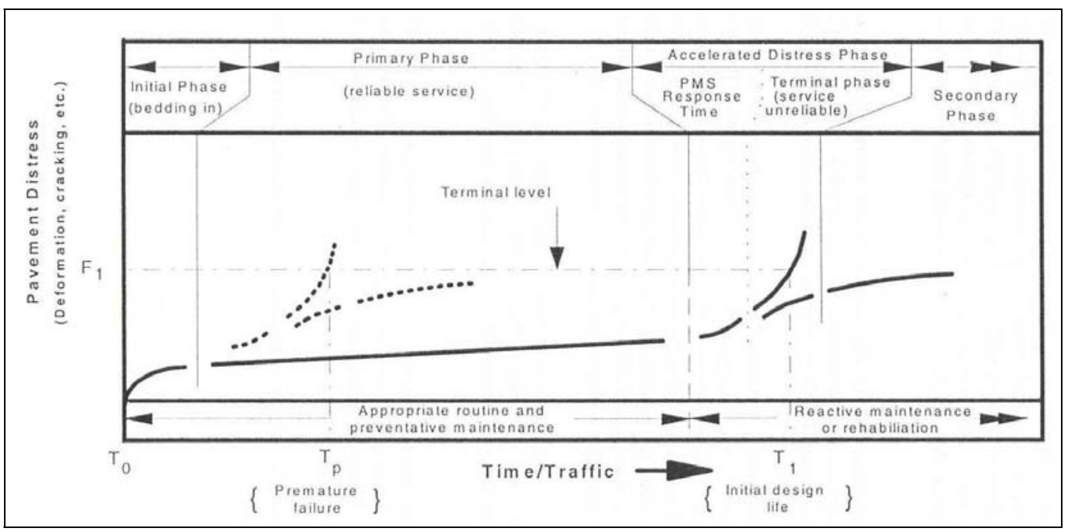


Figure 6.19: Stages in Road Deterioration (South African Department of Transport, 1997)

Most international models, such as HDM-4 (NDLI 1995), predict the rapid failure stage using a power or exponential model format. However, experience in New Zealand suggests that when pavements display an accelerated rut rate, they require immediate maintenance intervention, typically within 12 months. For that reason, knowing when accelerated rut initiates is of considerable important to maintenance programmers, rather than knowing the exact rut rate at a given date or time.

The definition of accelerated rut rate is strongly dependent on the pavement make-up of a particular network. For example, on a network with typically low strength granular pavements, the normal expected rut rate in New Zealand would be between 0.4 to 0.6 mm per year. On stronger pavement networks these values will be significantly lower.

For the purpose of this research, it was assumed that when the rut rate is **twice** the average expected rut rate, the pavement is considered to have initiated an accelerated rut rate. Defining the rut rate that would constitute accelerated rut rate then becomes a calibration requirement during the implementation of the model.

6.6.2 Exploratory Plots

For the purposes of this analysis, the CAPTIF rutting data was transformed into a binary format for the full duration of the loading period. This transformation is required for the development of a logistic model that predicts a discrete outcome of a true or false value. In this instance, a false value would be assigned to all rut rate values that are lower than the defined accelerated rut rate (say less than or equal to 1.2 mm). Once the pavement commences accelerated rut rates, the values switch to true (say, rut rate greater than 1.2mm).

Figure 6.20 depicts typical exploratory plots that investigate the significant factors which influence the timing of accelerated rutting. It displays two variables, the structural number (on the left hand plot) and the moisture content (on the right hand plot).

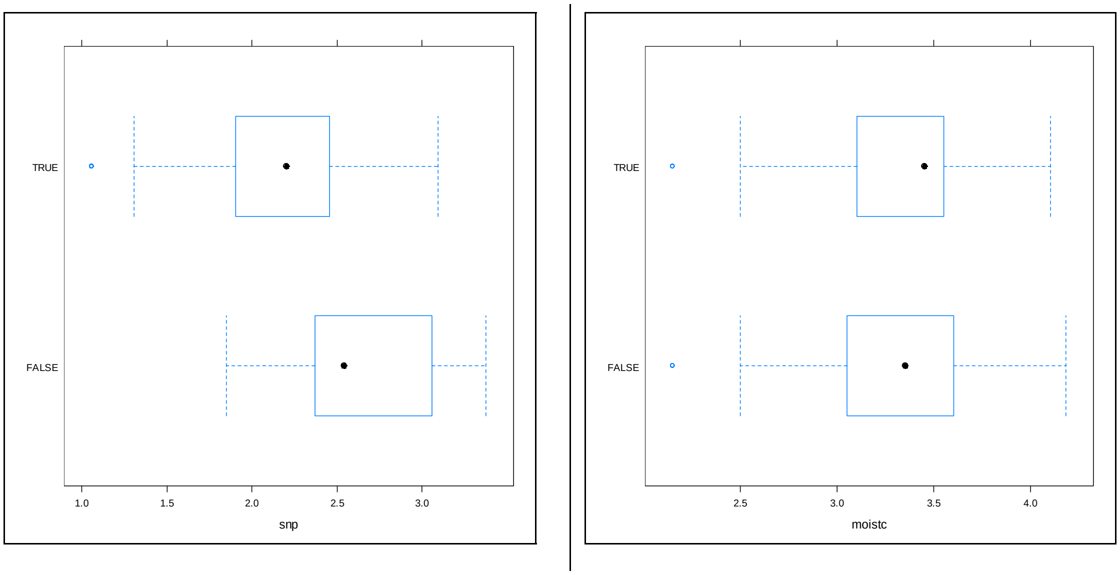


Figure 6.20: Accelerated Rut Progression versus Structural Number and Moisture Content Based on the CAPTIF Data

As expected, the structural number (SNP) has a significant influence on the likelihood of pavement failure manifesting in accelerated rut progression. As observed, pavements with higher SNP values would have a lower probability of accelerated rutting compared to pavements with lower SNPs under the same loading conditions.

In contrast, the moisture content showed little influence on the occurrence of accelerated rut progression. It should be highlighted however, that this moisture content is the compaction moisture, and does not assess moisture entering the pavements through cracks in the surface layer, or poor drainage conditions.

Additional observations from the exploratory statistics include:-

- Most of the accelerated rut progression took place on pavements with an SNP number lower than 2.4 and maximum deflection greater than 0.8 mm; and
- Accelerated rut progression only took place on pavements with layer thicknesses of 150mm or less.

The exploratory statistics confirmed the mechanism of accelerated rutting. This mechanism is largely found on low strength pavements that are over-stressed due to heavy vehicle loadings. Failure of these pavement types subjected to traffic with heavy loading normally occurs rapidly. Stronger/thicker pavements that are constructed according to specifications will not necessarily have a rapid failure. In these cases, the pavement deformation still takes place, but at a much lower rate. Eventually, terminal rutting (say 15 to 20 mm) will be reached without a noticeable acceleration of the rut rate towards the end of the pavement's theoretical life.

6.6.3 Regression Results

Regression analyses were undertaken on the rutting data according to the format described in the Section 6.6.2. A logistic model format was used during the analysis, since it provides a probability function that describes the likelihood of a discrete outcome, in this case, true or false. The regression results from the model using the CAPTIF data is listed in Table 6.7. It is noted from this table that the variables had a high significance in predicting the probability of accelerated rut progression. For this model, the probability of the rut rate exceeding a limit can be predicted.

Table 6.7: Accelerated Rut Rate Regression Results Obtained for the Logistic Model Based on the CAPTIF Rut Rate data.

	Estimate	Std. Error	z value	Pr(>|z|)	Significance
reps	3.78E-06	3.73E-07	10.145	< 2e-16	***
snp	-2.43E+00	2.91E-01	-8.362	< 2e-16	***
thickness.f150	4.43E+00	6.54E-01	6.771	1.28E-11	***
thickness.f300	4.74E-01	7.63E-01	0.621	0.534	

Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

Null deviance: 1369.66 on 988 degrees of freedom

Residual deviance: 635.93 on 984 degrees of freedom

AIC: 643.93

The resulting model is presented in Equation 6.7.

$$p(Rutaccel) = \frac{1}{1+e^{\left(-7.568*10^{-6}*ESA+2.434*snp-\left[\left(4.426,0.4744\right) for\ thickness=\left(0,1\right)\right]\right)}}$$

Equation 6.7

Where:

ESA	Equivalent Standard Axles
SNP	Pavement Structural Number
Thickness	0 for base layer thickness < 150mm, 1 for base layer thickness > 150mm

Note that the above model was converted to use ESA as a standard unit of measurement. The above model, Equation 6.7, is also graphically presented in Figure 6.21. The solid line represents the probability of a typical shallow pavement undergoing accelerated rutting for given cumulative traffic loading. The broken line represents the probability of thicker pavements that have the same theoretical structural number.

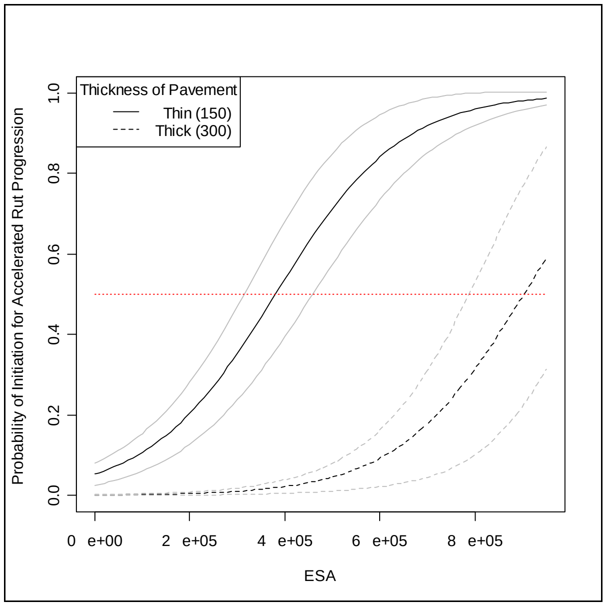


Figure 6.21: Final Logistic Model for Predicting the Initiation Point of Accelerated Rut Progression (SNP = 3)

6.6.4 Testing the Model on Network Data

As a limited number of LTPP sections displayed advanced deterioration, it was decided to test the accelerated rut model on some network data. The State Highway West Wanganui network was chosen, which represents a typical section of the New Zealand road network. This network consisted of a large proportion of thin/under-strength pavements. The network which was chosen mostly consists of shallow pavements and over recent years there has been a significant increase in heavy vehicles. Rutting has been recognised as an issue on this network.

Figure 6.22 illustrates the distribution of the predicted probability of accelerated rut rate to occur on this network. Note that only **thin pavements (<150mm)** were analysed. There is a significant part of the network showing relatively low probabilities of accelerated rut rate. This can be explained by either new pavements or lightly trafficked pavements. However, it is also observed that there is a significant part (approximately 1/6) of the total network showing probabilities higher than 0.8.

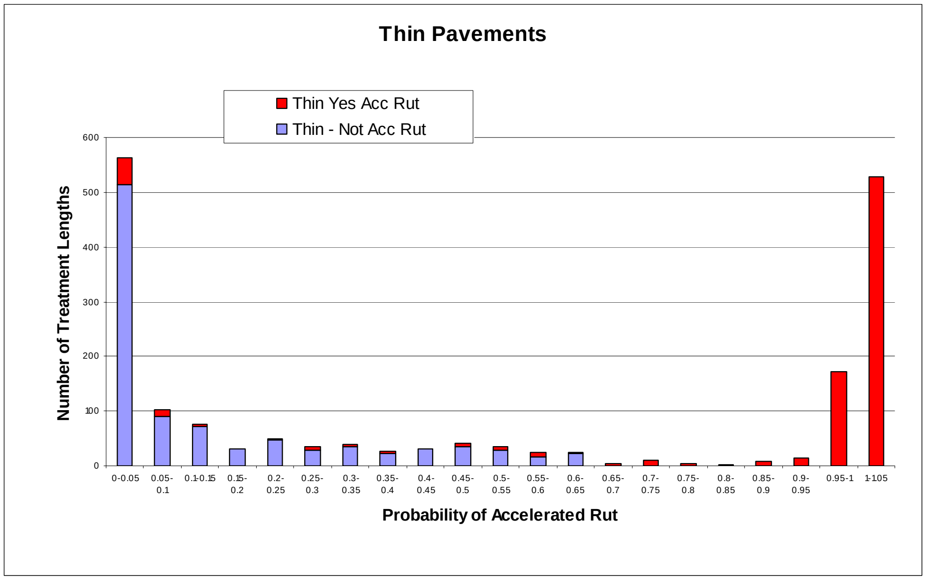


Figure 6.22: Comparing Predicted and Actual Accelerated Rut Rate on Network Level (Accelerated Rut Rate >1.5 mm/year)

Figure 6.22 shows where the actual rut rate exceeds the threshold (greater than 1.5mm) on the network. The 'blue' light bars indicate low or normal rut rates, and the darker 'red' bars indicate accelerated rut rate. For probabilities higher than 0.6, there was a 100% correspondence between the predicted and actual behaviour of the thin pavements. This observation provides confidence in the applicability and the robustness of the model, especially for pavements thinner that 150mm.

6.7 Rut Progression Summary

This chapter has described the development of a rut progression model for New Zealand roads. The most significant aspects are:

1. A new approach modelling the three distinct stages separately for rut progression according to:
    - A simple expression is proposed to model the 'initial densification'. This approach is similar to others published with the only difference that it incorporates variables that are readily recorded available in most Pavement Management Systems;

- A linear model is proposed to provide the rut progression rate for the stable progression stage; and,
- A logistic model is proposed for predicting the probability of a pavement to initiate accelerated rutting at any stage of its service life.

2. Given the age of pavements in the LTPP experiment, few have failed by 2006 when these analysis were undertaken. For that reason, alternative data sources had to be found in order to supplement the dataset for the modelling. Similar to work conducted in Australia (Martin, 2003), the extra data was obtained from an accelerated pavement testing programme. The introduction of the CAPTIF data had a successful outcome and it allowed the formulation of the model formats, which were then calibrated based on the limited LTPP data.

As with many research projects, the results and outcomes are limited by the available data. Using both the LTPP data and the CAPTIF data in the model development work gave excellent outcomes, but there were still some limitations in the data which prevented all the objectives being achieved.

In addition to this, the research work needs to continue in order to test and expand the applicability of the model developed. In particular, since the model formats were developed on the CAPTIF data, many variables that occur in the field could not be included. One example of this includes the ingress of water due to cracking or poor drainage. With the maturing of the LTPP data, more data during the failure processes will become available, and the model will have to be refined based on this data.

Further recommendations are:-

1. **Relative Performance of Different Material Types –** This research only included thin surfaced, unbound pavements. There is still some work to be completed in order to understand the difference in behaviour between different material types, such as dense graded and open graded porous asphalt pavements.
2. **Urban Environment –** Most of the data included in the current research represented pavements from the rural environment. With the data that became available on the

Land Transport New Zealand and Local Authority LTPP database, these models can now be tested for local urban pavements.

3. **Operational Research –** It should be acknowledged that the LTPP programmes delivered a wealth of data for research into practical aspects, such as data collection and maintenance practices. For example, the data can be used to validate some maintenance practices to address rutting. Similarly, there are a number of aspects that can be investigated regarding the collection of rutting data. These research areas should be encouraged, in order to get the full benefit from the data collected to date.

CHAPTER 7 THIS RESEARCH IN CONTEXT

7.1 Purpose of this Chapter

The work completed by the author for this research project has been the pioneer work for the LTPP establishment and data collection. For example, although this thesis only describes the State Highway LTPP establishment, the author has used the same concepts for establishing the local authority LTPP programme. In addition to that, the new modelling concepts introduced through this research also lead the way for other model development work to follow. Based on the experience of this and some other research projects, it is thus possible to value what this research has contributed to this topic area.

The purpose of this chapter is therefore to provide a high-level review of this research that was based on the State Highway LTPP programme and some of the initial findings of the new pavement deterioration models. It also provides strategic direction for future studies and regarding the focus of both the State Highway and Local Authority LTPP programmes.

7.2 LTPP Experimental Design

7.2.1 A Review of the Experimental Design

With the original experimental design, one of the main considerations was to obtain a representative cross section of most of New Zealand's road types and prevailing conditions. Chapter 3 illustrated the success in achieving this goal. In hindsight, three aspects as discussed in subsequent sections, could have been approached differently.

7.2.2 Climatic Regionalisation

The regionalisation technique discussed in Chapter 3 is valid and has been confirmed in the resulting cracking model. The result suggested that the low and limited sensitivity areas had noticeable slower crack initiation compared to the moderate and high sensitivity areas. Furthermore, it is believed that the combination of rainfall characteristics and moisture sensitivity of the subgrade is a practical and robust principle. The question remains as to the appropriate number of sensitivity regions for New Zealand.

Based on the findings in this research, it can be concluded that at least two sensitivity regions would be needed to explain pavement performance in New Zealand. The model results did not indicate a need for more than two sensitivity regions in order to explain climatic effects. This finding would be significantly different in other countries where diverse climatic and soil conditions exist such as the coastal and inland parts of Australia.

It is further believed that construction and maintenance practices may mask the impact of climate and subgrade sensitivity on the New Zealand roads. The investigation of these inter-effects fell outside the scope of this research and should be considered for future research.

7.2.3 Condition and Age

From the beginning of the research, it was anticipated that the pavement models would be of an incremental format similar to the HDM-4 models. For that reason, the original experimental design included an equal distribution between new to more deteriorated or older pavements. Having more pavements in the older category or in a poorer condition may have been more useful for the type of models resulted from this research. Rather than modelling the incremental step in deterioration, the new models placed larger emphasis on the timing of defect occurrence and/or the timing of pavement failure.

However, the current design matrix and the data obtained because of it, did, and may still in future achieve the following:

- **The data did confirm the exact behaviour of the pavements**. The theory of pavement behaviour and some accelerated pavement studies such as the Australian ALF (Martin, 2003) and the HVS from the South Africa (Theyse, 1996) show the classical three stages of pavement deterioration. To date modellers have struggled to reproduce this theoretical deterioration because of the inaccuracies in data collection. This research has confirmed that often the annual change in condition is far less compared to the expected accuracy of data measurement techniques. Secondly, the three-stage behaviour has been identified on the LTPP data, which allowed for the three-stage model for rutting. This was a significant finding that was only possible because of the range of age data available for the LTPP;

- **The lack of data for pavements in older or poorer category initiated this research using the CAPTIF data in conjunction with the LTPP data**. The origin of this idea was sourced from Martin (2003) who has developed relative performance ratios between the Australian LTPP and the accelerated programme (ALF). In this research, the LTPP and CAPTIF data sets were used in tandem to yield the new rutting model for New Zealand. The CAPTIF data was mostly used to develop the model format. This model was then refined or calibrated using the LTPP database;

- **There is a large portion of the database that is starting to produce data that represents advanced or deteriorated stages of pavement and surface lives**. There are some defects such as roughness that provided some unexplained intuitive trends. In future research, understanding these trends would only be possible when analysing an extended portion of the pavement's life. If mostly older and poor pavements had been selected in the original research, this would have limited future research application; and,

- **For a comprehensive understanding of pavement behaviour in relation to all variables, a section specific analysis is required**. This kind of analysis is only possible given this data set, which is available including a long-term condition history of most LTPP sections.

However, if all this information was available from the onset of the research, it is most likely that the same distribution of ages would have been chosen for the experiment.

7.2.4 Pavement Types

The majority of roads in New Zealand consist of thin flexible pavements sealed with a chip seal pavements. For that reason, the main objectives of the initial research were also focused on these pavements. The experimental design therefore, did not make a distinction between chip seal versus asphalt pavements, but rather split the sites into two categories of stronger and weaker pavements. For this classification strong pavement were classified as “unbound base with chip seal with total pavement deeper than 300mm or, (asphaltic surfaced pavements) or, Estimated SNP $\geq$ 3.

This research was undertaken largely on chip seal pavements hence there is a future need for a research focus shift towards Asphaltic pavements and even including some alternative technologies such as foam bitumen.

The recommended approach is to replace sites, which are lost due to rehabilitation to re-alignment with sections consisting of these materials. It is further recommended to maintain at least half the number of LTPP sections that will represent flexible chip seal pavements.

7.3 Data Collection

One of the major contributions of this research was the establishment of the LTPP monitoring programme that included the data collection of the LTPP sites according to a performance-based contract. Rather than specifying the equipment to be used for the surveys, and the accuracy tolerances, the repeatability and reproducibility of the measurements was specified.

Another key aspect of the data collection methodology was that the surveys had to be undertaken in a consistent manner. Therefore, the survey principles could only be changed with certainty on the impact of the change on the outcome of the data quality. This implied that in most cases changes were only allowed after intensive additional measurements confirmed the need for a process change plus the measurement outcome was well understood in terms of the long-term trend. For example, when an additional Walking Profilometer used on the surveys, a number of parallel surveys were undertaken before the one instrument was accepted to substitute the other.

The above-mentioned approaches have resulted in some highlights worth mentioning including:

- The tolerances on accuracy, repeatability and reproducibility were achieved, and resulted in extremely robust data. After eight surveys, there was no indication that the data was not accurate enough and some trends were observed that would not have been possible with traditional data collection processes used on other LTPP studies, such as HSD equipment. For example, Henning et al (2004b) has indicated that HSD type measurements yielded variances that are greater than the expected annual change in condition;
- The contract's quality assurance requirements insured that the data were validated and any erroneous data were identified and corrected; and,
- The surveys provided appropriate data of such quality that there is no need for the performance specifications to be changed.

There were however, some minor changes to the survey procedures, which are briefly summarised in Table 7.2

Table 7.1: A Summary of Some Changes Adopted with the Data Collection

Measurement	Proposed Change	Result/Impact
Rutting	The transversal starting position of measurements was kept at the edge of the carriageway edge line. Field observations suggested that some rutting extended beyond the edge line. New rutting measurement starting positions are shifted to include full extent of ruts.	Some individual sites have indicated an increase rut due to the shift. With parallel surveys on these sites, significant changes only due to the shift in measurement can be identified.
Roughness	For new pavements roughness measurement wheel tracks were	Parallel surveys indicated a difference in roughness development over time

	undertaken at predetermining positions and standard offsets (1.75m). After some years a few of these sections started to form a wheel track that was different from the assumed positions. It was proposed to change the roughness measurement wheel tracks to coincide with the actual wheel track.	between the assumed and actual wheel tracks. In addition, this difference is not constant over the years and some of these trends could not be explained. For that reason, these sites are still measured in both the assumed and actual wheel tracks.

7.4 New Pavement Prediction Models

7.4.1 Using an Continuous Probabilistic Model Format – Logit Models

7.4.1.1 Application

The use of probabilistic models in pavement engineering is not new. An earlier PMS application has used Stochastic or Marcovian modelling approaches to predict the change in the current condition state into a future condition state of a network (Rohde and Van Wijk, 1987). A common limitation of these types of models is that they were easier to apply to an entire network rather than individual road sections. In this regard, the deterministic and incremental deterministic models are of greater use.

The most widely adopted models of this kind are the World Bank HDM models. One of its strong characteristics is that these models were road section specific. Yet they have the limitation that they only predict the average condition outcome of the road section and little is understood regarding the distribution of the condition or defects.

This research has significantly contributed towards providing an alternative approach towards modelling pavement deterioration. It uses a Logit model form that is especially effective in predicting the likelihood of a defect event occurring, such as crack initiation and the initiation of accelerated rutting.

The use of statistical models capable of predicting the likelihood of probability of failure is more commonly used in other fields of engineering such as the modelling of mechanical or component failure. In his research, Watson (2005) used Bayesian models in order to predict the failure rate of water pipes. These models are especially effective as they use the historical performance of an entire network to predict future failure according to a limited number of independent variables. Therefore, in the study sample, both ‘failed’ and ‘un-failed’ data items are used in conjunction with some independent variables that influences failure rate.

The continuous probabilistic models such as the Logit models, provides a full distribution of failure likelihood or risk to failure. This provides much flexibility in the application of the model. For instance, it allows for:

- Variable intervention criteria based on different risk levels. For example, it is possible to intervene earlier on important arterials on a network by triggering treatments at lower risk levels;
- Probabilities or risk profiles is easier to understand, even by non-engineers. Therefore, different risk profiles could be communicated much more effectively as shown in Figure 7.1. This figure illustrates the decrease in lower probabilities levels (<15%) for crack initiation while the higher probability levels (45-60%) increases.

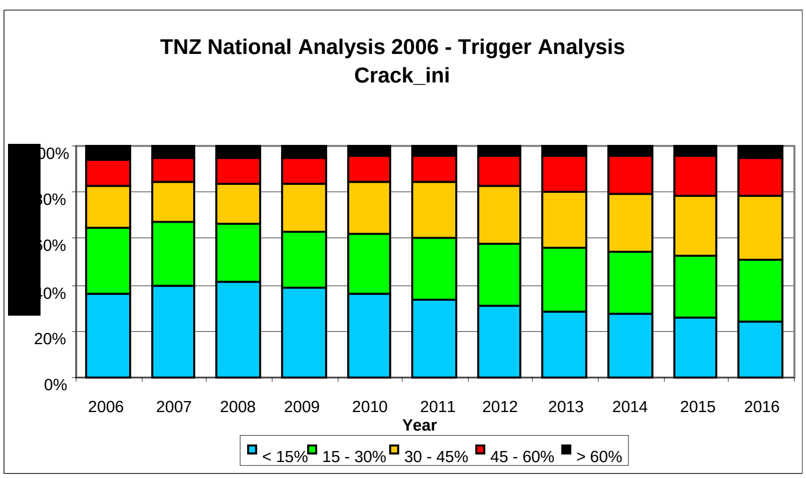
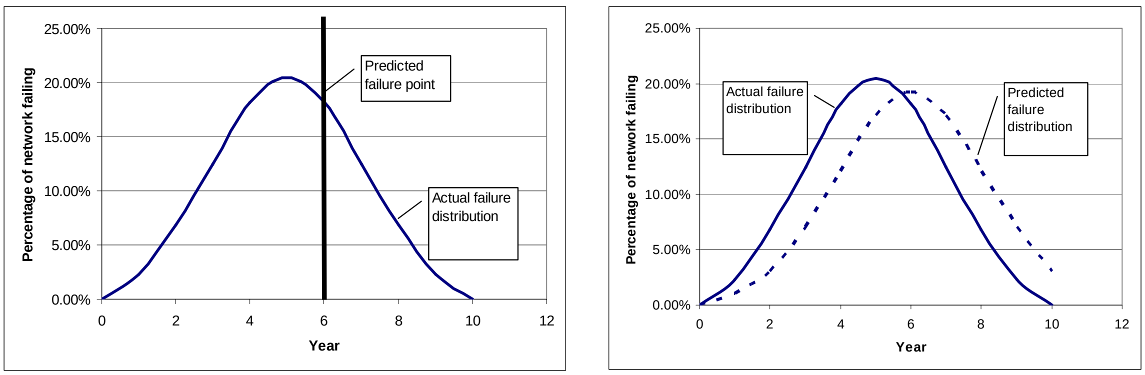


Figure 7.1: Probability of cracking due to decreased funding levels (Transit, 2007)

7.4.1.2 Accuracy of Models

The very nature of the Logit model will make it more accurate in terms of predicting failure behaviour for road sections. Figure 7.2 illustrate two predictions, the first predict a discrete failure point (left hand plot) and the second, a failure distribution (right hand plot). For both the predicted outcomes, the mean predicted failure was at year 6. In the case of the discrete predicted failure point there will be a large percentage of the network not failing at this point (there is only 18% of the network failing at year 6). The predicted failure distribution is also not matching the actual failure distribution perfectly. However, in this case a much larger portion of the network will have comparable failure probabilities.



Predicting a discrete failure point | Predicting a failure distribution

Figure 7.2: Comparing Predicted Failure versus Actual Behaviour

Both the crack initiation and rutting model have been tested on network data and remarkable good correlations were found. For example, an analysis of the East Wanganui State Highway network showed an 82% agreement between the predicted cracked status of the network compared to the actual cracked status (Henning, 2008).

The Logit model was also effective in predicting accelerated rutting for thin flexible pavements. It is recognised that that thick and stronger pavements do not necessarily fail through a rapid or accelerated rut progression. Rutting on these pavements occur over long periods until the terminal rut depth is reached according to the mechanistic design procedure.

7.4.2 Modelling Different Stages of Pavement Deterioration

This research has modelled rut progression in three distinct phases compared to the more traditional approach of modelling only two stages.

In this model development, the underlying principle was to align the model format and functionality to better represent the maintenance decision process. Pavement deterioration models are primarily used to predict future performance and indicate future maintenance needs. A better alignment with New Zealand maintenance practices was achieved with the introduction of the Logit models. For example, the majority of resurfacing programmes are strongly driven by the occurrence of cracking. Knowing when a pavement will crack is therefore more important to the maintenance engineer than knowing how much cracking exists. Likewise, rut depth is an important performance measure-reporting tool. However, when a pavement reaches the accelerated rut progression, it is normally maintained within a short time (sometimes within the same year). Therefore, knowing exactly when this accelerated rut commences is vital information for accurate maintenance predictions.

It was further recognised that a large portion of New Zealand roads consist of low volume roads typically carrying less than 1,000 vehicles a day. Pavement failure on these roads is typically more random than compared to stronger roads service higher traffic loadings. These thin flexible roads are therefore more prone to catastrophic failure due to minor over-loading and/or unusual high rainfall periods. Traditional deterioration models have not been effective on these types of networks. This research has provided useful models for this particular application since it makes the "risk to failure' approach possible. With these models, it is possible to investigate different traffic loading scenarios in order to identify high-risk pavements in terms of failure probability. This makes the identification of maintenance needs on these networks more accurate and related to risks, despite the fact that the current data does not always suggest these maintenance requirements. Hence, the models developed are therefore more useful in the maintenance decision process and the reporting of network status.

7.4.3 Model Limitations

In spite of the promising results obtained with the new models developed on this research, it should be realised that they were developed on a limited data set and further research is required to improve the applicability of the models to all pavement types and conditions. Specific limitations that will require further work are summarised in Table 7.2.

Table 7.2: A Summary of the Research Model Limitations and Recommendations

Model Limitation	Description	Recommendation
Empirical nature of models	Any empirical model has the limitation that it can only be applied with confidence to the sample area that it was developed for.	Given the dataset sampling, limited calibrations are required for New Zealand conditions. However, for all other areas outside of NZ, the models have to be used with care and recommended calibration processes have to be developed.
Completeness of the models	There were still some limitations to the data available. This was especially evident for the failure type data required for the rut model development.	With more data becoming available, the models can be refined. A focus area would be to investigate factors that engineers intuitively believe should be part of the model. An example is the interaction between cracking and accelerated rut progression.
Applicability of the models to all pavement/surface types	This research primarily focused on chip seal pavements. Consequently, the resulting models are not applicable to Asphalt pavements.	This research needs to be expanded to all Asphalt pavement types.
Testing failure models with actual failure data	All the models developed were tested at network level. Project level testing is required.	Test the models on individual LTPP sections that have failed (would contribute towards the understanding of failure development). Secondly, this information could be used to refine the existing models.

7.5 Past and Future Use of the LTPP Data from a National Perspective

7.5.1 This Research Contributing Towards a Wider Use of the Data

The LTPP establishment was the first main objective of this research. The aim of the LTPP establishment was primarily to provide appropriate data for the model development of this research. However, because of its quality, the LTPP data has been used for purposes much wider than just this research. Some of the most significant additional research outcomes included (Henning and Roux, 2007):

- The LTPP deflection data was used to develop a technique that counters the moisture dependency of FWD results. Salt and Stevens (2002) explored the use of the Moduli Ratio in order to provide a direct and effective measure of pavement quality that makes due allowance for site conditions and seasonal effects beyond a contractor's control;
- The LTPP data was further used in research to explore the measurement and understanding of pavement strength using alternative measurement techniques. In this research, Furlong, et al (2003) investigated the use of Seismic Analysis of Surface Waves (SASW) to determine the pavement strength and layer composition. SASW measures sound waves at different offsets and converts this electronic information into certain material and layer properties. The research demonstrated that SASW was able to provide a continuous picture of the elastic moduli of the pavement layers, and showed good comparison with existing pavement layer data from test pits. In addition, a specific modulus for the top base layer could be determined.

In addition to the above studies, there were also a number of smaller and operational research undertaken based on the LTPP Data. Henning and Roux (2007) listed additional research based on the LTPP data as depicted in Table 7.3.

Table 7.3: Practical Application of LTPP data

Application/Use	Description	Examples
Better Understanding of Condition Measurements	In most cases, condition measurements are a complex process, and there is still much to be learned about the measurement and the actual condition of the road that engineers want to quantify.	**Rutting** - Early work and the results contributed in proving the Transit HSD rutting measurements to underestimate the actual rut depth. This has led to a specific study during which the laser configuration was changed in order to measure a more representative rut depth **Roughness** – The LTPP data was one of the data sources used in order to develop best practice guidelines for surveys in the Urban environment. (Agrawal and Henning, 2005)
Benchmarking Network Surveys with LTPP data	Network HSD surveys do not always yield understandable trends and robust results. For that reason, Transit included the repeated measurement of the LTPP sections to the HSD contract. Based on the parallel data on these sections, benchmarking between the two datasets are possible.	In a study for Transit, Furlong and Henning (2005) have demonstrated the potential of benchmarking between LTPP data and network HSD data. This could then be used to validate any apparent bias in a specific year's survey. It can further assist in validating specific network trends.
Direct Use of LTPP Data for Equipment Calibration	Many local authorities have started to use LTPP data for their HSD equipment calibration.	The equipment calibration, based on LTPP data, will be broadly used across New Zealand. Based on the project for RIMS the LTPP data could become part of a national initiative to establish an accreditation system for HSD

Application/Use	Description	Examples
		suppliers in NZ.
Local Calibration of Pavement Deterioration Models	With the continued model development of PDM in NZ, LTPP data will still be used in order to calibrate models to local conditions.	

The success of the LTPP programme and the extended use of the data have resulted in both the State Highway and local authority LTPP's to extend the is monitoring programmes for another 5-10 years. These extension resulted in the objectives of the LTPP programme being reviewed to reflect future needs.

The objective of the on-going LTPP monitoring is to facilitate the better understanding of pavement behaviour under New Zealand conditions and practices. Specific goals related to this objective include (Henning and Roux, 2007):

- ***To increase understanding of road condition measurements and condition data****. Given the accuracy of measurements currently being undertaken, the LTPP data set lends itself towards being a benchmark for investigating most data measurements and statistical characterisation aspects;*
- ***To provide the required condition performance data for on-going pavement model development****. There are a number of models still to be developed and others that need further refinement work which is only possible with more data. For example, a number of new models predict failure probability. These models were developed based on limited data and more failed sections are required to finalise these models;*
- ***There must be an increased use of the LTPP data for operational purposes****. Many authorities are using the LTPP data on their networks for High-Speed Data (HSD) equipment calibration. This trend needs to be supported and widened to include a survey contractor accreditation process;*

* ***To use the LTPP programmes in research and development of new technologies such as pavement design and material use****. An effective link between the LTPP programme and the CAPTIF programme has been established. This demonstrated the potential of the LTPP programme to provide data for innovative and advanced research into these areas. This goal also highlight the importance of the LTPP monitoring to be flexible by including new technologies such as the use of foam bitumen;*
* ***With the availability of the data, higher-level industry goals could be supported****. Most prominent is to draw and develop new and existing practitioners into the asset management area.*

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The objectives of this research were to:

- establish a LTPP monitoring programme in order to produce pavement deterioration data that would be adequate for fundamental pavement model development; and,
- undertake development of a new modelling framework for application on the State Highway network. This framework was established on the basis of the two priority pavement models including crack initiation and rut progression.

Through this research, a successful LTPP monitoring programme has been established on the State Highway network.

From the analysis, it was concluded that the design matrix has sufficiently covered the spectrum of factors that influence road deterioration. Given the long-term nature of the monitoring programme, pavement failure information were limited during the initial stage of this research. As a result, LTPP analysis data were supplemented with network and accelerated pavement testing data. However, this LTPP long-term data is becoming more available given that the monitoring has been completed for eight years. The research also suggested that a more significant difference in performance was expected between the four chosen climatic regions. In this regard, the research was inconclusive and further work needs to confirm the exact number of climatic regions to stratify the pavement deterioration data.

The achieved data accuracy and repeatability that resulted from this research project's collection methodology is one of the highlights of this thesis. Based on the analysis completed in this research, it was concluded that the manual data collection methods adopted have achieved sufficient accuracy and repeatability levels that truly explains annual deterioration of road pavements. It was found that high-speed methods result in repeatability variations that well exceed annual changes in conditions.

A new modelling framework has been successfully established through this research.

Based on analyses completed, this research suggested that better predictive power is achievable from moving away from the traditional deterministic type models to more of a probabilistic approach. It was concluded that by recognising and modelling the variability in the initiation and failure points of pavement deterioration, a more realistic outcome can be achieved. Further strengthening of the predictive power is achieved through a probabilistic approach that uses the age factor of the pavement/seal as an independent variable instead of the predicted variable. In most cases, the age term is the most significant factor that influences pavement deterioration.

During this research, the Logit model format was chosen to forecast defect and failure initiation. It is a uni/multivariate method of predicting a probability of an event occurring or not. For example, it is capable of predicting probabilities of a specific section to be cracked 'or not' for a given year. One of the difficulties of evaluating the logit model is that the traditional R^2 test for 'goodness of fit' is not applicable. For that reason, the models developed were tested on independent network and LTPP data. From these analyses, it was concluded that the model had significant correlations (up to 75%) when comparing the predicted and test data.

This research also concluded that for defects with different deterioration stages, such as rutting progression, each deterioration stage should be modelled independently. Based on this research three rutting sub-models were proposed namely initial densification, stable progression and the probability of accelerated rutting. In splitting the model in the three respective stages, more realistic results were obtained compared to traditional methods that combine the latter stages in a exponential model format.

8.2 Recommendations – Models Developed

This thesis has documented the basis of new models developed for predicting crack initiation, and three rut progression sub-models. It has demonstrated how these models improve on the well-known HDM-III and HDM-4 models. It has also been tested on some State Highway network data to demonstrate its effectiveness and applicability in NZ. These models are depicted in Table 8.1 and .

Table 8.1: Crack Initiation Model Developed During this Research

Probability of Crack Initiation	$$\text{p(stat.aca)} = 1 \Big/ \left[1 + \exp\begin{pmatrix} -0.141\text{AGE2} + \{(5.062, 3.440)\ \textit{for}\ \text{stat.pca} = (0,1)\} \\ -0.455\text{Log(AADT)} - 0.275\text{Log(HTOT)} + 0.655\text{SNP} \end{pmatrix}\right]$$ Where p(stat.aca) is the probability of a section being cracked AGE2 is the surface age in years, since construction stat.PCAis the cracked status prior to 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

Table 8.2: Rut Progression Models Developed During this Research

Initial Rut Depth	$$Initial\_Rut = 3.5 + e^{(2.44 - 0.55SNP)}$$ Where: Initial Rut — is the rutting due to initial densification (mm) that will occur within the first year after construction. SNP — is the Modified Structural Number, calculated from FWD measurements.
Stable Rut Progression	**Thin pavements**: $$RPR = 9.94 - 1.38 \times a_1 SNP$$ **Thick pavements (>150mm)**: $$RPR = 14.2 - 3.86 \times a_2 SNP$$ Where: RPR = Stable rut progression rate in mm/million ESA; SNP = Modified structural number; a_1, a_2 = Model/Calibration coefficients
Probability of Accelerated Rutting	$$p(Rutaccel) = \frac{1}{1 + e^{(-7.568*10^{-6}*ESA + 2.434*snp - [(4.426, 0.4744)\ for\ thickness = (0,1)]}}$$ Where: ESA — Equivalent Standard Axles; SNP — Pavement Structural Number; Thickness — 0 for base layer thickness < 150mm, 1 for base layer thickness > 150mm

On the basis of the findings it is recommended to adopt these models in the NZdTIMS system. In addition it is also recommended that these models be calibrated to the different climatic areas to ensure they reflect local conditions.

8.3 Further Work

8.3.1 Recommendations to Address Limitations and Further Work

A summary of recommendations are listed in Table 8.3 and Table 8.4. These recommendations are split into two categories, namely recommendations that would address the limitations of this research, and secondly, recommendations for further work or future research.

Table 8.3: Addressing Limitations of Completed Research

Topic Area	Recommendation
Data Collection – Traffic Data.	This research has demonstrated that the pavement condition data is collected at appropriate accuracy and repeatable levels. The same cannot be said about the accuracy and robustness of the traffic data. The quality of the traffic data can be improved by: • Telemetry sites should be establish within the same traffic link as the LTPP section • WIM loading samples should be undertaken to supplement national traffic loading figures.
Current Model Make-up	Both the crack initiation and rutting models were developed based on limited data. Therefore, there are still some important factors not included to the current models. For example, cracking leads to water ingress into the pavement that contributes towards accelerated rut progression. With more data becoming available, these limitations must be addressed. Individual site analysis would be of particular interest to investigate some of these factors.
Empirical Nature of Models	All the models developed in this research are of an empirical nature. Therefore, its applicability outside of New Zealand is limited. A more fundamental-theoretical format of these models should be investigated for international application.

Table 8.4: Further Research and Monitoring

Topic Area	Recommendation
The future LTPP Programme Objectives.	The future LTPP Programme Objectives should address future research needs. the adjusted objectives for this programme are: • To increase understanding of road condition measurements and condition data; • To provide the required condition performance data for on-going pavement model development; • There must be an increased use of the LTPP data for operational purposes; • To use the LTPP programmes in research and development of new technologies such as pavement design and material use; and, • With the availability of the data, higher-level industry goals could be supported – such as widening the research pool for pavement performance type research.
Establishment of future LTPP Section.	With the replacements of current redundant LTPP sections or with the establishment of new LTPP sections, the focus should be placed on addressing data needs for future research. However, at least 50% of the sites should remain for the original design matrix.
Further Model Development	Further development of models should be undertaken according to national priorities – see Figure 8.1. According to this figure the priority models to be developed include: Flushing and Shoving. Following these, the next model development includes roughness and crack growth. It should be noted that the priorities of these developments are based on how much each individual performance measure is used as a driver, in the maintenance decision process.

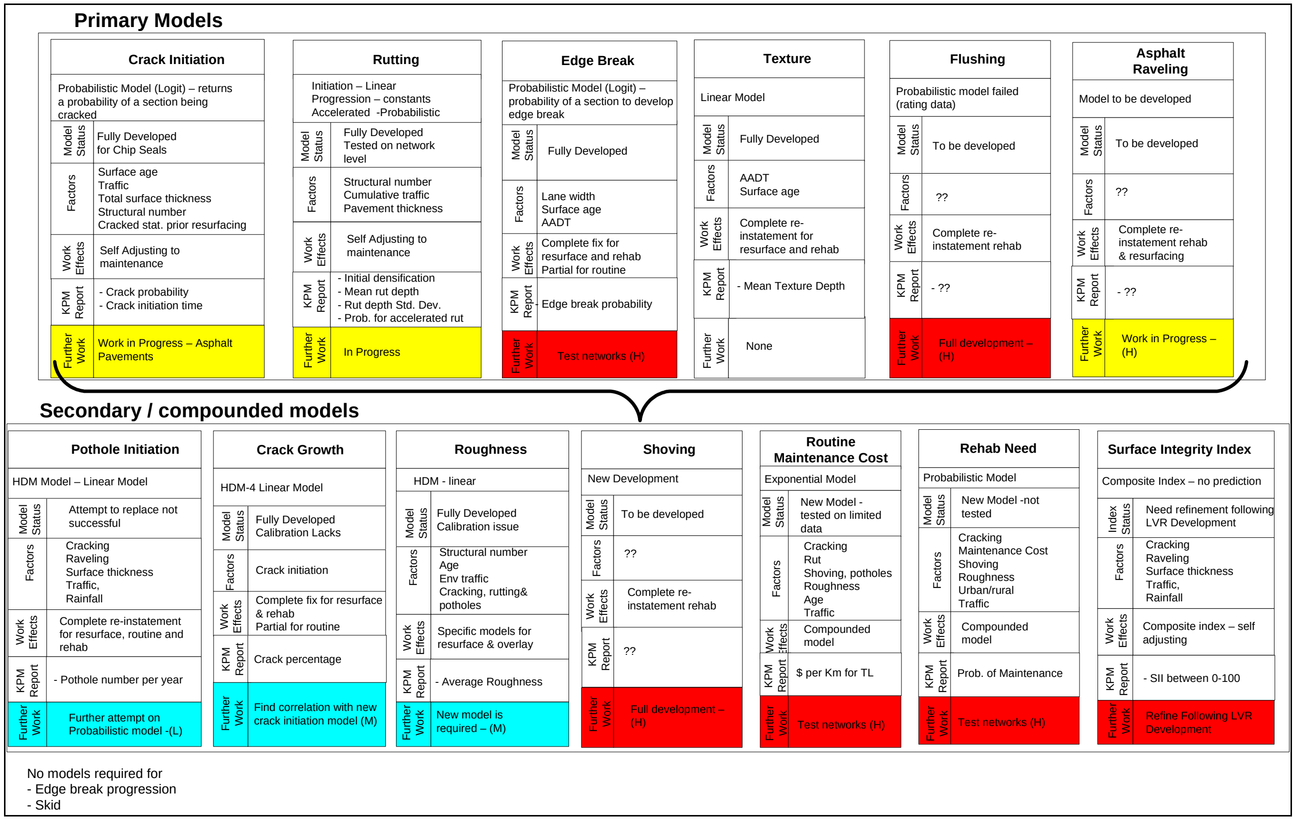


Figure 8.1: New Zealand Model Development Status and Priorities

Legend – Priorities: Red (High), Yellow (In progress), Blue (Low and medium priorities)

8.4 Lessons Learnt from this Research

This research has significantly contributed towards the understanding of pavement deterioration monitoring and modelling in New Zealand. Of the most significant lessons learned and re-confirmed based on the findings of this research include:

- For the purpose of model development, **condition monitoring** should be undertaken using manual measurement techniques. Outcomes from this research have suggested that variances in data due to the repeatability of high-speed data (HSD) measurements exceeds the incremental condition change of pavements;
- **Defining the data accuracy and repeatability tolerances** ensures the appropriate data quality is achieved;
- Data **measurement conventions** such as location referencing should not be changed during the monitoring programme – only with sufficient evidence of parallel surveys should there be changes in the methodology;
- The **distribution of the condition/age in the design matrix** should be carefully considered based on the desired model types to be developed;
- The **climatic effects across New Zealand** were less significant than expected. Instead of four climatic regions, only two different regions were required;
- The **development of model priorities** should be focused on the significance of the performance measure related to its ability to drive maintenance decisions;
- If different stages can be identified for the deterioration of a pavement, the models **predicting the deterioration should mirror these stages**. This results in simple, more robust models.

Reference List

AASHO, (1962). *AASHO Road Test.* Report 7, Summary Report, Special Report 61G. Highway Research Board, Washington, D.C.

AASTHO, (1986). *Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Official. Washington, D.C.

Agrawal, V. and Henning, T.F.P. (2005) Study of High-Speed Roughness Data on Urban Road Networks, Final Report for Land Transport New Zealand and Participating Councils Research Project 801/007773, MWH New Zealand Ltd

Alabaster, D. and Fussell, A (2006). *Fatigue Design Criteria for Low Noise Surfacings*. Land Transport New Zealand Research Report Number 307. Land Transport New Zealand, Wellington.

AUSTROADS, (1992). Pavement Design: A Guide to the Structural Design of Road Pavements. Austroads National Office, Haymarket, Australia.

Arnold, G. (1998). *Engineered Pavemenets.* Roading Geotechnics '98. New Zealand Geotechnical Society Symposium. The Institute of Professional Engineers, Wellington.

ASTM E867 (1987) *Terminology Relating to Vehicle-Pavement Systems.* ASTM International. West Conshohocken, USA.

Bennett, C.R., Chamorro, A., Chen, C., De Solminihac, H., Flintsch, G.W. (2006) *Data Collection Technologies for Road Management.* East Asia Pacific Transport Unit, The World Bank, Washington, D.C.

Bennett, C.R. and Paterson, W.D.O. (2000) *A Guide to the Calibration and Adaptation.* HDM - 4. Volume 5. The Highway Development and Management Series. PIARC

Bennett, C.R. (2004) *Sectioning of Road Data for Pavement Management*. 6th International Conference for Managing Pavements, Brisbane, Australia.

Benson, P. (1991) *Biases and Gaps in the GPS database.* Paper delivered at the Proceedings of SHRP midcourse Assessment, Guide No. SHRP-LTPP-UWP-91-514. Strategic Highway Research Programmes, National Research. USA

Brown, D.N. and Fong, S. *Validation and Repeatability Of Reference Measurements Used for Evaluating High Speed Roughness Data*. Report No. 01-261496.00-801CL. Opus Central Laboratories, Opus International Consultants Limited, Lower Hutt, New Zealand.

Cenek, P. (2001). Summary Report- Regionalisation of Roading Authorities for Pavement Deterioration Modelling. Memorandum to Ken Hudson, Opus Central Laboratories, Lower Hutt, NZ.

Chambers, J.M. and Hastie, T.J. 1992. *Statistical Model in S.* Chapman and Hall: London, UK

Crow (2003). A Literature *Survey on Tire Surface Friction on Wet Pavements Application of Surface Friction Testers.* Report 03-06, CROW, The Netherlands. Available for download from http://www.crow.nl/engels/news/Content/report_03-06.html.

CSIR, (1997). *A Short Course on: Rehabilitation Design of Felxible Pavements in South Africa.* RR 93/269. Devision of Roads and Transport, Technology, CSIR Pretoria

De Corte, W. and De Winne, E. (200) *Pavement Management System through Europe: Similarities and Differences.* 1st European Pavement Management Systems. Budapest, Hungary.

Dossey, T. and Hudson, W.R. (1994). Distress as Function of Age in Continuously Reinforced Concrete Pavements: Models Developed for Texas Pavement Management Information Systems. Transportation Research Record No. 1455. Pavement Management Systems. Transportation Research Board, Washington, D.C.

FHWA, (2000). *LTPP Product Plan.* Publication No. FHA-RD-01-086. US Department of Transportation. Federal Highway Administration. USA.

Furlong, W., Henning, T.F.P., White, H., De Beer, W. (2003) SASW – An Alternative Method to Determine Layer Thickness and Pavement Strength. Land Transport New Zealand Research Report Number 245. Land Transport New Zealand, Wellington.

Furlong, W. and Henning, T.F.P. (2005) *Benchmarking Transit High Speed Surveys with Calibration Surveys*. Transit New Zealand, Welligton, NZ.

GEIPOT (Ministerio Dos Transportes), UNDP (United Nations Development Programme), (1981) *Research on Interrelationships Between Cost of Highway Construction, Maintenance and Utilisation.* Final Report. Volume 2 – Methods and Organization. Brasilia

Haas, R. (1998). *Pavement Management: A Great Past but What about the Future?* 4th International Conference for Managing Pavements, Durban, RSA

Hatcher, W. (2007) *New Rutting Progression Model*. Internal Memorandum describing the outcome of testing the new rutting model for NZ adoption. Opus International Consultants. Napier.

Hatcher, W., Ul-Islam, R., Henning, T.F.P. (2007). *Development of RIMS dTIMS CT Pavement Performance Model 2007* - Enhancements based on inputs from Low Volume Roads study, Pavement Model Developments and User requirements. RIMS Group, Wellington.

Hall, K.T., Lee, Y., Darter, I., Lippert, D.L. (1994) *Forecasting Pavement Rehabilitation Needs for Illinois InterState Highway Systems.* Transportation Research Record No. 1455. Pavement Management Systems. Transportation Research Board, Washington, D.C.

Henning, T.F.P. (2001a) *A National Training and Dissemination Strategy for HDM-4 in Australia.* Report for AUSTROADS Project (BS.A.N.014). Transit New Zealand. Asset Management Team Wellington, New Zealand.

Henning, T.F.P. (2001b). *Transit New Zealand Calibration Sections Survey Contract* Number TNZ SM3-688/P6/10. Transit New Zealand, Napier

Henning, (2008). An Analysis of the East Wanganui State Highway Network. Unpublished Work.

Henning, T.F.P., Dunn, R.C.M., Costello, S.B., Hart, G., Parkman, C.C., Burgess G. (2004a) *Long-Term Pavement Performance (LTPP) Studies in New Zealand – Lessons, the Challenges and the Way Ahead.* 6th International Conference for Managing Pavements, Brisbane, Australia.

Henning, T.F.P., Dunn, R.C.M., Costello, S.B., Parkman, C.C. (2007) *A New Approach for Modelling Rutting on the New Zealand State Highways.* Accepted for publication in 2009 by ARRB Journal, Road & Transport Research, Australia.

Henning, T.F.P., Costello, S.B., Dunn, R.C.M., Parkman, C.C., Hart, G. (June 2004b) *The Establishment of a Long-Term Pavement Performance Study on The New Zealand State Highway Network*. ARRB Journal, Vol 13 No 2, Road & Transport Research, Australia.

Henning, T.F.P., Costello, S.B., Parkman, C.C., Bennett, C.R., Dunn, R.C.M. (2006) A Complete Review of the Crack Initiation Models for New Zealand. 22nd ARRB Conference – Research into Practice, Canberra Australia.

Henning, T.F.P., Costello, S.B., Watson, T.G. (2006). *A Review of the HDM/dTIMS Pavement Models Based on Calibration Site Data*. Land Transport New Zealand Research Report Number 907. Land Transport New Zealand, Wellington.

Henning, T.F.P., Furlong, W. (2005). *Benchmarking Transit High Speed Surveys with Calibration Surveys*. Transit New Zealand, Wellington.

Henning, T.F.P. and Hockley, S.R. (2003), dTIMS Calibration Sections for Local Authorities - Specification for LTPP Site Establishment. Transfund New Zealand, Christchurch, New Zealand.

Henning, T.F.P. and Roux D.C. (2007). Status Report - The New Zealand Long-Term Pavement Performance Programme. Land Transport New Zealand. Christchurch.

Henning, T.F.P. and Riley,M. (2000) Calibration Guidelines for Pavement Deterioration Models and Work Effects Models in dTIMS. HTC Infrastructure Management. Auckland.

Henning, T.F.P., Mahoney, J.P., Sadzik, E.S., Jackson, N.C. (1998). *Comparison of Pavement Management Systems in the Republic of South Africa and the United States of America*. Fourth International Conference on Managing Pavements. ICC Durban, ISBN: 0-620-22377-4, Volume

Henning, T.F.P, and Tapper, M. (1994). *Calibration of Pavement Deterioration Models Based on the 2003 Data*. The Asset Management Team. Transit New Zealand. Wellington, NZ

Henning, T.F.P. (1998). *The influence of Resealing on the Performance Models of Pavements in Gauteng*. Project towards the degree of M.Eng (Transportation). University of Pretoria, South Africa.

Henning, T.F.P., Roux D.C., Alabaster, D. (2007) *Benchmarking Pavement Performance Between Transit's LTPP and CAPTIF Programs*. Land Transport New Zealand Research Report Number 329. Land Transport New Zealand, Wellington.

Henning, T.F.P., Visser, A.T., Sadzik, E.S. (1999). *An Attempt to Calibrate the HDM-3 Prediction Models on the Basis of PMS Data*. 7th Conference on Asphalt Pavements for Southern Africa. Victoria Falls Zimbabwe.

Hewitt, A.E. 1998. New Zealand soil classification. Landcare Research Science Series No.1. Mannaki Whenua Press, Auckland.

HTC Infrastructure Management. 1999. dTIMS Technical Reference Manual Volume I. HTC. Auckland, New Zealand

Infrastructure Management: Auckland, New Zealand.

Jordaan, G.J. (1984). *Bituminous Pavement rehabilitation Design*. M.Eng (Civil) Thesis. University of Pretoria. Pretoria, South Africa.

Kadar, P., Henning, T.F.P., Parkman, C. C. (2006) *Effectiveness of Performance Indicators in Managing Road Networks*, 22nd ARRB Conference – Research into Practice, Canberra Australia.

Karamihas, M. (2004). *The 2004 FHWA Profiler Round-Up*. Presentation on the results of the 2004 Profiler Comparison Project. University of Michigan Transportation

Research Institute. Available for download from www.umtri.umich.edu/erd/roughness.

Lay, M.G. (1998). *Handbook of Road Technology*. Volume 1. Sinclair Knight Merz Ltd. Melbourne, Australia.

Lytton, R.L. (1987). *Concepts of Pavement Performance Prediction and Modelling.* Proceedings of the Second North American Conference on Managing Pavements. Toronto.

Mallela, R. and Wang, H. (2006). *Harmonising Automated Rut Depth Measurements – Stage 2*. Land Transport New Zealand Research Report 277. 58pp, Obtainable from (http://www.landtransport.govt.nz/research/reports/index.html)

Martin, T. (1994). *Pavement Behaviour Prediction for Life-cycle Costing.* Research Report ARR No. 255. Australian Road Research Board Ltd. Victoria, Australia.

Martin, T. (2003). Pavement Performance Prediction Deterioration Model Development- Data Review and Calibration of HDM-4 Road Deterioration Models . Research Report RC 2051. Australian Road Research Board Ltd. Victoria, Australia.

NDLI, (1995). Modelling Road Deterioration and Maintenance Effects in HDM-4. Final Report ADB BETA 5549, Vancouver, B.C.

Nordic Road & Transport Research. (1999). *Influence of road surface texture on traffic characteristics* Swedish National Road and Transport Research Institute, Available for download from http://www.vti.se/Nordic/2-99mapp/299sw.html.

Paterson, W.D.O. and Scullion, T. (1990). *Information Systems for Road Management: Draft Guidelines on System Design and Data Issues.* World Bank Technical Paper INU 77. Infrastructure and Urban Development Department, World Bank, Washington, D.C.

Paige-Green, P. (1989). The influence of Geotechnical Properties on the Performance of Gravel Wearing Course Materials. Unpublished Ph.D. Thesis, University of Pretoria, South Africa.

Pradhan, N. (2001). Evaluating the Sensitivity of Parameters in Predictive Pavement Deterioration Modelling. Transfund New Zealand, Research Report 212.

Prebble, W. M. (1998). *Landslides and Roading: Examples from Auckland, the Taupo Volcanic Zone, Rangitikei Valley and the Southern Alps.* Roading Geotechnics '98. New Zealand Geotechnical Society Symposium. The Institute of Professional Engineers, Wellington.

Road Information Management Steering Group –RIMS, (2007). NZ dTIMS CT Training Volume. Report Number DT/2007/01. Wellington.

Robinson, R., Danielson, U., Snaith, M. (1998). *Road Maintenance Management - Concepts and Systems*. Palgrave Publishers Ltd. The University of Birmingham and the Swedish National Road Administration.

Rohde, G.T. (1994). *Determining Pavement Structural Number from FWD Testing*. Transportation Research Record 1448, Transportation Research Board, Washington DC, USA.

Rohde, G.T., Jooste, F, Sadzik, E.S, Henning, T.F.P. (1998). "*The Calibration and Use of HDM-4 Performance Models in a Pavement Management System*". 4th International Conference on Managing Pavements. ICC Durban, ISBN: 0-620-22378-2.

Rohde, G.T., Pinard, M.I., Sadzik, E.S. (1996) *Long-Term Network Performance- a Function of PMS Maintenance Selection Policy.* Paper presented at the Transport Research Board Annual Meeting. Washington, D.C.

Rohde, G.T. and Van Wijk, A.J. (1987) A probabilistic Approach to Predict Pavement Performance and to Optimise Maintenance and Rehabilitation Strategies. Africon, Pretoria, South Africa.

Rohde, G.T., Wolmerans, I., Sadzik, E.S. (2002) *The calibration and Validation of HDM Performance Models in the Gauteng PMS.* 21st Annual South African Transport Conference, CSIR, South Africa.

Rolt, J. and Parkman, C.C. (2000). *Characterisation of Pavement Strength in HDM-III and Changes Adopted for HDM-4*. Proc. 10th Road Engineering Association of Asia and Australasia, Tokyo, Japan.

Salt,G. (1999). Determining the Structural Capacity of Unbound Granular Pavements in New Zealand Using Deflection Testing. Report for Transfund New Zealand,. Tonkin and Taylor Ltd, New Zealand.

Salt, S. and Stevens, D. (2002) Performance-Based Specifications Using Deflection Measurements Modular Ratios for Unbound Granular Pavement Layers – Their Basis and Use as Performance Indicators for New Pavement Construction Assurance, and Maintenance Contracts. Transfund Project PR 3 0511, Tonkin & Taylor Ltd, Pavements Group, Land Transport New Zealand, Wellington.

Sayers, A.W. and Karamihas, S.M. (1996). *Measuring and Analyzing Road Profiles.* Short Course. The University of Michigan Transportation Research Institute. USA.

Sayers, A.W. and Karamihas, S.M. (1998). *The Little Book of Profiling- Basic Information about Measuring and Interpreting Road Profiles*. University of Michigan Transportation Research Institute. USA

SHRP-LTPP, (1990). *Guide for Field Materials Sampling, Handling and Testing.* Operational Guide No. SHRP-LTPP-OM-006. Strategic Highway Research Programmes, National Research. USA

SHRP-LTPP, (1993). *Interim Guide for Laboratory Material Handling and Testing.* Strategic Highway Research Programmes, National Research. USA

SHRP-LTPP, (1999a). *Strategic Plan for Long-Term Pavement Performance Data Analysis.* Strategic Highway Research Programmes, National Research Council, Revised April 2001. USA

SHRP-LTPP, (1999b). *Introduction to LTPP Data.* Strategic Highway Research Programmes, National Research Council, USA

Simpson, A.L. (2001). *Characterization of Transverse Profiles*. FHWA-RD-01-024. Federal Highway Administration. National Technical Information Service, Springfield, Virginia

South African Department of Transport. (1997) *A Short Course on: Rehabilitation Design of Flexible Pavements in South Africa.* Course Material of Transportek of the CSIR PR93/269. CSIR, South Africa.

Steven, B.D., de Pont, J.J., Pidwerbesky, B.D., Arnold, G. (1999) A*ccelerated Dynamic Loading of Flexible Pavements at CAPTIF.* International Conference on Accelerated

Tepper, S. and Martin. T. (1999) *Long-Term Pavement Performance Maintenance (LTPPM) – Progress Report.* Report RC90268-1. for Austroads. ARRB Transportation Research Ltd. Melbourne, Australia.

Theyse, H.L., De Beer, M., Rust, F.C. (1996). *Overview of the South African Mechanistic Pavement Design Method*. Transport Research Record No. 1539. Transportation Research Board, National Research Council, Washington, D.C.

Thornthwaite, C. W. (1948). An approach towards a rational classification of climate, Geographical Rev.38. Australia.

Transfund New Zealand. (1997) *Manual of Competitive Pricing Procedures*. Volume 1: Physical works and professional services. Second revision, Transfund New Zealand, Wellington, New Zealand.

Transfund New Zealand. (2004) Project Evaluation Manual PFM2 Transfund New Zealand, Wellington, New Zealand

Transit, (2001). *Transit New Zealand Calibration Sections Survey* Contract Number TNZ SM3-688/P6/10. Transit New Zealand, Napier.

Transit, (2000). *2000/01 dTIMS National Report.* Asset Management Team Transit New Zealand, Wellington.

Transit New Zealand, (2005). *State Highway National Pavement Condition Report.* Transit New Zealand. Wellington.

Transit, (2007). *2006/07 dTIMS National Report.* Asset Management Team Transit New Zealand, Wellington.

Van As, S.C. and Joubert, H.S. *Applied Statistics for Civil Engineers*. Department of Civil Engineering, University of Pretoria, Pretoria, South Africa.

Van Zyl, G.D. (1994), *Visual Assessment for the Calibration of HDM-III.* Report prepared fon behalf of Africon for the Transvaal Roads Branch, Report DPVTC/283, CSIR, South Africa.

Vissser, A.T. (1999). "Siviele Ingenieursmateriale en Plaveiselontwerp." (Civil Engineering Materials and Pavement Design). Civil Engineering Course SGM 220, University of Pretoria, South Africa

Wang, K.C.P., Way, G., Delton, J. (1993). *Pavement Network Optimizarion and Implementation.* Special Report. Arizona Department of Transportation. Phoenix, Arizona, USA.

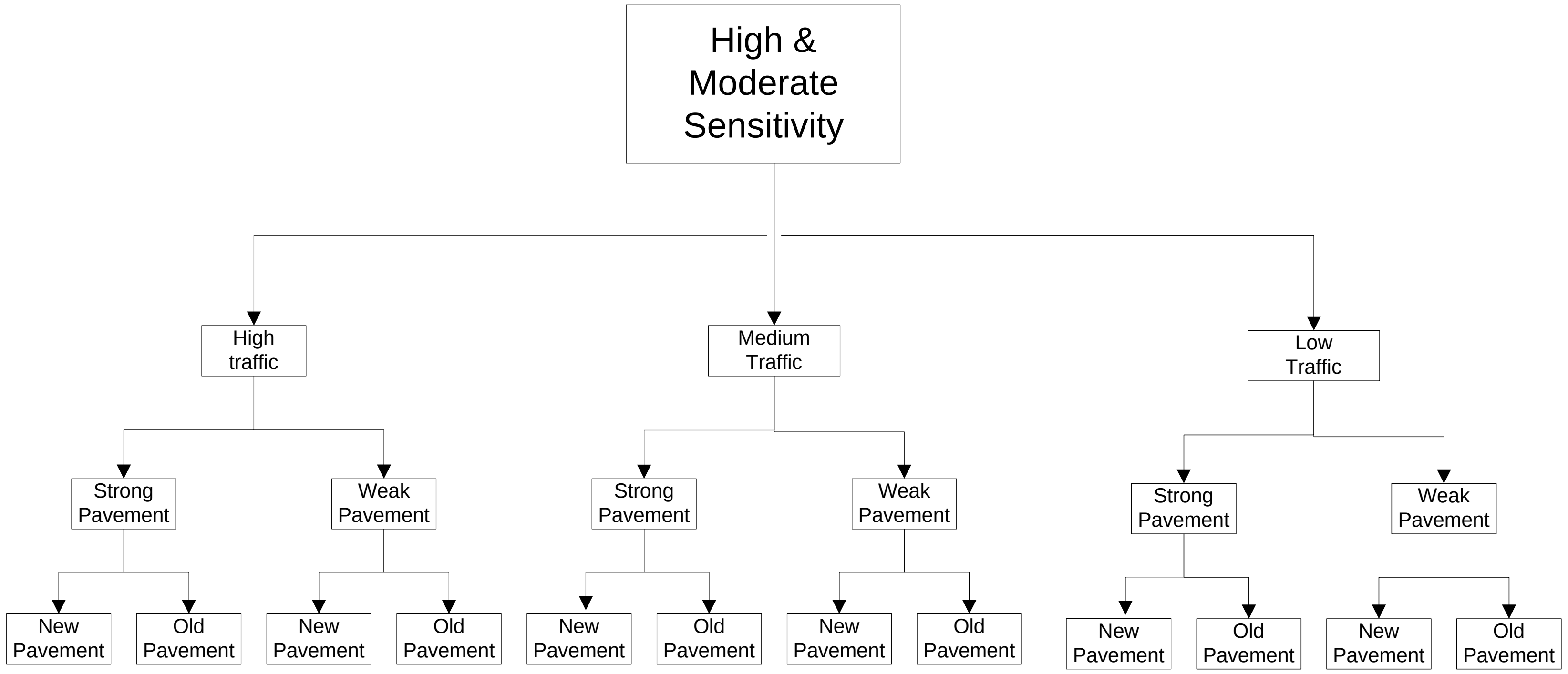
Watanatada, T., Harral, C.G., Paterson, W.D.O., Dhareshwar, A.M., Bhandari, A. Tsunokawa, K. (1987) *The Highway Design and Maintenance Standards Model.* World Bank. The John Hopkins University Press. Baltimore, UK.

Watson, T.G. (2005). *A hierarchical Bayesian model and simulation software for the maintenance of water pipe networks.* Thesis PhD-Civil and Environmental Engineering, University of Auckland, Auckland.

Weinert, H.H. (1974). A climatic Index of Weathering and its Application in Road Construction. Geotechnique, Vol. 24, Pretoria, SA.

Weinert, H.H. (1980). *The Natural Road Construction Materials of South Africa.* National Institute for Transport and Road Research. Pretoria, SA.

APPENDIX A: RESULTING DESIGN MATRIX FOR LTPP SECTIONS



	High traffic / Strong Pavement / New Pavement	High traffic / Strong Pavement / Old Pavement	High traffic / Weak Pavement / New Pavement	High traffic / Weak Pavement / Old Pavement	Medium Traffic / Strong Pavement / New Pavement	Medium Traffic / Strong Pavement / Old Pavement	Medium Traffic / Weak Pavement / New Pavement	Medium Traffic / Weak Pavement / Old Pavement	Low Traffic / Strong Pavement / New Pavement	Low Traffic / Strong Pavement / Old Pavement	Low Traffic / Weak Pavement / New Pavement	Low Traffic / Weak Pavement / Old Pavement
Northland	~~CS 1~~ CS 2	CS 3			**New Site**						CAL 4, CAL5 SNP?, CAL 6 SNP?	
Central Waikato			CS 11		CS 13a, CAL 13b		CAL 12					
West Waikato	CAL 17		CS 16									
Gisborne									CS 22			CS 21
West Coast					CAL 45a, CS 45b						CS 46	
Coastal Otago						CAL 48, CS 49				~~CS 50~~, CAL 51 **New Site**		

Low & Limited Sensitivity

	High traffic				Medium Traffic				Low Traffic			
	Strong Pavement		Weak Pavement		Strong Pavement		Weak Pavement		Strong Pavement		Weak Pavement	
	New Pavement	Old Pavement	New Pavement	Old Pavement	New Pavement	Old Pavement	New Pavement	Old Pavement	New Pavement	Old Pavement	New Pavement	Old Pavement
Auckland		CS 7a,CS 7b, CS 8a, CS 8b										
East Waikato					CS 14				CS 14			
PSMC	CAL 19				CAL 18							
Gisborne									CAl 23			
Napier						CS 26	CAL 25a, CAL 25b, CAL 27a, CAL 27b		CS 24			
Wanganui					~~CS 28~~,		**Replacement Site**		CAL 30			CS29
Manawatu		CS 31			CAL 32							
Taranaki	CS 33, SNP?						CAL 34					
Wellington		CAL 35					CS 36		CS 20			
Marlborough						CAL 38						CAL 37a, CS 37b
Nelson					CS 39					CS 40		
North Canterbury	CAl 41								CS 42, SNP?			
South Canterbury						CAL 43			CS 44			
Coastal Otago												CAL 52a, CS 52b