Rationalisation of the structural capacity definition and quantification of roads based on falling weight deflectometer tests

May 2010

G. Salt
Tonkin & Taylor Ltd

T.F.P. Henning
University of Auckland

D. Stevens
Tonkin & Taylor Ltd

D.C. Roux
MWH New Zealand Ltd

NZ Transport Agency research report 401

ISBN 978-0478-36408-8 (print)
ISBN 978-0478-36409-5 (electronic)
ISSN 1173 3756 (print)
ISSN 1173-3764 (electronic)

NZ Transport Agency
Private Bag 6995, Wellington 6141, New Zealand
Telephone 64 4 894 5400; facsimile 64 4 894 6100
research@nzta.govt.nz
www.nzta.govt.nz

Salt, G[1], TFP Henning[2], D Stevens[1] and DC Roux[3] (2010) Rationalisation of the structural capacity definition and quantification of roads based on falling weight deflectometer tests. *NZ Transport Agency research report no.401*. 56pp.

[1] Tonkin & Taylor Ltd, www.tonkin.co.nz
[2] Department of Civil and Environmental Engineering, University of Auckland, Private Bag 92019, Auckland 1142
[3] MWH New Zealand Ltd, L 2, Bldg C, Millennium Centre, 600 Great Sth Rd, Greenlane, Auckland

Keywords: Falling weight deflectometer (FWD), pavement life, pavement performance, pavement strength, SNP, structural capacity, structural indices, structural numbers

An important note for the reader

Acknowledgements

The project team appreciate the contributions and input from the peer reviewers:

Mr David Alabaster NZTA

Dr Peter Kadar, ARRB Group Ltd

Mr Jim McQueen, Dunedin City Council

Abbreviations and acronyms

AADT	Annual average daily traffic volume
AASHO	American Association of State Highway Officials (until Dec 1973)
AASHTO	American Association of State Highway & Transportation Officials (1974 on)
AC	Asphaltic concrete
ADT	Average daily traffic
APT	Accelerated pavement testing
CAPTIF	Canterbury Accelerated Pavement Testing Facility
dTIMS	Deighton Total Infrastructure Management System
ESA	Equivalent single axle
HDM	Highway design and maintenance model (World Bank)
KPM	Key performance measure
Land Transport NZ	NZTA (from 2004)
LTPP	Long-term pavement performance
MESA	Millions of equivalent standard axles
NAASRA	National Association of Australian State Roading Authorities
NCHRP	National Cooperative Highway Research Program
OGPA	Open-graded porous asphalt
PMS	Pavement management systems
RAMM	Road assessment and maintenance management system
SDC	Southland District Council
SI	Structural index
SIs	Structural indices
SNP	Adjusted structural number
Transit	Transit New Zealand
Transfund	Transfund New Zealand (to 2004)
VSD	Vertical surface deformation

Contents

Executive summary

Structural number concepts originated well before mechanistic analysis procedures became readily available to practitioners. The reason the adjusted structural number (SNP) can give an approximate indication of possible structural deterioration for a large network is that the progression of many distress modes will generally be deferred by improved load spreading (subgrade strain distribution). However, most of the techniques used to model this are based on a general indicator of strength that is derived from layer thickness and material quality. Therefore, SNP is not able to give any indication of how a particular pavement structure would behave for a given layer configuration. For example, a road consisting of a stabilised base on top of inferior material may have a high SNP, but would in fact fail rather quickly due to cracking of the base layer.

Mechanistic appreciation of pavement structural performance, which is the aim of the US National Cooperative Highway Research Program (NCHRP), is not yet at the stage where reliable models for progression of all distress modes in all materials are available. Advances in that research should be followed closely, as it should eventually lead to the most effective procedures for rational design. Meanwhile an improvement to the indirect SNP concept is required. An interim solution for practitioners is to use mechanistic procedures when deriving the fundamental structural parameters for network modelling.

As a replacement for SNP, an alternative structural parameter termed structural index has been proposed. For each of the currently recognised structural distress modes (ie rutting, roughness, flexure and shear) a corresponding structural index is required. This study provides the basis for structural indices for rutting and roughness.

The rutting index already has a substantial basis from accelerated pavement testing (APT) and long-term pavement performance (LTPP) data. However it requires further calibration as LTPP sites age or as specific roads with known rutting performance and past traffic are identified as suitable candidates for reliable calibration. The roughness model is provisional only as no significant change in roughness has yet developed on the LTPP sites. However the model has been tentatively calibrated assuming all the LTPP sites began life with minimal roughness, and that their past traffic has been realistically recorded. An ongoing study is investigating structural indices for flexure and shear. The flexure model is advancing to a moderately reliable stage, while the shear model is still in the early stages of development.

Each structural index is mechanistically derived and has the same range and general distribution as the traditional SNP, allowing straightforward implementation (substituting the relevant structural index for SNP) with minimal additional calibration needed for existing World Bank HDM/dTIMS asset management systems. As the amount of data from LTPP sites grows, the improved mechanistic understanding of pavement performance can be readily incorporated, by refining (or redefining the basis of) the structural index for each distress mode. Provided the base (raw) data remains stored in RAMM, updated structural indices may be readily generated at any future time for any network.

The structural indices were tested for:

- application to current pavement deterioration models by directly replacing the SNP with the structural index, with a distribution similar to that on LTPP sites. This test had a negative outcome and a recommendation is made to re-analyse the regression when a structural index is adopted. The reason for this is that the structural index is fundamentally different from the SNP, thus suggesting a simple replacement is not valid.

- application to pavement deterioration models, incorporating the index during the regressions of model development stage. This test had a promising outcome showing that the indices are more significant indicators than the SNP.
- use as a direct indicator of maintenance needs. This test showed expected trends in the results. However, it did not have a one-to-one relationship with the field decisions as it had not been part of the field validation process. In reviewing the results, it was noted that the indices were extremely helpful in making appropriate decisions. For example, the test highlighted that some scheduled resurfacings were not appropriate given low indices for rutting and shoving, which suggested a need for rehabilitation.

Further development and refinement work required are summarised in the table below.

Table 1 Further development and refinement work on indices

Item	Description of further work required	Data source/methodology
$SI_{rutting}$	Minor refinement. Calibrate to those regions with subgrades known to perform anomalously (eg Taranaki brown ash and Central Plateau ashes	Roads or networks with well-known performance (rutting distress and known past equivalent single axle)
$SI_{flexure}$	Wider calibration particularly to different surfacings: asphaltic concrete (AC) versus open-graded porous asphalt versus multiple seal layers	Project level testing of terminal sites
$SI_{roughness}$	Major refinement, as this is an important yet the most difficult parameter to characterise	The challenge is to find roads that have not been complicated by unknown past maintenance or 'non traffic' damage (eg service trenches)
SI_{Shear}	Separation of shear instability: • beneath AC surfacings • beneath thin seals on unbound basecourse • within multiple seal layers	Project level testing of terminal sites
Pavement prediction models	This research has demonstrated that pavement prediction models need to be redeveloped/refined from first principles if new indices are incorporated	LTPP and some limited network data
Network applicability	Extend the range of the indices by conducting more tests on other networks	Do this as part of the over-all network testing programme
Pavement modelling	Investigate further adoption of the indices within the dTIMS system. For example, it may well be utilised as triggers and additional reporting measures within the system	Deliver the structural indices to the modelling community for further investigation.
Risk index development	The indices promise a significant value to defining a risk index. Fundamental development work needs to occur in this area	Development needs to be based on a combination of network, LTPP and CAPTIF data

Appendix B of this report documents the recommended guidelines for FWD surveys and applications of the structural indices. These guidelines should be viewed in their complete form and are therefore not summarised in the executive summary.

Abstract

Pavement performance modelling for New Zealand roading networks, currently relies on an adjusted structural number (SNP) which is a single parameter intended to describe the performance of a multi-layered pavement structure in terms of its rate of deterioration with respect to all structural distress modes, as well as non-structural modes. This parameter had its origin in the AASHO road test in the late 1950s, before the advent of analytical methods. Hence refinement to keep abreast of current practice in pavement engineering is overdue.

This research describes the basis for a new set of structural indices and how these can be used to obtain improved prediction of pavement performance: both at network level and for project level rehabilitation of individual roads. The results are (i) effective use of all the data contained in RAMM, (ii) more reliable assignment of network forward work programmes, (iii) reduced cost through targeting only those sections of each road that require treatment and (iv) more efficient design of pavement rehabilitation through informed appreciation of the relevant distress mechanism that will govern the structural life of each individual treatment length.

1 Introduction

1.1 Background

The adjusted structural number (SNP) (previously used as modified structural number (SNC)) is the basis of most prediction models such as the World Bank Highway Design and Maintenance (HDM) models and the dTIMS maintenance planning system. It is an important measure of the pavement capacity of networks (especially for performance-based networks). It is the only measure to date that tells asset managers how much capacity/life can be expected from their networks. However, the SNP principle has its limitations which include:

- It can be measured and determined in different ways – and these methods do not always correspond (HTC 1999).
- It is based on the American Association of State Highway and Transportation Officials (AASHTO) design philosophy that aims to protect the subgrade – although in many cases New Zealand roads fail due to the weakness of constructed layers. For example, a strong pavement with a high SNP may still fail within the first year of construction due to a weak basecourse.
- The original (most widely used) SNP derivation is based on the summation of empirical layer coefficients, which are based on either test pits layer information or falling weight deflectometer (FWD) tests. Current research, however, is showing that much greater predictive reliability can be achieved by deriving SNP from a more fundamental relationship with permanent deformation and supplementing the determination with other (non-destructive) parameters usually available in the RAMM database: i) FWD; ii) high-speed data (rutting and roughness) and iii) past traffic (equivalent single axles (ESA)). This advantage is now practical because of the availability of high-speed data collected annually for the network.

SNP is a fundamental parameter for network analysis, and while it currently has deficiencies, it should be retained because of its well-established role describing pavement performance in terms of a single parameter. This report documents a process to rationalise the derivation of the parameter rather than seek an entirely new prediction procedure.

1.2 Problem statement

Road asset management systems are generally based on the concept of assigning a single parameter that relates to the characteristic structural capacity (sometimes loosely termed ‘strength’) of a defined segment of pavement. These segments are intended to represent potential treatment lengths that are essentially uniform in performance. The structural number concept was one of the most widely used parameters for many decades and while significant changes have evolved (ie SNC and SNP) the original basic principal was to provide a representation of the load-bearing ability of a pavement. Pavements are consequently ranked according to the relative permanent deformation expected within the subgrade for a given number of standard load repetitions.

The SNP can therefore be used as an approximate indicator for the structural life of pavements provided that:

1. rutting is the governing distress mechanism, ie no other trigger for rehabilitation applies
2. the majority of the rutting occurs in the subgrade rather than within overlying layers

3 the treatment length is correctly defined and relates to a uniform sub-section

4 the appropriate percentile (rather than average) SNP is determined corresponding to the percentage of road in a terminal condition which would trigger rehabilitation.

These four conditions must be satisfied before SNP can be used. The first condition, however, is questionable for many roading networks (Henning et al 2006), indicating a substantial limitation to the SN concept that needs to be addressed. In particular:

- The governing distress mode (ie the distress mechanism that triggers rehabilitation of any given treatment length) must be determined before any rational or reliable indicator of pavement life/structural capacity can be calculated.
- To determine the governing distress mode, deterioration models need to examine all potential distress modes using relevant parameters for each one. For example, when predicting cracking, an index should be used that reflects the pavement's stiffness and fundamental tensile strain conditions.

1.3 Objective of the research

There are currently many decisions – with substantial cost consequences – being made about network management that are based on an outdated parameter. These can be rationalised to provide a markedly more robust prediction of long-term pavement performance (LTPP). The task is now to apply the results of recent CAPTIF and interim LTPP research efforts to all NZTA and local authority sites and then to a variety of NZTA and local authority networks. The results of this task will calibrate and substantiate a new structural capacity derivation that will remedy the current limitations inherited from the SNP concept.

The objective of this research was to provide a method of deriving a structural measure that would deliver a parameter based on fundamental principles relating directly to the performance of pavements that have been closely monitored. Documentation on pavement performance dating back more than a decade is available from CAPTIF as a result of annual monitoring since 1999 of national LTPP sites set up by NZTA (formerly Transit NZ), and since 2003 of those set up by local authorities.

In order to achieve the above objective the following had to be undertaken:

- Develop structural indices, on the basis of fundamental mechanistic principles, to quantify the pavement structural capacity of road networks in New Zealand.
- Test the new structural indices in terms of their applicability to pavement performance modelling, maintenance decision processes and road network reporting.
- Investigate the potential of these indices to forecast the risk of pavement failure.

With an improved structural capacity measure, the outcome should be a robust, statistically derived sampling regime policy for FWD tests.

1.4 Scope of the research

The original objectives and scope of the research envisaged an outcome that would result in some adjustment to the SNP/SNC concept. However, the research was successful in delivering a completely new concept, which will significantly improve the usefulness and applicability of the structural number concept for both asset management and maintenance design purposes.

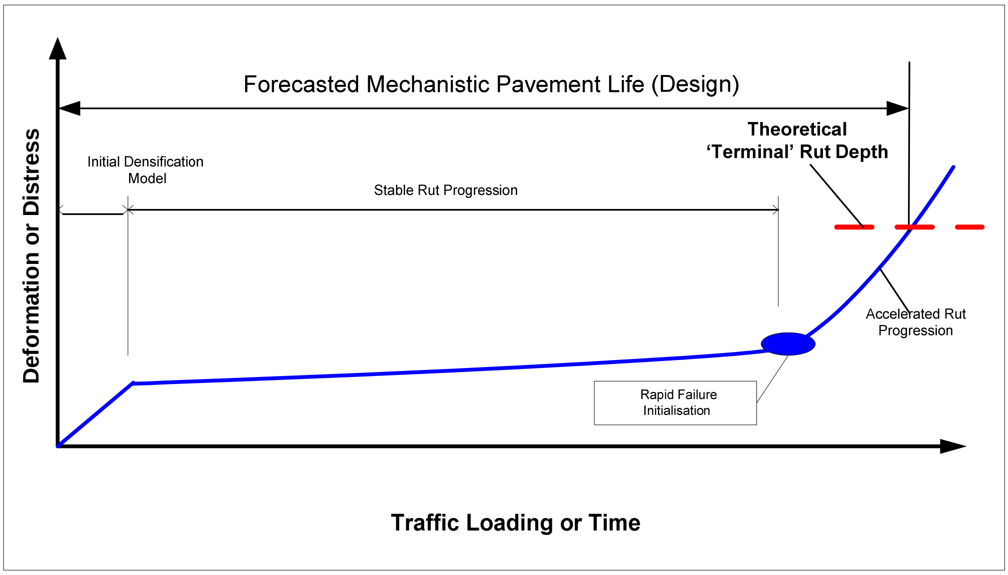
This report discusses the delivery of the following outcomes:

- a framework for reporting pavement structural capacity/performance based on multiple structural indices (SI), one for each distress mode including:
  - $SI_{Rutting}$
  - $SI_{Flexure}$
  - $SI_{Roughness}$
  - SI_{Shear}
- successfully tested the applicability of these indices on network FWD data
- demonstrated the significance of the indices as independent variables within the New Zealand pavement predictions models.

1.5 Concepts and definitions relevant to this research

Given the nature of this research, there is a strong overlap between pavement design principles and principles used for asset and network management. For example, both use the word 'model' to describe a mathematical/statistical expression that forecasts the future 'performance' of the road pavement. This section explains the difference between these approaches and defines the definitions applicable to this report. Figure 1.1 present a typical deterioration curve of a pavement as a result of rutting.

Figure 1.1 The relationship between pavement design model and pavement deterioration model



The figure illustrates the two forecasting mechanisms: the pavement deterioration model and the pavement design mechanistic model. The differences between these and their relationship to the SNP is presented in table 1.1.

Table 1.1 The relationship between SNP/SNC for respective applications

Application	Description	How the SNP/SNC relates
Pavement deterioration models eg: • initial rut depth • stable rut progression • probability of accelerated rutting	Most of the New Zealand-based deterioration models consist of: • probabilistic empirical models, which predict the initiation of a defect or occurrence • empirical deterministic model, which predicts the progression of a defect.	Almost all the New Zealand and HDM-4 models use SNP as an indicator of pavement structural capacity as an independent variable.
Mechanistic design models (eg Austroads 2009)	These models use layered elastic theory in order to model the stresses and strains for given layers and subgrade. Once the stresses and strains are calculated/ modelled for a standard load ESA, transfer functions are used to calculate the ultimate life/capacity of the pavement.	In theory, there is not necessarily a relationship with SNP/SNC. However, through research, empirical relationships have been developed to relate SNP/SNC to in situ test results such as the deflection bowl from FWD (Salt 1999).
Structural indices: • $SI_{Rutting}$ • $SI_{Flexure}$ • $SI_{Roughness}$ • SI_{Shear}	The structural indices consist of a combination of the mechanistic behaviour of the pavement as measured with the FWD and pavement condition.	One would not expect a one-to-one relationship with the SNP as these indices describe different failure mechanisms. Later chapters in this report contain more details.

2 Limitations of the SNP concept

2.1 Pavement distress modes

Dawson (2002) identified 23 distress modes in pavements, although some of these are consequential manifestations of one or more of the other listed modes, and others are surfacing wear rather than structural deterioration. The focus of the current study is to provide improved systems for structural life prediction, based primarily on pavement data that are routinely measured in current practice, ie information currently stored in RAMM.

If one excludes non-structural distress modes, those that are rarely encountered and those that refer to unsealed roads, Dawson's 23 modes can be reduced to the following:

Rutting – vertical surface deformation resulting primarily from one dimensional densification (compaction) of the pavement layers and the subgrade. Some lateral movement may also take place in the early life of the pavement but in the current classification for 'rutting' it is assumed lateral movement rates will be minimal after the 'bedding in' phase.

Shear – lateral deformations or shoving within the pavement layers primarily related to shear. There will be an associated increase in rut depth which is likely to increase rather than stabilise with ongoing load repetitions. Shear instability will commonly lead quickly to subsequent defects such as cracking of the surfacing, pumping and potholing.

Roughness – loss of shape longitudinally along each wheel path. There are two prominent causes of roughness progression which include environmental effects and traffic load (Watanatada et al 1987). The load associated progression is primarily governed by structural non-uniformity (longitudinally) leading to variations in rut depth. Roughness could also be a secondary effect of shear instability and repaired defects such as crack sealing and pothole patches.

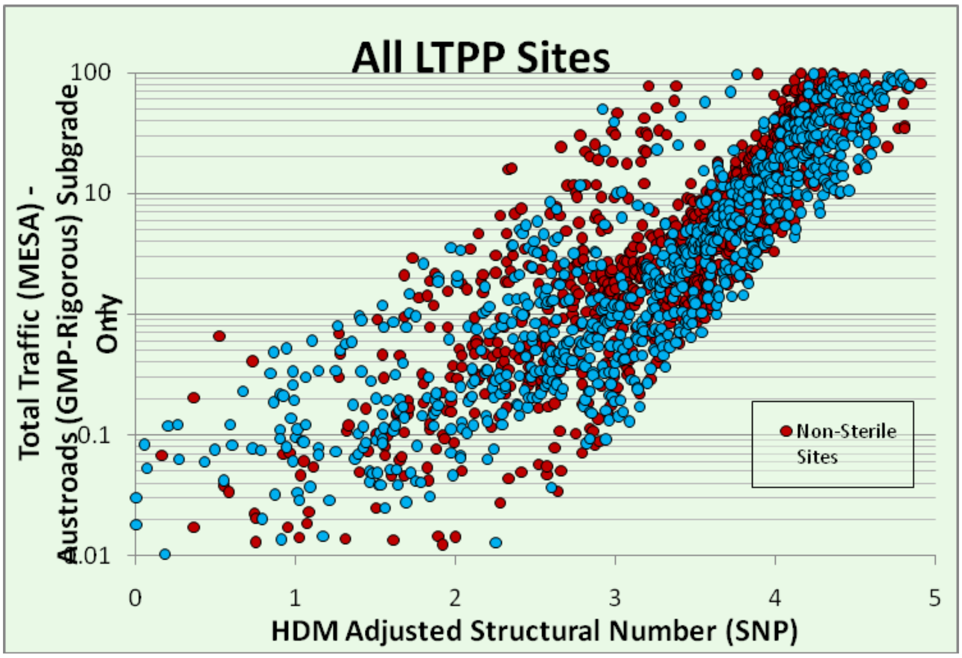
Flexure – the imposition of horizontal strains within the surfacing as a result of trafficking. Strain reversal will occur as the deflection bowl passes along the wheel path: (compressive – tensile – compressive) at the bottom of the surfacing and generally the reverse sequence at the top. The tensile strains result in crack initiation within the surfacing, followed by water ingress, secondary shear instability, pumping and potholing. This mode primarily affects the surfacing and usually results in excessive maintenance costs. Additional surfacing may be sufficient for substantial life extension if the existing surfacing is thin, but thick or aged surfacings suffering from excessive flexure are likely to require replacement or other structural rehabilitation.

2.2 Using structural number as an indicator of pavement capacity

The empirical structural number concept has been widely used in American procedures. It had its origin in the American Association of State Highway Officials (AASHO) road tests in the late 1950s, before mechanistic design methods were in general use (AASHTO 1986). In the 1980s and 90s, structural number became the structural backbone of the HDM III model (Watanatada et al 1987) and the AASHTO pavement design guide (AASHTO 1986). However as AASHTO moved towards mechanistic design in the originally planned 2002 release of its mechanistic pavement design guide (M-EPDG) (now under continuing development as the NCHRP (Ullidtz and Larsen 1998)), the structural number concept was abandoned for the purposes of project level assessments. However at the network level the HDM-4 model still retains the concept as SNP.

In mechanistic terms, SNP would be expected to have an approximate relationship with vertical compressive strain at the top of the subgrade induced by a single equivalent single axle (ESA) loading and hence with total rutting life (in ESA as determined by the Austroads subgrade strain criterion). The correlation for all national LTPP sites is shown in figure 2.1. Predicted traffic in excess of 100 million ESA has been ignored as not practically credible.

Figure 2.1 Traditional adjusted structural number vs predicted subgrade life (total ESA) using the Austroads subgrade strain criterion



Note: Sterilised sites are sections excludes any routine maintenance
Non-sterilised sites will receive maintenance as normal

The number of ESA to a terminal rutting condition using the Austroads subgrade strain criterion apparently ranges over two or three orders of magnitude for a given SNP value. Also it is now clear from observed performance of pavement trafficking that even under well-controlled conditions such as accelerated pavement testing (Stevens 2006), predictions of the rutting life of a new, or near new, pavement based on structural number concepts can result in errors of two or more orders of magnitude in terms of the number of ESAs to a given terminal rut depth. This has been demonstrated at CAPTIF, (Stevens 2006), while similar findings have resulted from the Australian accelerated loading facility ALF (reported by Salt and Stevens 2005). The other structural distress modes (shear, roughness and flexure) must inevitably show even poorer or no correlation with SNP, because SNP is a parameter that is basically a measure of load spreading to the subgrade.

The problem is that the structural number concept is a ‘one size fits all’ approach. It provided an excellent starting point at the time but its nature precludes any progression of the state of the art. It does not acknowledge all the advances over the last 50 years in pavement engineering in general and mechanistic analysis in particular. The NCHRP rejection of the structural number concept is therefore appropriate. An outline of the replacement system (the mechanistic-empirical pavement design guide or M-EPDG) is given in the following section.

2.3 NCHRP modelling of pavement strength

With the 2002 pavement design guide (explained during the development stage by Tseng and Lytton 1989), the NCHRP has adopted a mechanistic-empirical pavement model, where all inputs are pavement properties that define the response of pavement to traffic and climate loads, primarily by assessing the moduli of the pavement materials and hence allowing determination of stress and strains throughout the pavement under a standard axle load. Empirical criteria are then derived from observed performance to complete the model.

Assessment of structural adequacy for an existing pavement is based upon:

- load-related distress, trafficking spectra
- material durability
- back-calculated layer elastic moduli
- visual examination of pavement cores
- physical testing of samples to determine moduli and strength.

For the structural modelling of a pavement, critical stresses, strains and displacements (due to traffic loading and climatic factors) are calculated over the total pavement thickness in a layered elastic model using layered elastic theory. The M-EPDG uses either linear elastic or finite element methods for non-linear materials. The pavement is modelled to accumulate monthly damage over the design period. This 'incremental damage' is then related to specific distress modes with calibrated empirical models relative to pre-defined treatment criteria. The model for each distress mode incorporates only those physical properties that current research has shown contribute to the mechanism of pavement failure. That is, it requires fundamental pavement layer properties instead of conventional empirical parameters like SNP.

Since there is a large degree of uncertainty in the input data, much of the modelling utilises probability distributions for the data, and the designer can then select the level of design reliability they wish to proceed with. The model has been designed with the data available from the US LTPP sites. In particular the enhanced integrated climatic model, EICM, uses a large body of climatic data collected alongside structural and traffic data for pavements to give sound inputs for seasonal variation in pavement stresses.

Mechanistic modelling in the Austroads (2009) guide to pavement technology series is more simplistic, but does not explicitly consider the same number of distress modes addressed by the M-EPDG. Ultimately the M-EPDG is likely to provide an effective and comprehensive means of pavement design and performance prediction. However, so far it has been poorly supported, apparently due to its complexity. Many of its parameters are not currently available in the RAMM database; hence at this time it is impractical for local practitioners. However, there is scope for a pavement capacity model which is intermediate between NCHRP and Austroads procedures, limited solely to parameters currently contained in RAMM.

2.4 Requirements for an improved pavement structural capacity model

The focus of this study has been to establish a practical system for improved prediction of pavement structural capacity. The essential elements for the system are:

1. Rational modelling of all relevant structural distress modes using fundamental mechanistic concepts including allowance for either linear elastic or non-linear materials as applicable.

2 Ensuring practical inputs, ie limiting to collected data (or data that can readily be collected).

3 Straightforward incorporation into existing asset management software (eg dTIMS) and pavement deterioration models contained in the software.

4 Ease of incorporating improvements to pavement technology or data collection methods.

5 Ease of calibration to different networks or sub-networks where different materials or construction practices apply.

The NCHRP model meets only the first of the above five criteria and hence at this stage has not been regarded as suitable for inclusion in the study or for consideration by practitioners who need a system for immediate application. It will, however, be important to continue to review the status and potential for future application of this major research project.

Austroads principles apply to most of the five criteria. A key exception is that the Austroads principles do not fully acknowledge non-linear (stress dependent) moduli. Many parts of Australia have materials that are not saturated silts/clays and hence are essentially linear elastic, (D Mangan pers comm.). However those in New Zealand are predominantly non-linear as shown in a study of New Zealand LTPP site characteristics (Salt and Stevens 2006).

An interim measure for improved pavement modelling is a replacement of the SNP with mechanistically derived and fundamental structural parameters for rational prediction of pavement behaviour. Separate parameters are required for each structural distress mode under consideration. SNP is incorporated in many pavement deterioration models used in management systems such as dTIMS. This report presents the **rutting** and **roughness** parameters which have been developed based on observed performance from both APT and national LTPP sites. Mechanistic analyses have been used to determine the moduli, stresses and strains under a single ESA loading.

In order to establish the basis for pavement structural capacity models, it was necessary to first define all the essential rules or characteristics that the model must acknowledge (and hence incorporate) in order to be rational and then carry out the development in such a way that ensured the model remained as simple as possible for practical purposes. As a result of the literature study and APT/LTPP data, about 20 essential elements for a pavement performance model were defined (see appendix A).

The reason for setting out the elements that need to be considered in the model (intended to reflect current consensus) is so that the basis of the current model can be readily understood and critically reviewed by other practitioners. Hence this process should facilitate future refinements or revisions of the capacity model. The need for refinement will be indicated by better or more easily generated parameters that show improved prediction. These parameters would be based on the steadily growing LTPP database.

2.5 Inherent model limitations

There are fundamental reasons why a high-precision model will never be obtained for pavement structural deterioration. Some of the greatest influences on performance predictability are:

1 Subgrade: non-uniformity inherent with naturally formed strata.

2 Construction: in-service pavements will inevitably have some degree of variation in layer thickness, compaction (laterally and vertically) and water-proofing (laterally).

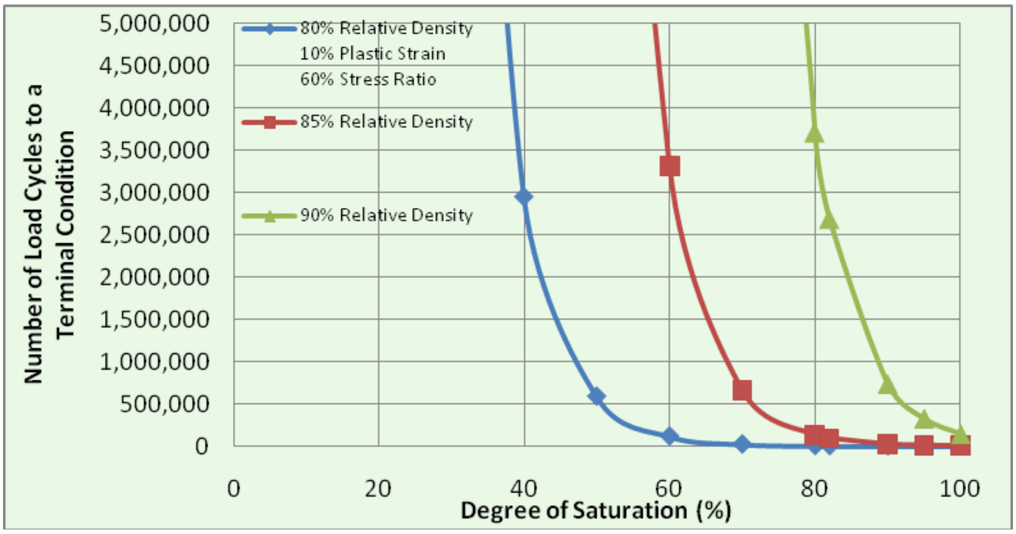
3 Materials: granular pavements are composed of particulate materials, ie each layer comprises an assemblage of discrete particles of varying sizes. Statistically, for any layer, the particle size distribution will vary both laterally and vertically within the layer, even if 'practically uniform' mixing

has been achieved during construction. Consequently, the deformation characteristics will change from point to point.

4 Environment: even along a short treatment length, environmental conditions will change due to differences in drainage (both surface and subsurface) and exposure (aspect, shading etc).

5 Material water content: most of the above items contribute to continuous variation in the material water content. Various researchers have reported that pavement deformation characteristics are extremely sensitive to water content. Very small changes in in-situ water content can generate disproportionately large changes in performance of unbound granular layers. An illustration of this process is available from laboratory repeated load triaxial testing where the rutting life (number of load cycles to a given permanent deformation) diminishes rapidly at high levels of saturation (Theyse 2002). While subtle differences may apply for in-situ performance of aggregates, a similar trend for rapidly diminishing number of load cycles to a terminal rutting condition can be expected as saturation increases.

Figure 2.2 Impact of saturation on the performance of unbound aggregates (after Theyse 2002)



3 Development of improved parameters

3.1 Conceptual basis

SNP is traditionally used along with other parameters (notably past and future traffic) to predict pavement structural capacity. The process uses observed performance and regression analyses to get the best fit of predicted to observed performance.

In order to allow existing regression equations to be readily adapted or redefined, a set of additional structural parameters has been established. To distinguish them from SNP, these have been termed structural indices (SI), one for each distress mode.

Addressed in this study:

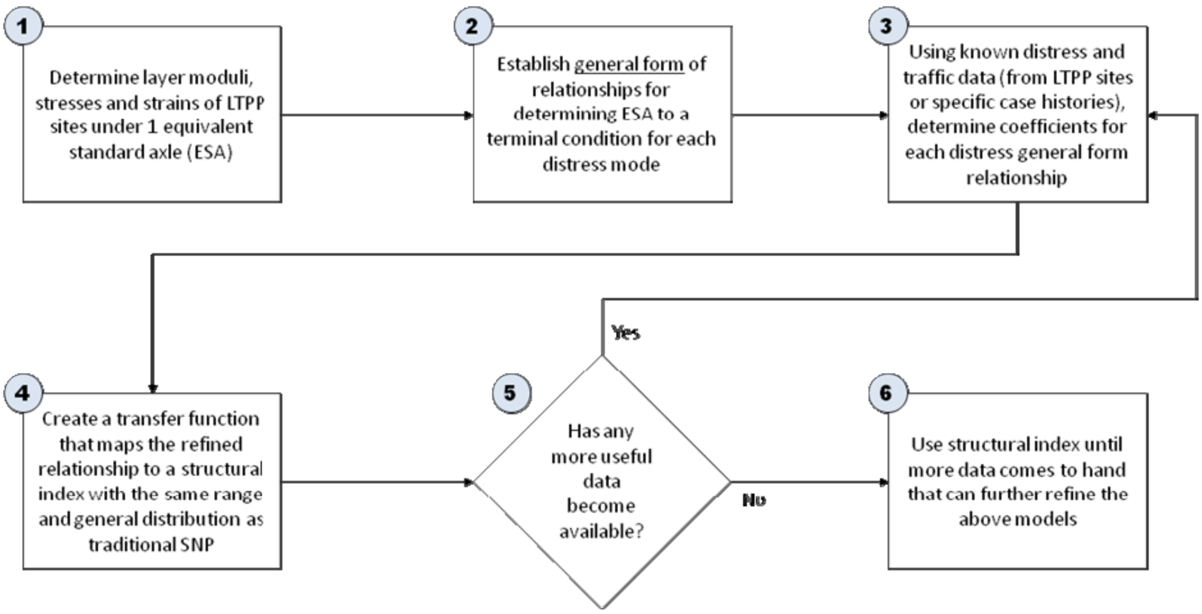
- rutting: $SI_{Rutting}$
- roughness progression: $SI_{Roughness}$

Addressed in an ongoing, separate study:

- flexure related distress: $SI_{Flexure}$
- Shear instability: SI_{Shear}

The general process for determining the structural index for each distress mode is illustrated in figure 3.1 and further expanded in the following list.

Figure3.1 Process of developing structural indices



1 Undertake structural analyses of APT/LTPP sites determining layer moduli, stresses and strains under heavy traffic loading (1 ESA).

2 Establish the general form of a rational relationship that can be expected to determine pavement life (ESA to a terminal condition) for the specific distress mode as a function of fundamental mechanistic parameters (focusing on Austroads principles, moduli and critical stresses/strains). These include

primarily parameters that are currently recorded in RAMM although any other readily available sources should also be considered.

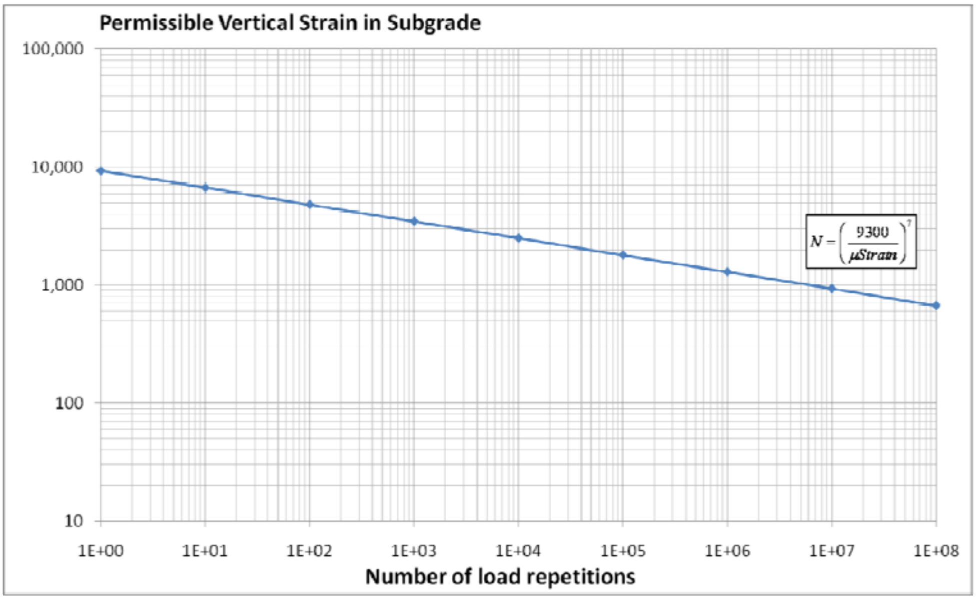
3 Using the observed distress at APT/LTPP sites, supplemented with other case histories of pavements due for rehabilitation, generate a preliminary calibration for the general relationship established. These are currently all the forms of an expected pavement life in terms of number of ESA (NMODE) until a terminal condition is reached for the specific distress mode.

4 Determine a transfer function to convert the pavement life into a structural index (for that specific distress mode) so that the structural index has the same range and general distribution as the traditional SNP. By adopting the same range the adoption is simplified, promoting the acceptance and understanding from the industry.

5 Repeat the above steps to further refine (or totally revise) the model as more data come to hand from the LTPP sites and other sources.

A starting point for all models was to explore existing transfer functions that have been widely used for many decades, eg for rutting the allowable subgrade strain for a given traffic loading (ESA) is shown in figure 3.2.

Figure 3.2 Austroads transfer function for anisotropic subgrades



Using the deflection bowl from a FWD test the strain at the top of the subgrade is readily calculated (Austroads 2009) and the number of ESA to a terminal rutting condition is calculated from figure 3.1 following steps 1 to 3. This would provide the simplest method of estimating rutting life using Austroads principles. Step 4 is then calculated using an appropriate transfer function as described in the following section. The above transfer function was last modified by Austroads in 1998 (Moffat and Jameson 1998). When a better method of estimating rutting life is established (through developments in testing methods or interpretation), the process can readily be repeated to obtain an improved estimate of the structural index under consideration. The proposed new system is simply a framework to allow the structural number concept to be refined until a comprehensive mechanistic model (eg NCHRP) for all distress modes, advances to the stage that it can be readily adopted by practitioners.

The structural indices generated are normalised to a similar range of SNP to minimise the effect on existing regression relationships already obtained with the New Zealand LTPP programme (Henning 2008) or HDM-4 (Hoque et al 2008).

The SNP or structural indices other than the one applicable for the specific distress mode under consideration could prove significant in a new regression analysis which should be an indication that the mechanistic basis of the pavement performance model needs closer examination.

3.2 Structural indices for specific distress modes

3.2.1 Rutting

The method of generating a structural index for rutting given in section 3.1 using only Austroads principles and subgrade strain is now recognised as being an over-simplification. Strains in the overlying layers clearly contribute to rutting also and quantification is relatively straightforward. The development of an interim rutting model from existing APT and LTPP data is described in Tonkin & Taylor (2006a). The model is based not only on the vertical compressive strain at the top of the subgrade (as standard for Austroads procedures), but also the vertical compressive strains at the mid-depth of each pavement layer, and the thicknesses of these layers.

The general form of the widely used subgrade strain criterion was adopted as the starting point for evaluation:

$$N_{SG} = \left(\frac{k_{SG}}{\mu\varepsilon_{SG}} \right)^{n_{SG}}$$ **Equation 3.1**

where:

$\mu\varepsilon_{SG}$ is the vertical strain at the top of the subgrade (microstrain)

N_{SG} is the number of load repetitions to reach a terminal condition

k and n are constants for the material

FWD testing and evaluation of national LTPP sites indicate that strains in any unbound basecourse or subbase layer could be limited by a granular strain criterion of the same general form as that used for the subgrade, namely:

$$N_{G_i} = \left(\frac{k_G}{\mu\varepsilon_i} \right)^{n_G}$$ **Equation 3.2**

where:

$\mu\varepsilon_i$ is the vertical strain at the mid-depth of layer i (microstrain)

N_{Gi} is the number of load repetitions to cause a terminal condition in layer i

The procedure for design using strains in all layers is as follows:

1 Determine the number of load repetitions (NSG) expected to cause a terminal condition in the subgrade

2 Determine the number of load repetitions (NGi) to cause a terminal condition in each of the granular layers (1)

3 Sum the relative wear per repetition using a weighted average of the wear in the subgrade and granular layers, and hence determine the total number (N_T) of load repetitions to reach a terminal condition from permanent deformation within the full depth of pavement

Using this process, the following relationship is implied:

$$N_T = \frac{1}{\sum_{i=1}^{nLayers}\left(\frac{a_i}{N_{G_i}}\right) + \frac{a_{SG}}{N_{SG}}} \quad \text{- Equation 3.3}$$

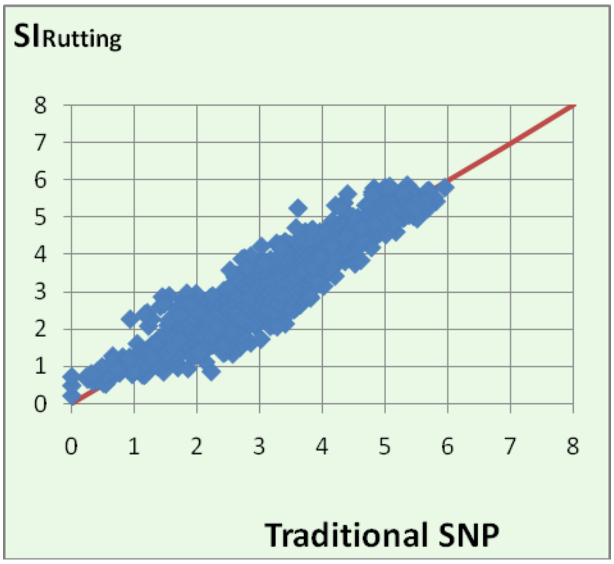
where:

a_i and a_{SG} are weighting functions determined from back analysis of the standard pavement design chart for unbound granular pavements with further verification and calibration from LTPP sites.

Equation 3.3 may be regarded as the pavement rutting life based on multi-layer strains. If the layers overlying the subgrade have small permanent strains or if they are neglected then the first term in the denominator vanishes and the expression reduces to the traditional subgrade strain equation. Further details are given in Tonkin & Taylor (2007b).

This rutting model gives a predicted life (ESA to a terminal rutting condition, $N_{Rutting}$) directly and transfer functions were then trialled to find the best fit for a structural index for rutting ($SI_{Rutting}$) to the range and distribution of SNP for all LTPP sites, as shown in figure 3.3.

Figure 3.3 $SI_{Rutting}$ vs SNP for LTTP site data



To illustrate the difference between the old and new parameters, as an example it may be noted from figure 3.3 that a traditional SNP value of 3 will be replaced in the new system with a value which may be as low as 1.7 or as high as 4.2, once the more fundamental stresses and strains are evaluated. The predicted life (number of load cycles to a given deformation) can therefore be substantially different in the two systems for a specific road. The network average pavement life, however, could be very similar for the two cases.

The current form of the transfer functions is explained in chapter 3, along with the inverse functions that could also be used as a basis when a new regression is being explored. Note the constants given for the functions in table 3.1 apply to state highways. If they are used for local authority networks (where the thickness of pavements is likely to be lower), large calibration shifts may be needed. Further discussion on local roads is given in section 3.4.

3.2.2 Roughness

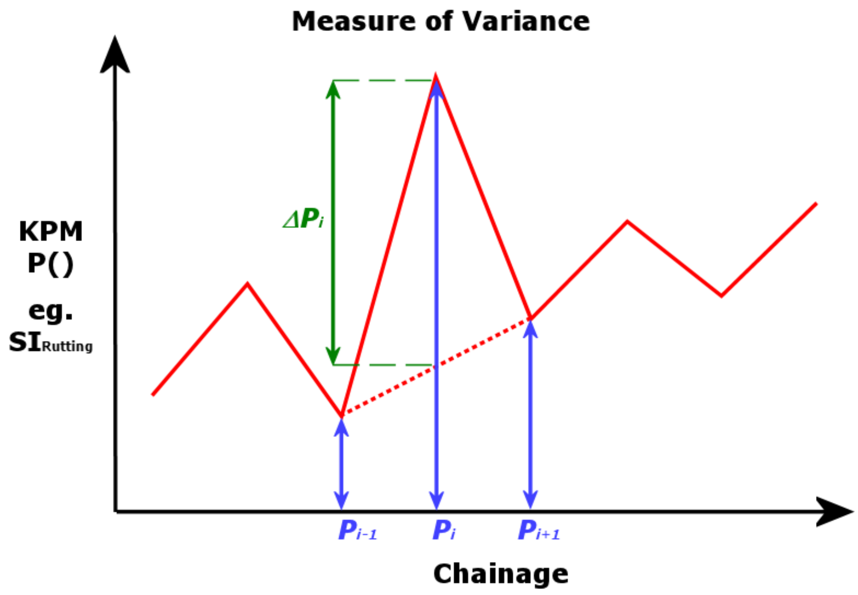
All LTPP sites had been trafficked for many years prior to initiation of the LTPP study; hence the true start of life condition for each site can only be assumed. So far, the change in roughness has been minimal at all national LTPP sites over the period of monitoring, and measurement of roughness progression has been necessarily limited in the relatively short lengths involved with local APT studies.

Therefore the only way to develop a model was to base it on the current roughness of the LTPP sites with necessarily approximate assumptions on the original conditions (immediately after construction). It would probably take several more years of observation of the LTPP sites before the roughness progression model would have good reliability (Henning et al 2008). The interim model was based on a recently proposed measure of pavement structural uniformity (based on variations in stiffness longitudinally in each lane), together with the vertical compressive strains in all layers including the subgrade. Superimposed on this was an annual roughness progression based on environmental impacts.

Roughness progression at the LTPP sites showed, as expected, a very approximate dependence on rut depth and rut depth standard deviation. Note that the HDM-4 roughness progression model has rut depth standard deviation as one of its variables (NDLI 1995). The roads least susceptible to roughness progression appear to have progressed at a rate of about one NAASRA count per 1mm of rut depth, while those most susceptible to roughness progressed at about five NAASRA counts per 1mm of rut. The reason for different rates of roughness progression is likely to be due to differences in longitudinal non-uniformity (variance) of each pavement structure, subgrade or construction quality of layers. Several pavement structural parameters were investigated to determine any likely candidate as a measure of non-uniformity. A quantitative key performance measure (KPM) for non-uniformity would also be a useful tool in construction quality control.

Note that it is the variation between *immediately adjacent points* on a road that governs roughness, ie the common measure of standard deviation (used in the HDM-4 model) is not appropriate. The reason is that a given treatment length may have a rut depth which increases *constantly* with distance (say from 0mm at the start of the treatment length to 20mm at the end). The standard deviation of rutting would necessarily be substantial over that treatment length but because the rut depth decreases so steadily, roughness would be expected to be relatively low, compared with a treatment length where rut depth fluctuated repeatedly up and down by 10mm over the full treatment length. The concept of the proposed measure is illustrated in figure 3.4.

Figure 3.4 Definition of local variance (LV) for any given parameter P



$$LV = \sum_{\text{average all stations}} \frac{\Delta P_i}{\text{Average}\left(P_{i-1}, P_i, P_{i+1}\right)}$$ **Equation 3.4**

where:

LV is the local variance for parameter p (p could be say rutting)

P is the parameter considered (p could be say rutting)

ΔP is the absolute value of the change in the parameter, refer to figure 3.4.

This clearly provides a much more relevant measure of variability for the roading situation, compared with the traditional measure of standard deviation.

By summing and finding the average of selected structural parameters in the above expression, various measures of non-uniformity - here termed *local variance* may be obtained. A range of structural measures was investigated to see which would be likely candidates for explaining roughness progression. By ranking the local variance (LV) for a given treatment length in relation to the local variances for the treatment lengths on all LTPP sites, an approximate assessment could be made of where in the scale of roughness susceptibility (above), the performance of given treatment length could be expected. This is the intended methodology for further development and calibration of the roughness progression model, once sufficient data are available from the LTTP sites, or from pavements with well-documented terminal roughness condition and past traffic loadings.

As an interim measure a combined local variance (CLV) has been determined in this study from trial weightings (Tonkin & Taylor 2008b) of three structural parameters normally evaluated for all FWD test points provided by:

$$CLV = LV(SI_{Rutting}) + 0.8 * LV(1-2n) + 0.9 * LV(NMR)$$ **Equation 3.5**

where:

$SI_{Rutting}$ is the structural index for rutting

n is the subgrade modulus exponent for stress non-linearity (Ullidtz 1987)

NMR is the normalised modular ratio, ie. the ratio of moduli between successive granular layers (Salt and Stevens 2007) compared with that expected by the Austroads guide (Austroads 2009)

Using the above, the following gives the resulting $SI_{Roughness}$ in relation to SNP for all LTPP sites.

Figure 3.5 $SI_{Roughness}$ vs SNP for LTTP site data

As noticed in figure 3.3, figure 3.5 also illustrates the problem of a generic 'one size fits all' structural parameter. SNP, designed as a measure of subgrade protection (and hence related to rutting) correlates with $SI_{Rutting}$ over the whole data set (but not for specific test sites). On the other hand, subgrade protection relates to roughness progression only indirectly as the primary determinants are longitudinal non-uniformity in all layers (not just the subgrade) and deformation of the surfacing layer. Consequently there can be little expectation of any correlation between $SI_{Roughness}$ and SNP, hence the absence of any trend in the scatter on figure 3.5.

This scatter also suggests that using SNP as a primary structural indicator of roughness progression, has little prospect of success regardless of any form of calibration. The new approach using a structural index for roughness, establishes a framework but accurate prediction for individual treatment lengths will still be severely limited until longer-term monitoring from available LTPP site show more marked changes than have been exhibited so far. Another overriding consideration is of course that roughness prediction is inevitably thwarted by unrecorded maintenance or disturbance (eg trenching for services).

3.2.3 Structural indices for other distress modes

The original scope of this research envisaged a refinement of the SNP concept. However, it became clear that the only way forward was to develop individual indices for the identified failure mechanisms. To this extent, rutting and roughness indices were developed while acknowledging that both would still require ongoing calibration and adjustment as more data became available.

In addition, two additional indices for cracking (flexure) and shoving (shear) have been developed in concept only. These indices were tested on network data and are documented in chapter 4. A brief description of the potential make-up of these indices is provided in the subsequent sections. Significant refinement on these indices will be required before they can be published.

3.2.4 Flexure index

A structural index for cracking is readily generated from the widely recognised fatigue criteria based on tensile strain within any bound layers. Austroads (2009) defines these for both cement-bound materials and asphaltic concrete, allowing the number of ESA to a terminal condition to be calculated directly after back-analysis of FWD deflection bowls. Cracking is often followed quickly by entry of water to the granular layers then potholing and is often reflected by increased maintenance costs. The overall process can, however, be regarded as being initiated by flexure (tensile stresses in either the top or bottom of a bound layer). The ESA deduced from the tensile fatigue criterion can then be ranked to a structural index for flexure as discussed in section 3.1. Further information will shortly be available as a further stage of this project.

3.2.5 Shear index

A structural index for shear instability (or shoving) in the uppermost unbound granular layer is under investigation using a combination of in-situ measures obtained from FWD testing:

- vertical compressive strain in the centre of the uppermost layer (from back calculated modulus)
- dissipated energy in the layer (using energy lost during the FWD test)
- residual deflection (permanent deformation) after the FWD impact

A testing programme is underway to investigate occurrences of shear, to refine these and other indicators to give reliable methods for assessing shear potential. One aspect that is becoming clear is that shear instability of pavements surfaced with thin asphaltic concrete is not as easily identified as shoving in a chipseal pavement. The former is often manifested as localised alligator cracking in the wheeltrack (and hence can potentially be confused with flexure) while the latter tends to form as accelerated rutting in the wheelpath and adjacent heave in the shoulder.

3.3 Developing structural indices from a generic model format

In the existing indices, pavement structural life (N_{MODE}) for a specific distress mode is converted to the corresponding structural index (SI_{MODE}) using a transfer function. The form that proved most suitable for this purpose (the Lorentzian cumulative function) has the following structure:

$$SI_{MODE} = a + \frac{b}{\pi}\left[\tan^{-1}\left(\frac{Log_{10}\left(N_{MODE}\right) - c}{d}\right) + \frac{\pi}{2}\right]$$

Equation 3.6

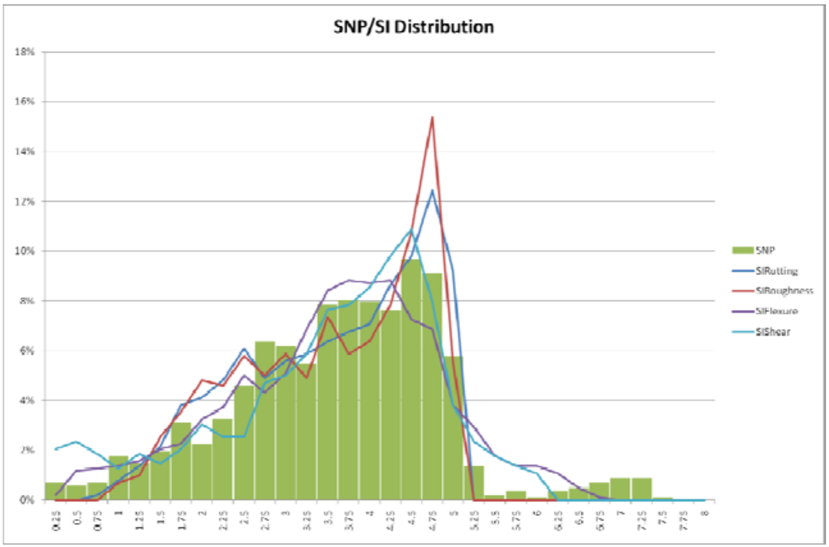
where:

a, b, c and d are constants derived from the optimisation of the distribution of N_{MODE} to the SNP distribution for the network concerned. In this instance the LTPP sites have been used to represent the NZTA's state highway network. The constants for each mode are presented in table 3.1 and illustrated graphically in figure 3.6:

Table 3.1 Function coefficients for structural indices based on NZTA LTPP sites

Mode	A	B	C	D
Rutting	-0.516	6.176	6.61	0.969
Roughness	-0.302	5.709	6.454	0.58
Flexure	-0.478	9.65	8	0.784
Shear	-0.483	9	8	0.851

Figure 3.6 Distributions of structural parameters for all national LTPP sites.



The transfer functions have been derived from the complete set of national LTPP sites and while they are not simple functions, the end result is straightforward in concept and minimal, if any, calibration is likely to be required with the changeover from SNP to the relevant structural index for a similar suite of state highways. Also the new approach will readily allow any other method of determining the number of ESA to a terminal condition to be adopted. The following example from one treatment length of an LTPP site on SH1 illustrates the type of variation between traditional and new parameters.

Table 3.2 Example structural indices from LTPP site BM01

Chainage	SNP	$SI_{RUTTING}$	$SI_{ROUGHNESS}$	$SI_{FLEXURE}$	SI_{SHEAR}
0.100	3.92	4.31	3.40	3.43	4.01
0.150	3.88	4.43	3.53	3.53	4.04
0.200	3.47	3.85	2.97	3.25	3.94
0.250	3.58	3.96	2.33	3.54	4.20
0.300	1.83	2.56	1.94	3.86	4.39
0.350	2.19	3.01	2.47	3.65	4.04

For the transition period (as the new approach is implemented) all five structural parameters can be readily generated from FWD data. If using SNP alone meets the accuracy required for a given network (when assessing structural deterioration and forward work programmes), then clearly no change is necessary. However, where the traditional approach is found to be limited, then the upgrade can be made simply by substituting the relevant structural index in place of the SNP (provided appropriate coefficients are use, as discussed in the following section)..

Equation 3.7 gives the inverse function for equation 3.6.:

$$Log_{10}\left(N_{MODE}\right)=d\cdot\tan\left[\frac{\pi\left(SI_{MODE}-a\right)}{b}-\frac{\pi}{2}\right]+c$$

Equation 3.7

This function could therefore be used to determine the remaining useful life for a pavement.

3.4 Application to local roads

It is important to note that the coefficients in table 3.1 apply for NZTA's LTPP sites on state highways. As local roads are likely to be structurally thinner on average, using table 3.1 would tend to over-estimate the structural capacity, but the relative ranking will still apply, resulting in the need for substantial calibration. However, it is a straightforward procedure to develop the coefficients for any network by following the steps given in figure 3.1. At step 4 in that figure it is important that mapping is to an appropriate SNP distribution for the network under consideration.

For standardisation there are four immediate options being examined through ongoing research:

1 mapping to the SNP distribution for all state highway LTPP sites

2 mapping to the SNP distribution for all local authority sites

3 mapping to the SNP combined distribution of all both highway and local LTPP sites

4 mapping to the SNP distribution of the network concerned.

The first three options will give standardisation between networks, but require recalibration, while the fourth option would be expected to result in minimal or no recalibration

4 Testing the indices at network level

4.1 Indices in the pavement deterioration models

Henning et al (2008) demonstrated that the New Zealand pavement deterioration models correlated better on sections where the primary failure mechanism and the respective model matched. For example, the rutting model would be more accurate on sections where the predominant failure mode was rutting. This was expected as the fundamental format of the models was developed on sections that failed according to that particular defect mode.

It was therefore expected that the new structural indices would be more appropriate variables in the pavement deterioration models, compared with the SNP. The following sections present the testing of the structural index concepts for application on the pavement deterioration models. Two methods were used for these tests: in the first test the SNP was directly replaced by the structural indices and in the second the model was redeveloped using the structural indices as independent variables.

Two networks were used in the testing of the models, namely state highways on the West Waikato network and local roads on the Southland District Council network.

4.1.1 Direct replacement of SNP with structural indices

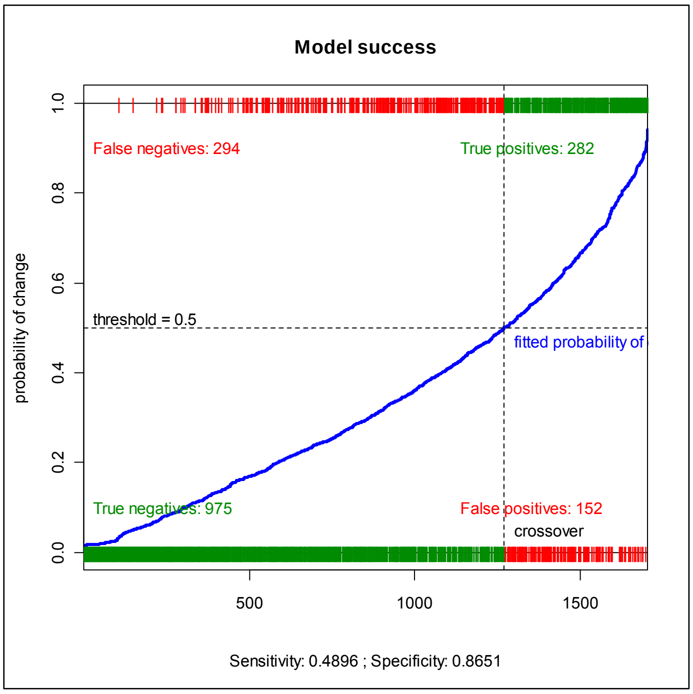
The first test simply replaced the SNP in the pavement deterioration models with the new structural indices. Two models were used namely the crack initiation and rutting models with the $SI_{flexure}$ and $SI_{rutting}$ indices respectively. In both cases the variable and model format were retained but a regression analysis was completed in order to get the variable coefficients and significance of the estimates.

4.1.1.1 Crack initiation

The crack initiation probability model (Henning et al 2006) had a similar outcome regardless of the strength parameter used. The regression process indicated that $SI_{flexure}$ was not a significant factor in the forecasting of crack initiation whereas the SNP was. The overall correlation of the model (containing $SI_{flexure}$) was 75% compared with the actual crack initiation. Figure 4.1 illustrates the model correlation on a network level. Although the overall success of the model predictions seems high it was disappointing to note that of all the cracked sections, only half of these were predicted to have cracked (true positives vs false positives).

The green ticks represent outcomes where the predicted outcomes corresponded with the actual values (true positives and true negatives).The red ticks represent errors: either false positives or false negatives. The threshold value has been assumed as 0.5 (50%) and the ‘positives' are where the probability is above the threshold (50%) and likewise the ‘negatives’ are where the probability is below the threshold (50%). The fitted values have been sorted in ascending order and are indicated by the blue line on the graph. The sample size is the sum of the positives (true and false) and the negatives (true and false). False positives are negative cases that the model missed.

Figure 4.1 Crack probability model containing $SI_{flexure}$



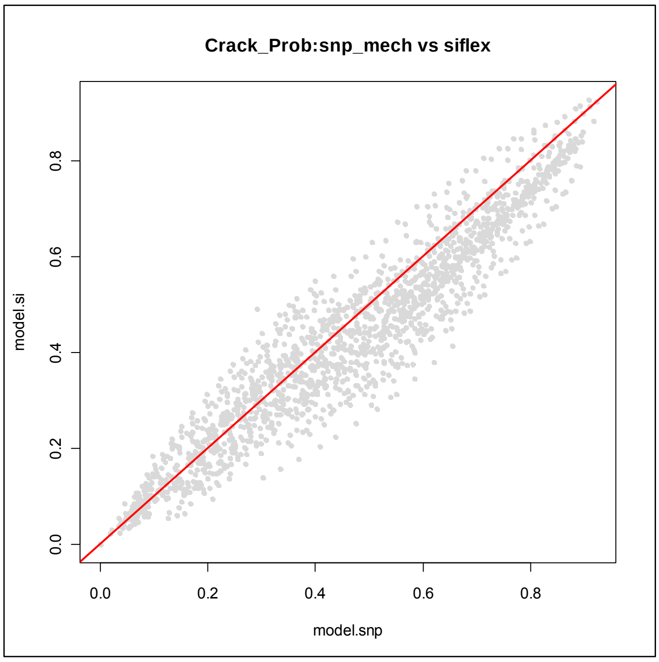
Note: The sensitivity is defined as the ability of the model to find the 'positives', ie sites that actually cracked: sensitivity def =(true positives)/(total positives). In this case 282/(282+294)=0.4896. Specificity is the proportion of 'negatives' that are correctly predicted, which is:

Specificity def = (true negatives)/(total negatives). For this case it is 975/(975+152)=0.8651.

The model is good at finding sites that did not crack (975 out of 1127), and is average at finding sites that did crack (282 out of 576). (Rossiter and Loza 2004)

Figure 4.2 illustrates the relationship between the forecasted crack probability as a function of $SI_{flexure}$ (model.si) and SNP respectively (mdeol.snp). The figure shows that there is a significant variation in outcome from the models, yet there is a one-to-one relationship between these outcomes. It does appear though that the crack probability based on an SNP gave more conservative outcomes overall.

Figure 4.1 Comparing the pavement model outcome between using SNP (model.snp) versus $SI_{flexure}$ (model.si)

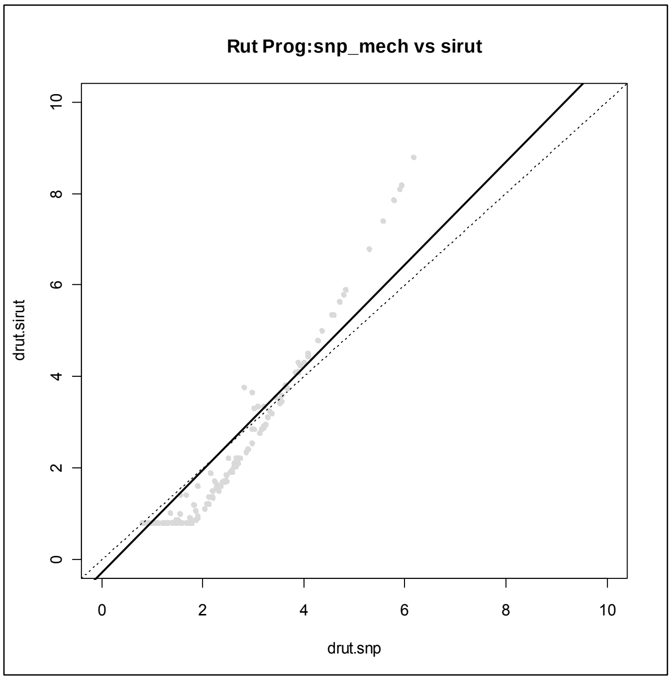


4.1.1.2 Rutting

A similar outcome between the models using the SNP and $SI_{rutting}$ was obtained for the accelerated rut progression.

In terms of the stable rut progression, the forecasted rut rate using SI_{rut} as a strength indicator was more sensitive compared with the model using SNP (refer to figure 4.3). This figure shows that the forecasted rut rate would be similar for an SNP and SI_{rut} of approximately 4mm/year. Below this point, the SI_{rut} index will yield lower rut rates and above four the SI_{rut} will yield high rut rates.

Figure 4.3 Comparing rut progression model outcomes between SNP (drut.snp) and $SI_{rutting}$ (drut.sirut)



Although some outcome was achieved from the tests on these two models, it is fair to say that the results were inconclusive. For this reason it was decided to test the indices on models that were developed based on first principles. The following section discusses these results.

Finding:

It has been demonstrated that the structural index values are significantly different from the SNP. While the correlation between average structural indices and SNP may be strong, the significant variance along the road makes this correlation weak, hence the need for individual structural indices.

Based on this section, it is recommended that the SNP in performance models cannot simply be replaced by the structural index. This replacement needs to include a full regression process which includes all potential variables into a new expression.

4.1.2 Redefining pavement deterioration models using the structural indices

Based on the inconclusive nature of the first test, it was decided to use the new indices on a fundamental model development process. This differed from the previous test in that the structural index was used on an equal basis with all other independent variables, including SNP. Once the significant variables were determined from a number of statistical processes the selected variables were used in a regression analysis in order to determine the final make-up of the model.

This test was undertaken in addition to research work that developed pavement deterioration models for asphalt pavements (Henning and Roux 2008). The aim of this research was to develop cracking, ravelling and rutting models for asphalt-surfaced pavements in New Zealand. There were some constraints that were relevant to the outcome including:

- Due to the shortage of LTPP data for asphalt-surfaced pavements, network data had to be used. Therefore the outcome is still subject to review. In addition, the model development for the rut progression and accelerated rutting was not successful.
- Given the status of the structural index development only the $SI_{rutting}$ was advanced enough to use for the research, regardless of the model that was developed.

Despite the limitations mentioned the test had promising results. The research was successful in delivering a robust model to forecast crack initiation for dense-graded asphalt concrete (AC) surfaces and ravelling for open-graded porous asphalt surfaces (OGPA). In both cases the $SI_{rutting}$ was a significant factor of the models. In the case of the AC crack initiation model, the $SI_{rutting}$ in combination with traffic loading was significant to a 99% confidence level (refer to table 4.1). A similar result was obtained for the OGPA crack initiation model whereas a lesser significance was found for $SI_{rutting}$ used in the OGPA ravelling model.

Table 4.1 Regression outputs for asphalt crack initiation (Henning and Roux 2008)

	Estimate/ coefficient	Std error	z value (sample variance)	Pr(>|z|) (confidence interval)	Significance
(Intercept)	-2.277	0.435	-5.23	1.70E-07	***
H_{new}	0.008	0.006	1.284	0.19914	
factor(PCA)1	3.900	0.462	8.431	< 2e-16	***
AGE2	0.228	0.0160	14.227	< 2e-16	***
R	0.001	0.000	2.075	0.03801	*
factor(PCA)0:Surfnum	-0.003	0.1250	-0.024	0.98102	
factor(PCA)1: Surfnum	-0.678	0.0856	-7.913	2.51E-15	***
Log(ESA):SI_{rut}	**-0.020**	**0.007**	**-2.836**	**0.00457**	**

Notes: Significance codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

H_{new}	is the thickness of the top surface layer
Factor (PCA)	the cracked status of the previous surface layer (0,1) means (false, true)
AGE2	surface age
Surfnum	number of surface layers
ESA	average equivalent standard axles per day
SIrut	structural index for rutting

The findings from this development work are significant as they re-emphasise the logic of having strength indices that simulate the strength based in a failure mechanism rather than having a single parameter based on generalised information. Even though the rutting index was used in the cracking model, it was significant as a predictor. It is believed that the rutting index gives a more realistic estimate of the pavement capacity due to subgrade strain criteria. This capacity has a direct relation to the expected life of the asphalt, hence the better correlation from the regression. The combination of the appropriate index (cracking) combined with the cracking model would display an even stronger relationship. Refinement of the pavement models based on finalised indices is therefore recommended.

4.2 Direct use of indices as maintenance decision tool

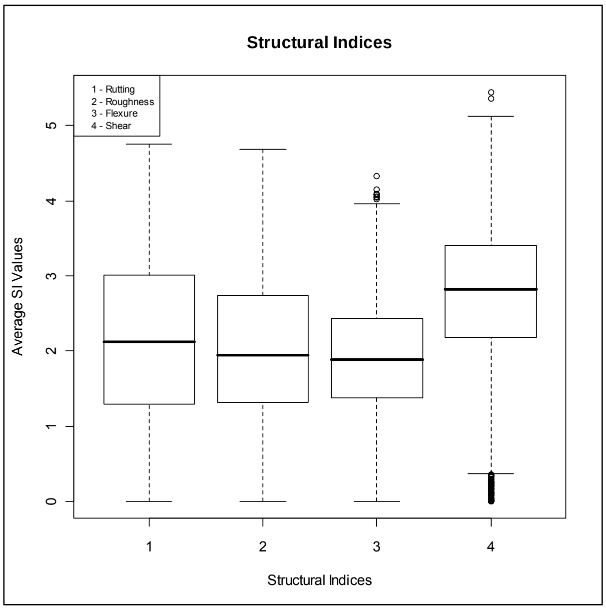
The strength (structural capacity) indices of the pavement are useful decision tools at both network and project levels. In the past, the pavement structural capacity, in combination traffic loading was used to infer a granular overlay need. This information was used at a network level in order to determine whether some of the network pavement life was consumed or not. Although this information gave some useful information it was not very useful at a project level where a diagnostic approach was used to determine the main failure modes and associated remedial actions.

For this investigation the Southland District Council (SDC) network data was populated with the structural indices and compared with the actual maintenance decisions on the network. The result are summarised in the following sections.

4.2.1 Network structural capacity composition

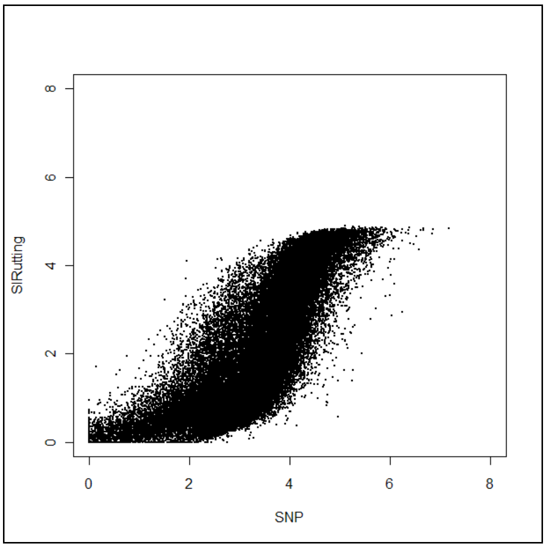
Figure 4.4 illustrates the distribution of the structural indices on the SDC network. It is clear from the plots that the distributions of the indices are different, although the averages seem to be within the same order. The only exception to this observation is the structural index for shear which has a much higher average.

Figure 4.4 Distribution of indices for the SDC network (Schlotjes 2009)

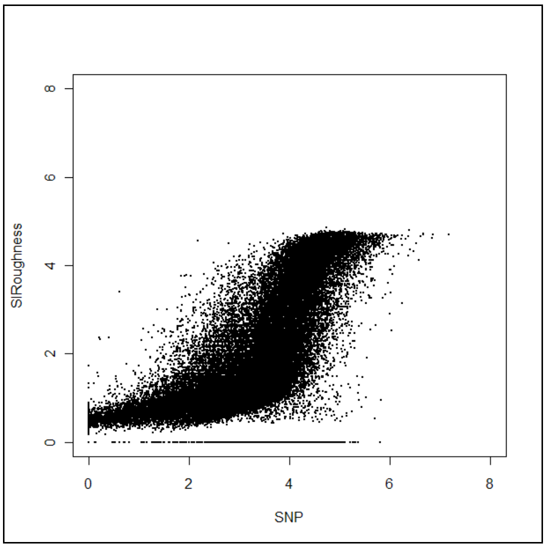


The structural indices on the network have also been compared with the structural number and the results are presented in figure 4.5. This figure shows that there are some similarities with both the rutting and roughness indices compared with the SNP. There is (as expected) no clear trend between the structural index for shear and SNP, while the index for flexure seems to have an exponential relationship with the SNP. Although there are no conclusive interpretations based on these results, it is evident that the indices are different and it is expected that different networks will have different index distributions based on their pavement characteristics.

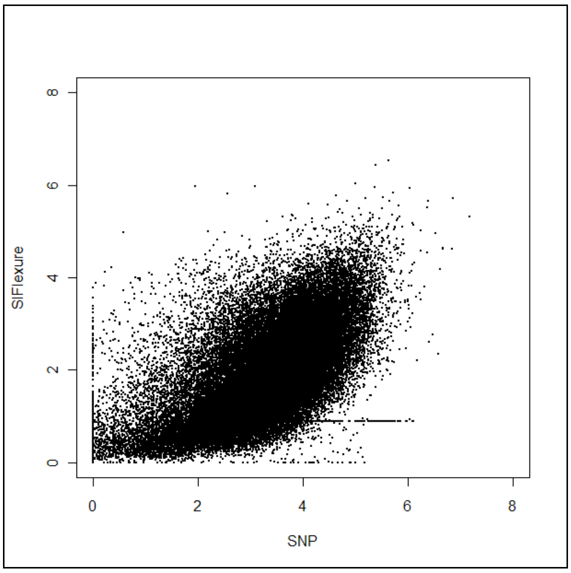
Figure 4.5 Comparing structural indices with structural number on SDC network



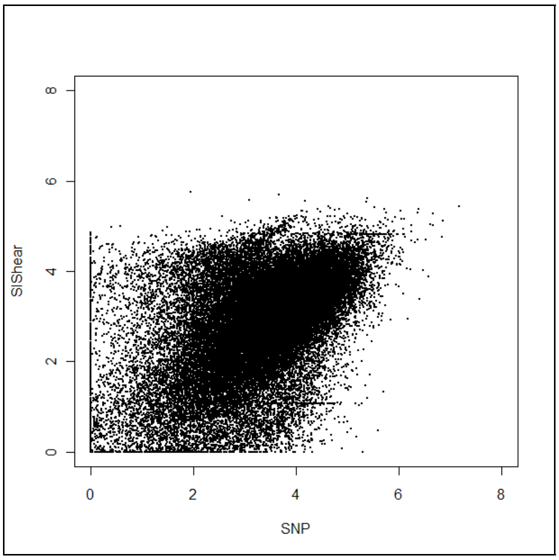
Structural index for rutting VS SNP



Structural index for roughness VS SNP



Structural index for flexure VS SNP

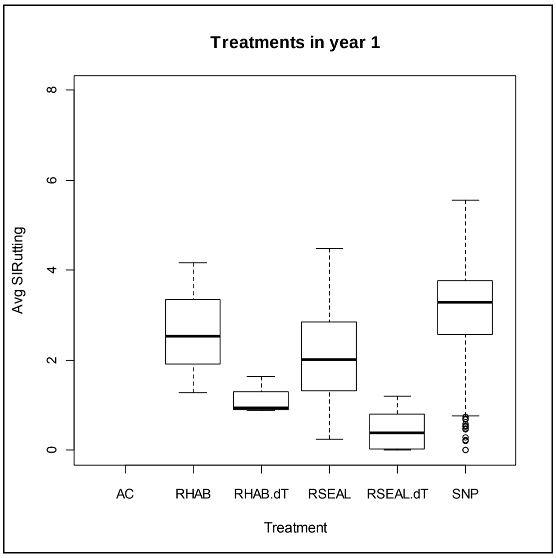


Structural index for shear VS SNP

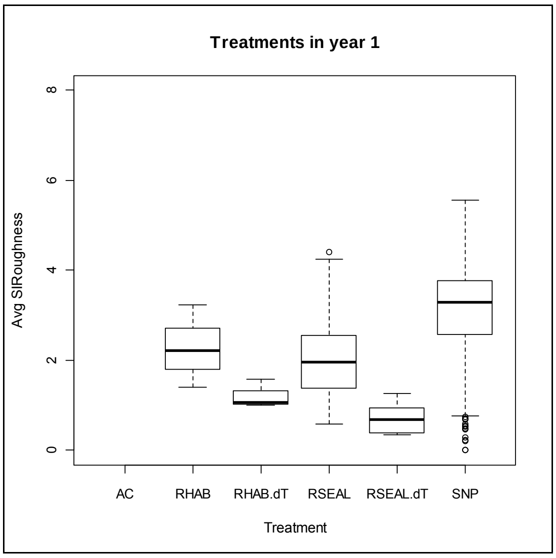
4.2.2 Comparing indices with maintenance decisions

The index values of the SDC were compared with the actual maintenance decisions for the network. Note that the network maintenance decisions were based solely on the existing condition data and no inputs from either dTIMS or the structural indices were utilised. A summary of these comparisons is presented in figure 4.6. For these comparisons, reseals and rehabilitation identified during the first year were compared with the corresponding four index values. As an extra benchmark the identified maintenance forecast from the dTIMS system is also presented. Note that the last box plot presents the SNP distribution for the respective sections.

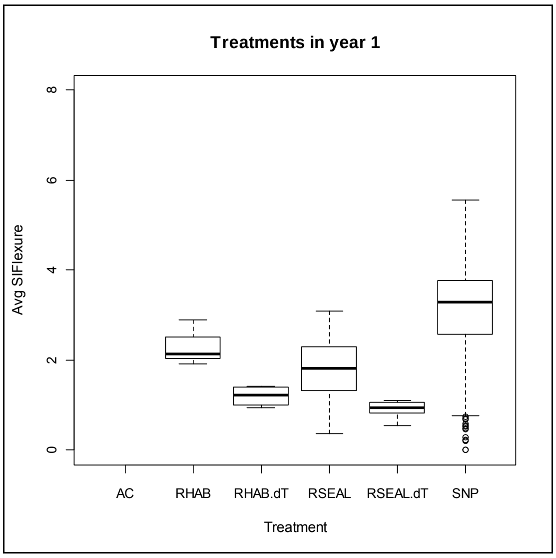
Figure 4.6 Comparing maintenance decisions and structural indices



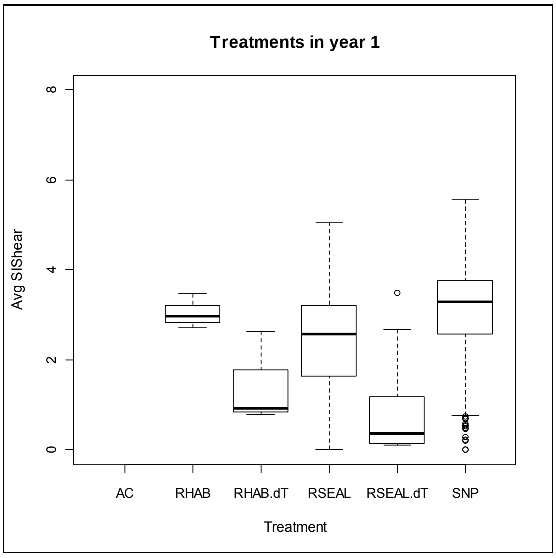
Maintenance compared with rutting index



Maintenance compared with roughness index



Maintenance compared with flexure index



Maintenance compared with shear index

The following observations were made from the comparisons:

- The rehabilitation and resurfacing sections were undertaken at relatively lower index values when compared with the corresponding SNP value.
- It was of concern that the resurfacing treatment was undertaken on sections with lower index values.
- The dTIMS treatments were consistently identified at lower index values, compared with the actual maintenance decisions.
- The roughness and rutting indices had relatively similar results.

These results confirmed that the field decision process could be more effective by incorporating both modelling results (dTIMS) and the structural indices. It would especially be helpful to identify sections that would need rehabilitation rather than a resurfacing treatment, which currently happens through not taking modelling or structural index information into consideration.

4.3 Using the indices as an indicator of failure probability

In essence, the structural indices indicate a pavement's ability to carry traffic loading for each failure mode under consideration. This knowledge contributes to defining failure probability in two areas, by:

1 allowing for easier identification of the most likely failure mode

2 giving an estimate for the remaining capacity of the pavement.

Earlier research has indicated that there is an increasing trend to predict the probability of failure rather than a straight prediction of the deteriorating condition measures. There are two reasons why an understanding of failure risk adds useful information to the asset manager:

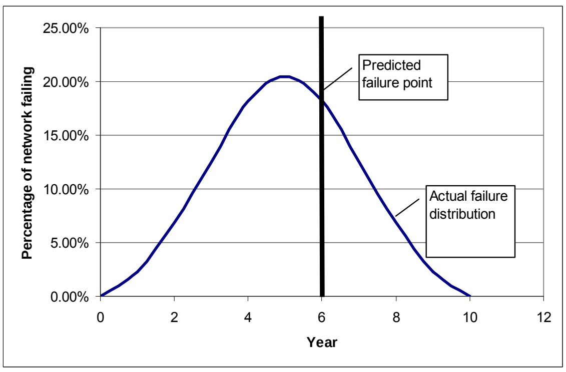
1 There is an increased demand on New Zealand roads for a traffic loading that is much higher than the design traffic. For example, in some rural areas there is a loading increase of around 20% which is attributed to a switch from traditional sheep farming to dairy farming. Some pavements are able to cope with these increased loadings better than others. The ability of pavements to sustain higher loads could best be defined by failure risk.

2 Given the variability of pavements, materials and construction quality, there is also a corresponding significant variation in pavement failure. For this reason, the use of probabilistic models is becoming more widely used. See the extract below:

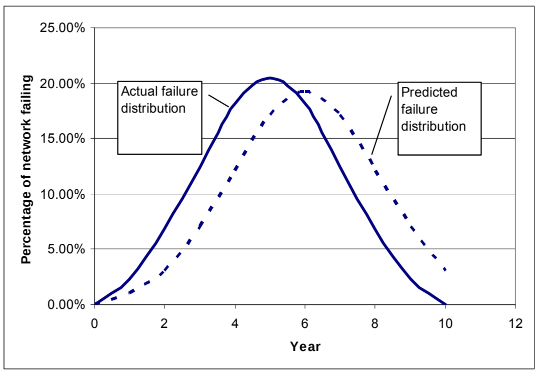
In his research Henning (2008) explains why probabilistic models give more realistic answers compared with discrete deterministic models:

> *The very nature of the probabilistic model will make it more accurate in terms of predicting failure behaviour for road sections. Figure 4.7 illustrates two predictions, the first predicts 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.*

Figure 4.7 Comparing predicted failure versus actual behaviour



Predicting a discrete failure point



Predicting a failure distribution

Both the crack initiation and rutting model were tested on network data and remarkably 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 with the actual cracked status.

4.3.1 Defining failure risk for pavements

The classical definition of risk is:

$$Risk = Probability\ of\ Event \times Consequences \qquad \textbf{Equation 4.1}$$

In the context of this research this formula is re-written as:

$$Risk\ of\ Pavement\ Failure = Probability\ of\ Failure \times Likely\ Maintenance\ Cost\ of\ Failure \qquad \textbf{Equation 4.2}$$

As indicated in section 1.3, the research also investigated the potential of using the indices to forecast the risk of pavement failure. The probability of failure would be the most challenging to establish.

4.3.2 Theoretical approach towards developing failure probability

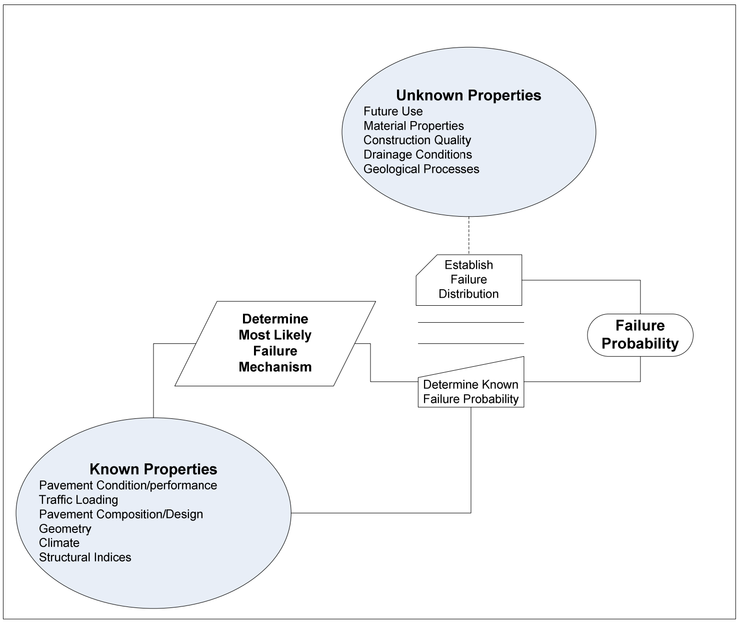
The probability of failure is a function of a number of factors including:

- pavement design and composition
- material used in the pavement layers and surfaces
- construction quality
- drainage provision and consequential moisture within the pavement
- traffic loading on the pavement
- environmental and topographical factors
- maintenance regime on the pavements
- age of pavement and/or existing defects.

Some international work such as Salem et al (2003) and Yang (2008) have started developing different approaches towards predicting failure probability. Although these publications indicated some promising results, there is a concern regarding the practicality for New Zealand application given the comprehensive data requirements for developing failure probability.

The approach recommended for New Zealand needs to make use of existing data such as structural parameters and pavement performance data. These data items combined with the appropriate modelling approach will yield a practical probability to failure and risk index. This approach is schematically indicated in figure 4.8. This figure shows there is a part of the failure risk that can be predicted using empirical data. Then there is a component that would only consist of a statistical model that would forecast failure according to main expected failure distribution.

Figure 4.8 Development of a failure probability model for New Zealand



4.3.3 Preliminary result on risk index development

Schlotjes (2009) has started with the development of a pavement failure risk index. For this development both the SDC data and the LTPP data were used. Given that this is a comprehensive three-year research project, only some preliminary results are available. At this stage some of the main findings are:

- There is a strong link between some physical geometry attributes and distress mechanisms. This applies especially to narrow roads that experience more water ingress from the shoulders compared with wider roads.
- Pavement deterioration shows good correlation with expected input parameters such as the structural indices and traffic loading.
- There is a poor correlation between maintenance programmes, and structural indices and current pavement condition.
- The structural indices show promising results in forecasting condition and failure. However, some of the indices need further development and refinement.

5 Findings and recommendations

Structural number concepts originated well before mechanistic analysis procedures became readily available to practitioners. The reason SNP can give an approximate indication of possible structural deterioration for a large network is that the progression of many distress modes will generally be deferred by improved load spreading (subgrade strain distribution). However, most of the techniques used to model this are based on a general indicator of strength that is derived from layer thickness and material quality. Therefore, SNP is not able to give any indication of how a particular pavement structure would behave for a given layer configuration. For example, a road consisting of a stabilised base on top of inferior material may have a high SNP, but would in fact fail rather quickly due to cracking of the base layer.

Mechanistic appreciation of pavement structural performance, which is the aim of the American approach (NCHRP), is not yet at the stage where reliable models for progression of all distress modes in all materials are available. Advances in that research should be continuously followed, as that should eventually lead to the most effective procedures for rational design. Meanwhile an improvement to the indirect SNP concept is required. An interim solution for practitioners is to utilise mechanistic procedures when deriving the fundamental structural parameters for network modelling.

As a replacement for SNP, an alternative structural parameter, termed structural index has been proposed. For each of the currently recognised structural distress modes (ie rutting, roughness, flexure and shear) a corresponding structural index is required. This study provides the basis for structural indices for rutting and roughness.

The rutting index already has a substantial basis from APT and LTPP data. However it requires further calibration as LTPP sites age, or as specific roads with known rutting performance and past traffic are identified as suitable candidates for reliable calibration.

The roughness model is provisional only because no significant change in roughness has yet developed on LTPP sites. However the model has been tentatively calibrated assuming all the LTPP sites began life with minimal roughness, and that their past traffic has been realistically recorded. An ongoing study is investigating structural indices for flexure and shear. The flexure model is advancing to a moderately reliable stage, while the shear model is still in the early stages of development.

Each structural index is mechanistically derived and has the same range and general distribution as the traditional SNP, allowing straightforward implementation (substituting the relevant structural index for SNP) with minimal additional calibration needed for existing HDM/dTIMS asset management systems.

As the amount of data from LTPP sites grows, the improved mechanistic understanding of pavement performance can be readily incorporated, by refining (or redefining the basis of) the structural index for each distress mode. Provided the base (raw) data remains stored in RAMM, updated structural indices may be readily generated at any future time for any network.

5.1 Further work required

Further development and refinement work required are summarised in table 5.1.

Table 5.1 Further development and refinement work on indices

Item	Description of further work required	Data source/methodology
$SI_{rutting}$	Minor refinement. Calibrate to those regions with subgrades known to perform anomalously (eg Taranaki Brown Ash and Central Plateau ashes.	Roads or networks with well known performance (rutting distress and known past ESA)
$SI_{flexure}$	Wider calibration particularly to different surfacings (AC versus OGPA versus multiple seal layers)	Project level testing of terminal sites
$SI_{roughness}$	Major refinement, as this is an important yet the most difficult parameter to characterise.	The challenge is to find roads that have not been complicated by unknown past maintenance or 'non-traffic' damage (eg service trenches)
SI_{Shear}	Separation of shear instability: • beneath AC surfacings • beneath thin seals on unbound basecourse • within multiple seal layers	Project level testing of terminal sites
Pavement prediction models	This research has demonstrated that pavement prediction models need to be re-developed/refined from first principles if new indices are incorporated	LTPP and some limited network data.
Network applicability	Extend the range of the indices by conducting more tests on other networks	Do this as part of the over-all network testing programme
Pavement modelling	Investigate further adoption of the indices within the dTIMS system. For example, it may well be utilised as triggers and additional reporting measures within the system	Deliver the structural indices to the modelling community for further investigation.
Risk index development	The indices promise a significant value to defining a risk index. Fundamental development work needs to occur in this area.	Development needs to be based on a combination of network, LTPP and CAPTIF data

6 References

AASHTO (1986) *AASHTO guide for design of pavement structures.* Washington, DC: American Association of State Highway and Transport Officials.

AASHTO (1993) *AASHTO guide for design of pavement structures.* Washington, DC: American Association of State Highway and Transport Officials.

ARA Inc and ERES Consultants Division (2004) Guide for mechanistic-empirical design of new and rehabilitated pavement structures. National Cooperative Highway Research Program final report. *www.trb.org/mepdg/guide.htm*

Austroads (1992) *Pavement design: a guide to the structural design of road pavements.* Sydney: Austroads.

Austroads (2004) *Pavement design: a guide to the structural design of road pavements.* Sydney: Austroads.

Austroads (2006) Investigation of the load damage exponent of unbound granular materials under accelerated loading. *Austroads technical report AP-T73/06*.

Austroads (2009) *Guide to pavement technology series*. Sydney: Austroads.

Cochran, WC (1997) *Sampling techniques.* 3rd ed. New York: John Wiley.

Dawson, A (2002) The mechanistic design and evaluation of unsealed and chip-sealed pavements. University of Canterbury workshop briefing paper. Accessed 25 April 2010. *www.pavementanalysis.com/papers/documents/pavementsworkshop02/briefing.pdf*

Gray, W and G Hart (2003) Recycling of chip sealed pavements. *Paper to PIARC World Road Conference, Durban, South Africa*. October 2003.

Henning, TFP (2008) The development of pavement deterioration models on the state highway network of New Zealand. A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Engineering, University of Auckland. Available from *http://researchspace.auckland.ac.nz/handle/2292/4236*

Henning, TFP, WHRM Abeysekera, K Paudel and V Ramisheswar (2008) Understanding the long-term performance of ACC's asphalt surfaced sections. *NZIHT Conference, Napier*.

Henning, TFP, SB Costello, RCM Dunn, CC Parkman and G Hart (2004) The establishment of a long-term pavement performance study on the New Zealand state highway network. *ARRB Journal 13, no.2.*

Henning, TFP, SB Costello and TG Watson (2006) A review of the HDM/dTIMS pavement models based on calibration site data. *Land Transport NZ research report no.303*. 123pp.

Henning, TFP and DC Roux (2008) Pavement deterioration models for asphalt-surfaced pavements in New Zealand. *NZ Transport Agency research report no.367*. 56pp.

Highway Research Board (1961) *The AASHO road test: report 1, history and development of the project.* Special report 61A. Washington DC: National Academy of Sciences.

Hoque, Z, T Martin and L Choummanivong (2008) Development of HDM-4 road deterioration (RD) model calibrations for sealed granular and asphalt roads. *Austroads Project No. AT 1064.*

HTC Infrastructure Management (1999) *dTIMS technical reference manual*. Volume I. Auckland: HTC.

Kerali, HGR (2000) *Overview of HDM-4. Volume 1 of the Highway Development and Management Series.* Paris: PIARC and Washington: The World Bank.

Kancherla, A (2004) Resilient modulus and permanent deformation testing of unbound granular materials. Master's thesis, Texas A&M University.

MHW New Zealand Ltd and Beca Carter Hollings & Ferner Ltd (2003) *Safety related surface issues.* Wellington: Transit New Zealand.

Moffat, MA and GW Jameson (1998) Characterisation of granular material and development of a subgrade strain criterion. *ARRB Transport Research WD R98-005.*

NDLI (1995) *Modelling road deterioration and maintenance effects in HDM-4*. Final Report ADB BETA 5549, Vancouver, BC.

Opus International Consultants Ltd (2008) *Pavement performance modelling dTIMS report.* Auckland: Manukau City Council.

Rada, GR (1996) *SHRP – LTPP monitoring data, 5-year report. SHRP-P-696.* Washington: FWHA.

Rossiter, DG and A Loza (2004) *Technical note: analysing land cover change with logistic regression in R.* 1.3 ed. Enschede: International Institute for Geo-information Science & Earth Observation.

Salem, O, S AbouRizk and S Ariaratnam (2003) Risk-based life-cycle costing of infrastructure rehabilitation and construction alternatives*. Journal of Infrastructure Systems 9,* no.1: 6–15.

Salt, G (1999) Determining the structural capacity of unbound granular pavements in New Zealand using deflection testing. Report for Transfund NZ. Tonkin & Taylor Ltd, New Zealand.

Salt, G and D Stevens (2005) Performance based specifications for unbound granular pavements: procedures for demonstrating achievement of design life. *7th Annual NZIHT/TRANSIT NZ Conference, Christchurch*.

Schlotjes, MR (2009) The risks associated with pavement failure in New Zealand. PhD Dissertation. Unpublished. University of Auckland.

Stevens, D (2006) MET (ELMOD) strain criterion – equivalent relationship for the Austroads (EFROMD2) subgrade strain criterion. *www.tonkinandtaylor.com.au/documents/pavementsworkshop06/backanalysis.pdf*

Theyse, HL (2002) Stiffness, strength and performance of unbound aggregate material: application of South African HVS and laboratory results to California flexible pavements. *California Partnered Pavement Research Program Report.* 86pp.

Tonkin & Taylor Ltd (2006a) *Deflection trend study, SH1 RP 746.* Transit New Zealand.

Tonkin & Taylor Ltd (2006b) Structural evaluation of Transit New Zealand's LTPP sites 1999–2006. Draft Report to Transit NZ.

Tonkin & Taylor Ltd (2007) Notes for the development of a rutting progression model for NZ unbound granular pavements. Internal report 891045.2.

Tonkin & Taylor Ltd (2007a) Life prediction for unbound granular pavements using strain criteria for the granular layers as well as the subgrade. Transit NZ LTPP study. Dunedin.

Tonkin & Taylor Ltd (2007b) Background to the development of a rutting progression model for NZ unbound granular pavements. Transit NZ LTPP draft study. Dunedin.

Tonkin & Taylor Ltd (2007c) Re-analysis of permanent deformation at CAPTIF using non-linear elastic theory. Transit NZ LTPP study. Dunedin.

Tonkin & Taylor Ltd (2007d) Permanent deformation in New Zealand unbound granular pavements. Transit NZ LTPP study. Dunedin.

Tonkin & Taylor Ltd (2008a) Modelling of unbound granular pavements using adjusted structural number (SNP) determined from permanent deformation characteristics. Transit LTPP study.

Tonkin & Taylor Ltd (2008b) Roughness progression of unbound granular pavements at long term pavement performance sites. Internal report. Dunedin.

Transfund NZ (2002) *Maintenance guidelines.* Wellington: Transfund NZ.

Tseng, KH and RL Lytton (1989) Prediction of permanent deformation in flexible pavement materials. *Implication of aggregates in the design, construction and performance of flexible pavements. ASTM STP 1016*: 154–172.

Ullidtz, P (1987) *Pavement analysis.* Elsevier.

Ullidtz, P (1998) Modelling flexible pavement response and performance. Pavement analysis. In *Developments in Civil Engineering 19*: Narayana Press, 1998.

Ullidtz, P and H Larsen (1998) *Development of improved mechanistic deterioration models for flexible pavements.* Danish Road Institute.

Yang, J (2008) Estimating failure probabilities of flexible pavements under competing risks. *Proceedings of TRB 87th Annual Conference.*

Vorobieff, G (2005) Design of foamed bitumen layers for roads. Accessed 25 April 2010. *www.auststab.com.au/pdf/tp44.pdf*

Watanatada, T et al (1987) *The highway design and maintenance standards model vol 1. Description of the HDM III model.* Baltimore: John Hopkins University Press.

Appendix A: Best practice guidelines

FWD surveys – sampling techniques

How much is enough data?

Although some authorities are in a position to conduct a falling weight deflectometer (FWD) survey of their entire network, it is not possible for others. Two questions arise:

- How much of my network should be surveyed, ie what length of the network needs to be covered?
- Once the length to be sampled has been established, what sampling frequency will be required on each road section?

Unfortunately, there are no right or wrong answers to the questions above since they largely depend on a number of factors including:

- **The ultimate use of the data**. For example, for asset management purposes a less frequent sampling on certain parts of the network may be acceptable, whereas for design purposes we need detailed sampling for each road.
- The **existing knowledge of the network**. Some authorities are privileged to have skilled and experienced maintenance engineers. Although these engineers utilise applications in order to do optimal maintenance planning, they know the performance of their network and associated issues very well. Utilising this knowledge may allow a more targeted sampling process that could drastically reduce the overall sampling size.
- The **extent of the current network**, plus how much of it has been sampled before.
- The **availability of funding**. Obviously, more available funds will allow more sampling and better information may result from it. However, in most cases, asset managers have to do the best they can within the funding levels.
- **Sophistication of use**. Some authorities already have good quality data for other items and use FWD results with vigour including a dTIMS modelling process. For these authorities a more comprehensive sample is advisable, whereas authorities still basing their evaluations on simplified methods may not need such a comprehensive survey.

The following section recommends a process for selecting the appropriate sampling regime for different situations.

Selecting the appropriate sample for a network

The factors mentioned in the previous section will determine the overall sampling size of the network. This may result in say 5%, 10%, 20% or 50% FWD test coverage of the whole network. A decision also has to be made on which sampling technique to use for a FWD survey. Some of these techniques include (Cochran 1997):

- Random sampling - a number of roads or the length of the network are selected for FWD testing without any pre-defined considerations.
- Systematic sampling – a number of roads are selected according to a set process. For example, we may select all roads with even road IDs. This method was used in the past when all roads with AADT

above 2500 were surveyed. This approach was adopted since the HDM-III models used at that stage were sensitive only to those traffic volumes.

- Clustering sampling –a network may be surveyed based on, for example, historical wards or barrows.
- Stratified sampling – the entire network is classified according to given factors such as traffic loading, hierarchy, geology type, or development history. Once the network is classified into the defined matrix a random sample is taken from each cell of the matrix.

The most effective method of sampling New Zealand roads would be to use the stratified method combined with either random or systematic approaches for the final road selection. This method is to be used according to the following steps:

Step 1: Understanding performance of your network

Either through detailed statistical analysis of the network or by utilising existing knowledge of the network you can define a number of factors that drive the performance and maintenance planning of the network. Some of these factors may include:

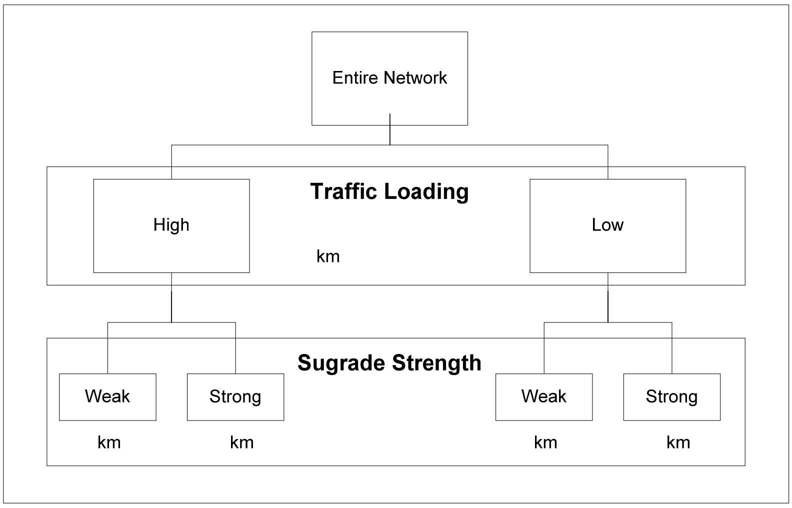
- geological make-up of the network
- topography
- historical development of the network (for example there may be parts of the network which were built according to a different design or construction approach)
- in-situ subgrade conditions
- rainfall
- traffic loading
- pavement types
- dominant failure patterns.

The secret of this step is to choose three to five factors that have the greatest effect on the behaviour of pavements and consequential maintenance decisions.

Step 2: Stratify the network according to chosen factors

At this stage it may be useful to start using a map (preferably a GIS map) and colour code the roads according to the chosen factors. Once this process is completed the resulting length for each cell should be calculated. Figure A.1 gives an example outcome for this approach.

Figure A.1 Example stratification matrix of a network



Step 3: Choose the sampling size and further selection method within a cell

Choose the overall sampling size according to the factors listed on page 47. Remember that the smaller the sample size, the less likely it is to represent the overall network. Experience has indicated that the sampling size should be at least 25% to 50% in order to be a good indication of the overall network status.

The overall sampling size (say 25%) is assigned to each of the classification cells indicated in step 2. This can be done by means of either a random or systematic methods.

Step 4: Assign the road sections to be surveyed

Based on the information from step 3, it is now possible to assign testing to specific roads and start the survey. Note that the actual survey frequency for each road has to be conducted according to the following section.

Interval of FWD measurements

The following sets out procedures which may be adopted for standard structural testing (with FWD), and also for an alternative technique of varying the spacing of test points depending on the length of the road and nature of the terrain.

Testing may be carried out on:

- unsealed roads
- unbound granular pavements with chipseal surfacing
- structural asphalt surfacing.

The procedure is only applicable for asset management purposes of roads which are not due for immediate rehabilitation. Where a budget for standard testing is available, see table A.1 for a recommended spacing of FWD test points:

Table A.1 Recommended frequency of measurements for FWD surveys

Centreline length of 2-lane road	FWD test spacing
0-200	**5 tests (3 IRP lane, 2 DRP lane)**
200-500	**100m intervals in each lane**
500-2000	**10 tests in each lane**
>2000	**200m intervals in each lane**

Data storage, processing and reporting

The additional storage now required in RAMM is the four additional fields for the structural indices. Customarily, much of the data stored for FWD testing has been limited to peak stress and deflections. The remainder of the full-time history captured at the time of the test has been discarded. There is no extra effort required to store the raw data (known in the industry as the 'full-time history'. This file gives the full wave pattern induced by the loading and there are 'signatures' within it that are now becoming recognised as being distinctive of shear instability. For this reason all requests for FWD testing should include a requirement for provision of the full-time history record. However the files are large and at present there is little need to store other than the peak data in RAMM as the fields for date of collection and testing contractor can be used to source the raw data, should the structural indices need to be re-evaluated. In practice, there is no significant technical reason to store the peak data in raw form as opposed to standardised form (corrected for a plate stress of 575kPa), as suits the data manager.

Processing to generate the indices simply requires a recognised transfer function as described in section 3.1 and it is important to appreciate that regions and network around the country are demonstrably different (particularly in relation to rutting behaviour which is controlled by the different types of natural subgrades).

Reporting needs to state the basis of derivation, eg if other than the Austroads subgrade strain transfer function is used for the rutting model then that should be supplied. In this way future revisions can either adopt the status quo, or seek a more appropriate relationship, once more regional data become available for calibration.

Application of structural capacity parameters

Based on the findings of this report the recommended application areas for the structural indices are discussed in the following sections.

Network level rehabilitation diagnostics

The original intention in developing the structural indices was to provide a structural capacity parameter that could be used in asset management processes. One of the significant outcomes of this research was the framework for having four separate indices that represented the four main failure mechanisms of New Zealand flexible roads. This allows the indices to be used intensively for making more informed decisions on the maintenance regime for given road sections. Table A.2 contains the decision framework for using the structural indices.

Table A.2 Using the structural indices to Identify appropriate maintenance actions

Index/indices	Possible mechanism/extra information	Potential remedial action
	For low values of individual index	
$SI_{rutting}$	There is a possibility that the subgrade may be overstressed or that the compaction of the layers are not sufficient. Also consider actual defect data including rutting and cracking information.	The pavement needs to be strengthened. Options include: • overlay • recycle existing material (preferably adding some additional depth).
$SI_{flexure}$	There is potential for layers with tensile capacity in the pavement to become distressed. This may lead to further defects. If the pavement does not contain layers with tensile capacity, further investigations are needed to find the cause of the problem.	With sufficient strength in the pavement, a resurfacing may be considered to keep the pavement watertight. For distressed bound layers in situ, stabilisation may be considered. Make sure that the lower pavement layers provide sufficient support for stabilised layers.
$SI_{roughness}$	There is significant variation in structural parameters taken along the length of road. This variation may result from consolidation of the pavement layers or some in-situ soil variations of the subgrade.	Only when the roughness gets above recommended levels, should any maintenance be considered. Therefore monitor roughness progression and fix any local unevenness using normal routine maintenance practices.
SI_{Shear}	The base layer displays shear issues. It is expected that localised distress such as cracking, shoving and potholes will start forming.	Monitor actual occurrence of defects. Once maintenance cost increases, rehabilitation needs to be considered. This may include in-situ recycling and/or overlays
	Combined indices with low values	
$SI_{flexure}$ & $SI_{rutting}$ &/or $SI_{roughness}$	The pavement is at an advanced stage of failure. The subgrade is over-stressed and some of the layers are starting to show distress also.	Full rehabilitation options
$SI_{flexure}$ & SI_{Shear} &/or $SI_{roughness}$	The base layer is at an advanced stage of distress	Repair base layer - more detailed testing and investigation may be needed to determine whether the material in the base layer can be utilised in some form or other.

Pavement modelling and network reporting

It has been demonstrated that the introduction of structural indices to the pavement deterioration models would improve the overall status and usefulness of the models. However, it is recommended that the structural indices are further explored to judge their usefulness for:

- use as intervention/trigger mechanisms
- additional reporting measures to give an indication of network pavement capacity. For example, the current system contains a granular overlay need which is a function of the SNP and the traffic loading. Something similar could be developed for the structural indices.

The last point is also of interest to overall network performance monitoring and reporting. According to theoretical definition, the SNP does not change over time. However, the deflection results of roads do change over time, which may lead to a change in SNP.

The structural indices differ fundamentally from this approach. It is expected that the structural indices will change over the life of the pavement as it is a function of the failure mechanism assumed during the design. Therefore, the structural indices would be an excellent indication of the structural capacity of the network and how it changes over time. It is further recommended that the four indices are reported separately as each will convey a different aspect of the network performance.

A process of refining the strength indices

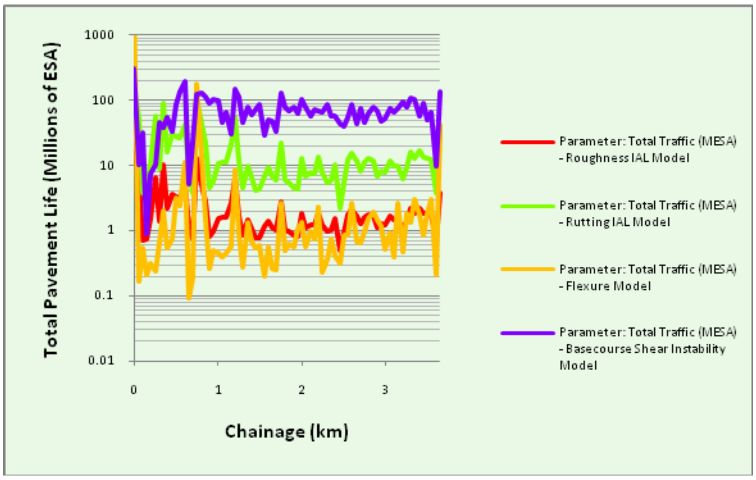
The key to refining the strength indices is to first test the concept at project rather than network level. That is, finding specific treatment lengths that have each reached a terminal condition, and then testing the indices to see how well the predictions for individual distress modes fit with observed performance. The recommended process for this is:

1. Identify a treatment length in a terminal condition that (a) has comprehensive condition data contained in RAMM (including HSD rutting/roughness and deflection data), and also (b) can be discussed with a pavements engineer who is closely familiar with its historic performance and any intervention taken since first construction.
2. Assume (if necessary) reasonably expected values for initial rutting and roughness.
3. Use the current fatigue criteria (or other parameters) to find predicted life for each distress mode.
4. Review the predicted with the observed condition together with the pavement engineer familiar with its historic performance.

After applying intensive test data to a series of project sites, validate or refine the indices to ensure consistency of observed performance with the structural index.

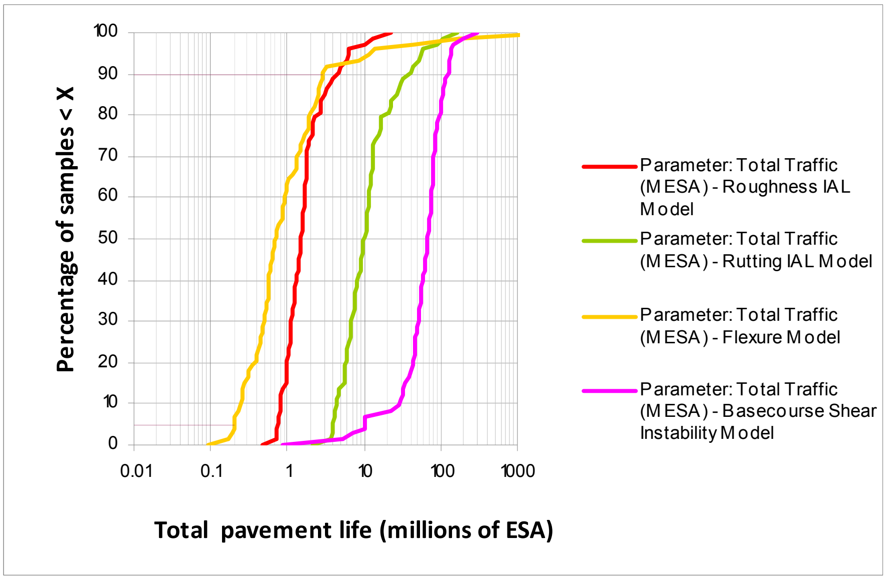
As an example, figure A.2 shows the expected total life (in terms of millions of equivalent standard axles (MESAs)) for a pavement using the structural index modelling. This is a recent case history of premature failure on a section of SH2. Each of the four failure modes is shown, but clearly it is only the lowest of these graphs that is relevant, and would need to be discussed with the local pavement engineer to ascertain a full picture. This would include determination of whether the predictions are accurate in absolute terms (should any graph be translated up or down), and in relative terms (ie do the chainages where greater severity of distress is expected, coincide with actual observed performance), and also allow consideration of historic knowledge that may not be included in specific data collected.

Figure A.2 Example distress mode analysis for validation of the PPM



The same data can be viewed as a cumulative distribution to quantify the critical distress mode at the level of interest to the NZTA (usually the 95% reliability model, ie the number of ESA that will result in only 5% of the pavement reaching a terminal condition).

Figure A.3 Example of life predictions (the left graph is the most critical)



In this case it has been reported that at least 90% of the pavement has failed. From the above chart (see uppermost red arrow), the PPM predicts that the life of the pavement would be about 3 MESA with flexure (cracking) being the principal distress mode. This estimate is in the right order (prior to any refinement or calibration) with the observed ESA.

It is also of interest to note that (had the pavement not failed through cracking) roughness would have eventually limited the life of this pavement, rather than rutting or shoving which the pavement performance model indicates are essentially not critical (green and purple graphs).

Appendix B: Interim APT-LTPP model of pavement performance – principal elements of the structural mechanistic model

1. Austroads 2004–2009, AASHTO 1993, Ullidtz 1998 are the starting points for general concepts of pavement performance. To be pragmatic a pavement performance model must be 'no more complicated than it has to be' (Major, pers comm), and ideally should be flexible enough to be used either for quick preliminary scoping (where warranted because of economic or other limitations) or for comprehensive evaluation where the value-added component has a clear cost benefit.
2. In any given region (where a specific form of pavement construction has been pursued), APT currently provides the most relevant source of data for modelling early life performance under controlled environmental conditions. APT usually continues to be a reliable predictor to at least 1 MESA, the practical maximum often used in testing.
3. Long-term pavement performance (LTPP) sites are providing a growing database of *mid-to-late* life in-service performance, allowing continuous improvement of an APT-LTPP model.
4. *Prime* LTPP sites now allow reliable linking of APT and LTPP data. Prime sites denote those that are monitored from the time of first construction, with initial conditions and parameters measured as soon as the surfacing is in place but *prior* to any trafficking.
5. The APT studies use vertical surface deformation (VSD) as a primary measure of deterioration hence this is an appropriate basis for primary rutting progression and to a lesser extent for roughness progression. Rutting and roughness from high-speed data, provide a less accurate but convenient measure of existing condition for in-service pavements. However, rut depth cannot be directly correlated to VSD because of the unfixed origin of the multi-laser profilometer which adopts a 2m virtual straight edge. In wide lanes or where traffic wander is considerable, the virtual support points themselves experience occasional trafficking and therefore reported rut depth is *less* than VSD. Conversely in narrow lanes or where traffic wander is minimal, trafficking of the virtual support points is rare and lateral deformation effects are likely to cause slight dilation away from wheelpaths resulting in elevation of the support points. In this case, reported rut depth from the laser profilometer will be *greater* than VSD. The difference between VSD and reported rut depth from high-speed data must be rationally accounted for when models are derived from APT data.
6. Rutting progression rates are not constant for a given pavement profile, but are fundamentally related to elapsed ESA since construction, as well asto the structural stiffness of *all* component layers. This characteristic is now clearly demonstrated by the LTPP sites (Tonkin & Taylor 2007).
7. Roughness progression is related fundamentally to not only VSD and loading but also to the structural capacity at each point and structural uniformity along each wheelpath. Therefore it follows that to obtain a meaningful model for roughness progression, treatment lengths need to be separated into discrete lanes, rather than encompassing the full road width. However, if roughness is not a driver for rehabilitation, then lane separation is not essential.
8. Structural uniformity that relates to roughness progression is a function of the local (point to point) variability of the pavement stiffness along the wheelpath, not to a steady uni-directional change in stiffness and hence not necessarily related to standard deviation as used in the HDM and NCHRP models.

9 It is important to note that more than 60% of LTPP sites have subgrade moduli that cannot be sensibly modelled as linear elastic. It is now widely recognised that unbound pavement materials, particularly subgrades with CBR< 7, have non-linear moduli, ie they may exhibit strain-dependent behaviour (stress hardening or stress softening) and such characteristics must be correctly modelled or both back-calculated and forward-calculated parameters will be invalid. Approximating non-linear moduli by attempting to use a linear elastic model with sub-layering will give not give a realistic pavement model (Ullidtz 1999).

10 There is no practical measurement procedure to determine anisotropy of in-service pavements. Worldwide, the majority of pavement models (including HDM-4) are based on isotropic moduli. Therefore to be able to be able to draw on other international research, the most pragmatic approach (keeping the model 'no more complicated than it has to be') is to retain the concept of isotropic moduli. Anisotropy promoted by Austroads is counter-productive, but the overriding consideration is that it is needlessly out of step with worldwide practice.

11 Pavement structural life is governed not only by strains at the top of the subgrade but also by strains at depth within the subgrade and within each pavement layer. Observations from APT confirm that reliance on a subgrade criterion alone, can give errors of over two orders of magnitude if predicting the number of ESA to a terminal rutting condition. An extended compressive vertical strain criterion can be readily adopted that includes both the standard subgrade strain criterion and, based on LTPP data, suitable strain criteria from all individual pavement layers.

12 A pavement design chart (eg Austroads 2004, figure 8.4) that has provided reliable empirical design in the past should be a reliable basis for back-analysing an appropriate subgrade strain criterion. This process is also applicable for determining appropriate strain criteria for the overlying layers, provided proven modular ratios (as promoted by Austroads) are used for unbound granular materials. Back-analysis results can be unconservative, hence for all cases checks need to be made with respect to LTPP sites that exhibit substantial wear and have reliable estimates of past traffic (ESA).

13 Flexure under traffic loading and its associated horizontal tensile strains require consideration for all bound structural layers, including layers that are often regarded as non-structural such as porous asphalt and sprayed seals where the maximum tensile strains are often found to be at the top of the layer (if it is thin), as opposed to the case of thick structural asphalt when the maximum tensile strain is invariably at the bottom of the bound layer.

14 The inclination of the road, speed of traffic and braking/cornering stresses require consideration when assessing flexural life of thin asphalt or sprayed seal layers. The standard 1ESA loading should in this case include the relevant tractive horizontal shear stresses as well as the normal vertical loading. Flexural life of porous asphalt is generally dictated by ravelling which is considered to be related to both traction and flexure.

15 Where unbound basecourse forms the top layer and is not protected by structural asphalt, life will be limited by degradation of the aggregate which will eventually lead to shear instability.

16 Newly constructed unbound granular pavement layers compacted using best practice to accepted standards, will increase in stiffness markedly during early life trafficking, particularly in the first year or at least the first 20,000 ESA. The degree of stiffness improvement and early life rutting may be predicted from the ratios of layer moduli (relative to Austroads expectations) measured at the time of construction.

17 Unless subject to variable watertable levels that come close to the top of a fine-grained subgrade, pavements that are adequately protected by watertight surfacings soon tend to reach an equilibrium

condition as far as stiffness is concerned. Subsequent changes in stiffness due to seasonal changes then tend to be minimal. Until either shear instability develops or cracking is initiated, the structural stiffness of all pavement layers will not reduce markedly even with prolonged trafficking.

18 A small change in equilibrium water content of an unbound granular layer will induce a large change in the rate of progression of permanent deformation. In particular, from laboratory repeated load triaxial testing, an increase of just 1% in water content (above the optimum) in a basecourse can increase the rutting rate (per load cycle) in that layer by an order of magnitude. Similar sensitivity is inferred from in-service pavements. For this reason a relevant environmental parameter is fundamental for any pavement deterioration model and also for reaching conclusions regarding the potential in-service performance after repeated load testing of non-standard aggregates. Adoption of the Thornthwaite Index as promoted in both New Zealand and Australia appears the most promising parameter at present. Equilibrium water content fluctuation is, and is likely to remain, the primary reason for the inherent difficulty in predicting pavement performance. Repeated load triaxial testing has its value in the assessment of the relative performance of materials in the laboratory, but results must be used with caution when assessing absolute performance or parameters for in-service pavements.

19 Pavement material parameters seldom exhibit a normal (Gaussian) distribution. Adopting relevant percentiles for characterising design parameters avoids the need to assume any particular distribution and is therefore a practical course.

20 Appropriate selection of treatment lengths on the basis of structural homogeneity is critical for rational prediction of deterioration and forward work programmes. Initial treatment lengths chosen on the bases of surfacing type must subsequently be refined by sub-sectioning on the basis of structural uniformity (as required by the NZTA, but seldom found in practice).