Pavement Performance Prediction: A Comprehensive New Approach to Defining Structural Capacity (SNP).

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Stevens D.[1]; Salt G.[1]; Henning T.F.P.[2] & Roux D.C.[3]

1. Tonkin & Taylor Ltd

2. The University of Auckland

3. MWH New Zealand Ltd., Auckland

ABSTRACT

Pavement performance modelling for New Zealand (NZ) roading networks currently relies on Adjusted Structural Number (SNP); 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 1950’s before the advent of analytical methods. Refinement to keep abreast of current practice in pavement engineering is long over-due.

An advanced model for pavement structural capacity is under development overseas (NCHRP), but it is not in general use because of its complexity (including requirements for destructive test information). The focus of a recent NZ Transport Agency (NZTA) research study has been to set the framework for obtaining the most practical indices for New Zealand pavements based on parameters which are currently stored in RAMM, while at the same time maintaining flexibility for ongoing upgrades that might utilise future developments in the way of pavement data collection. In many pavements, structural distress can be assigned to one or more of at least four discrete categories: rutting, roughness, crack initiation and shear instability (shoving). In this study, data from all NZ’s Long Term Pavement Performance (LTPP) sites have been analysed to explore the benefits of replacing SNP with four separate “Structural Indices” - each being determined mechanistically (considering stresses and strains induced by an Equivalent Standard Axle) from data commonly available in RAMM.

Each Structural Index has been developed to fall within the same range as the traditional SNP, allowing straightforward implementation with minimal additional calibration needed to implement these in existing NZ Pavement Deterioration models and asset management software such as dTIMS. This paper presents the new developments resulting from the ongoing study, and shows the basis for the 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 enable: (i) effective use of all the data contained in RAMM, (ii) more reliable assignment of network Forward Work Programmes, (iii) reduced cost by 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

The Adjusted Structural Number, SNP (previously utilised as modified structural number - SNC), is an important measure for road pavements because it is the basis of most network prediction models (such as the World Bank (HDM) models and the dTIMS maintenance planning system). The reality is that SNP is, to date, the only measure that gives asset managers a simple overall indication of how much capacity / life can be expected from their networks.

However, the SNP principle has its limitations. In particular, it is based on the AASHTO design philosophy that aims to protect the subgrade – although, in many cases, NZ roads fail due to failure 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 based on test pit layer information and/or Falling Weight Deflectometer (FWD) tests, but current research is showing that much greater predictive reliability can be achieved by deriving structural parameters based on mechanistic concepts (using calculated stresses and strains induced throughout the pavement by an equivalent standard axle).

SNP is a fundamental parameter for network analysis, and while it currently has deficiencies, it is important to retain this concept in view of its well established role describing pavement performance simplistically. This report documents a process to rationalise the derivation of network structural parameters, rather than seek an entirely new prediction procedure.

The adjusted structural number can be used as an approximate indicator for structural life of pavements - provided that:

(i) rutting is the governing distress mechanism (i.e. no other trigger for rehabilitation applies);
(ii) the majority of the rutting occurs in the subgrade rather than the overlying layers;
(iii) the treatment length is correctly defined and relates to a uniform sub-section; and,
(iv) the appropriate percentile (rather than average) adjusted structural number is determined that corresponds to the percentage of road in a terminal condition which would trigger rehabilitation.

All four of these conditions must be satisfied before the adjusted structural number can be used meaningfully. However, the first condition is questionable for many roading networks (Henning et al, 2006) which indicates a substantial limitation to the structural number concept that needs to be addressed. In particular, the governing distress mode (i.e. 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 individual distress mode (for example, when predicting cracking, one needs to use an index that reflects the pavement's stiffness and fundamental strain conditions that will lead to cracking).

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 (i.e. information currently stored in RAMM).

When non-structural distress modes are put aside, as well as those that are rarely encountered, and also 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 be increasing rather than stabilising with ongoing load repetitions. Shear instability will commonly lead quickly to subsequent defects such as cracking of the surfacing, pumping and potholing.

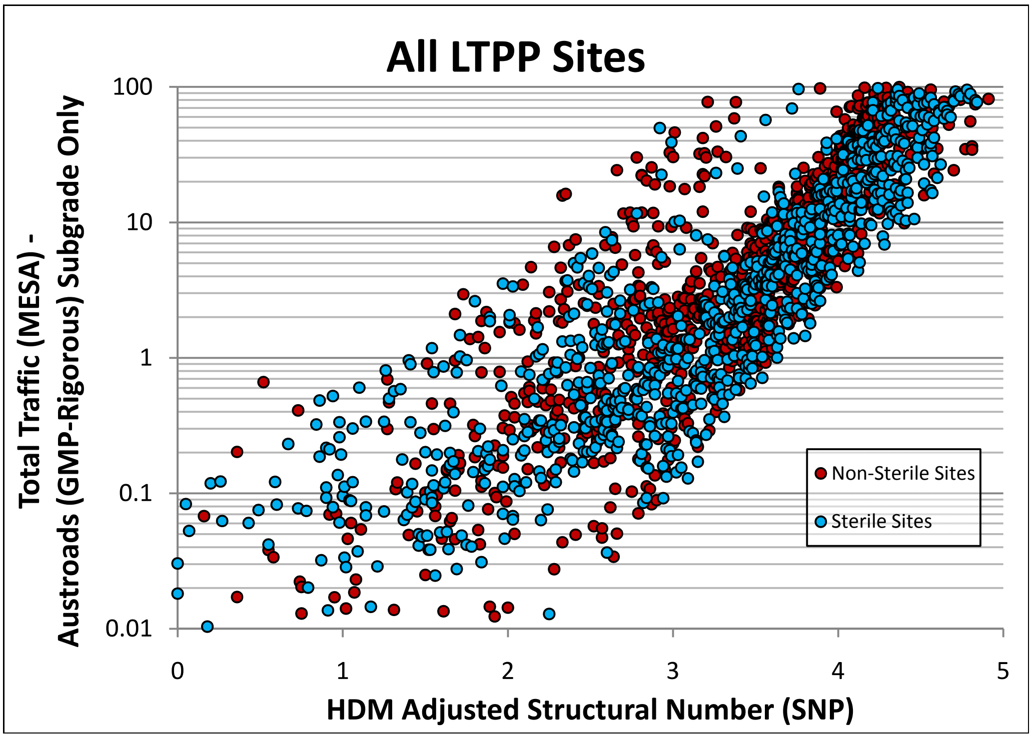
Roughness – loss of shape longitudinally along each wheel path. There are two prominent causes of roughness progression which include environmental effects and traffic load (Watanatada, et al. 1987). The load associated progression is primarily governed by structural non-uniformity (longitudinally) leading 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 of the surfacing. 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 is commonly reflected by 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 AASHO Road Test in the late 1950's, before mechanistic design methods were in general use (AASTHO, 1986). In the 1980s and 90s, structural number became the 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 planned 2002 release of its Mechanistic Pavement Design Guide, now under continuing development as the NCHRP (Ullditz 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 adjusted structural number (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's as determined by the Austroads subgrade strain criterion). The correlation for all national LTPP sites is shown in the following *Figure 1*. Predicted traffic in excess of 100 million ESA has been ignored as not practically credible.



Note: Sterilised Sites are sections which exclude any routine maintenance
Non-Sterilised sites receive maintenance as normal

Figure 1 - Traditional Adjusted Structural Number vs Predicted Subgrade Life (Total ESA) using the Austroads Subgrade Strain Criterion

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 numbers of Equivalent Single Axle-Loads (ESAL's) to a given terminal rut depth. This has been demonstrated at CAPTIF, (Stevens, 2006), while similar findings have resulted from ALF (reported by Austroads, 2006). 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 basically is 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-E PDG) is given in the following section.

2.3 REQUIREMENTS FOR AN IMPROVED PAVEMENT STRUCTURAL CAPACITY MODEL

The focus of this study is 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 (i.e. limiting to collected data or data that can readily be collected);
3. Straightforward incorporation into existing asset management software (e.g. dTIMS) and Pavement Deterioration Models contained in the software;
4. Ease of incorporating improvements to pavement technology or data collection methods; and,
5. Ease of calibration to different networks or sub-networks where different materials or construction practices apply.

Austroads principles apply to most of the five criteria. A key exception is that the Austroads principles do not rationally acknowledge non-linear (stress dependent) moduli. Many parts of Australia have materials that are essentially linear elastic, but those in New Zealand are predominantly non-linear. A study of NZ LTPP site characteristics addressing this issue is contained in Tonkin & Taylor (2006).

An interim measure for improved pavement modelling proposed in the current study, 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. There are many pavement performance models that incorporate SNP in pavement deterioration models used in Pavement Management Systems such as dTIMS. This report presents the Rutting and Roughness parameters which have been developed based on observed performance from both accelerated pavement testing (APT) and national long term pavement performance (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.

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.

3. DEVELOPMENT OF IMPROVED PARAMETERS

3.1 CONCEPTUAL BASIS

Currently, adjusted structural number (SNP) is 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 adjusted structural number, 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 below in *Figure 2*.

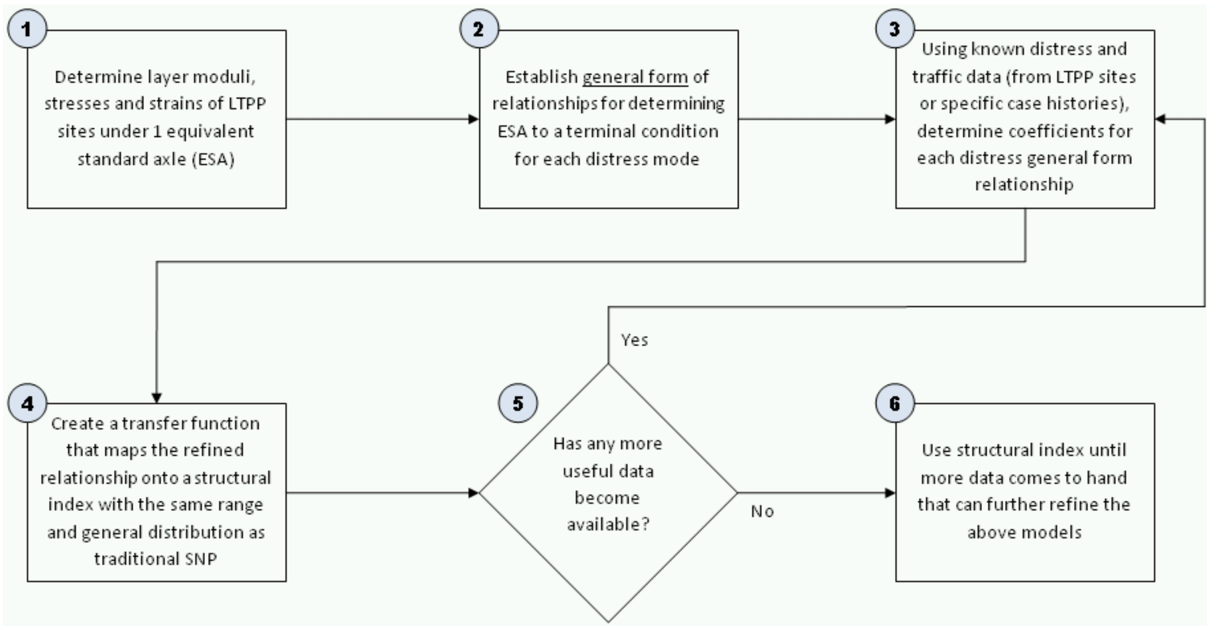


Figure 2 - Process of Determining Structural Indices

A starting point for all models was to explore existing fatigue relationships that have been widely used for many decades (e.g. for rutting the allowable subgrade strain for a given traffic loading (ESA) as shown below in *Figure 3*).

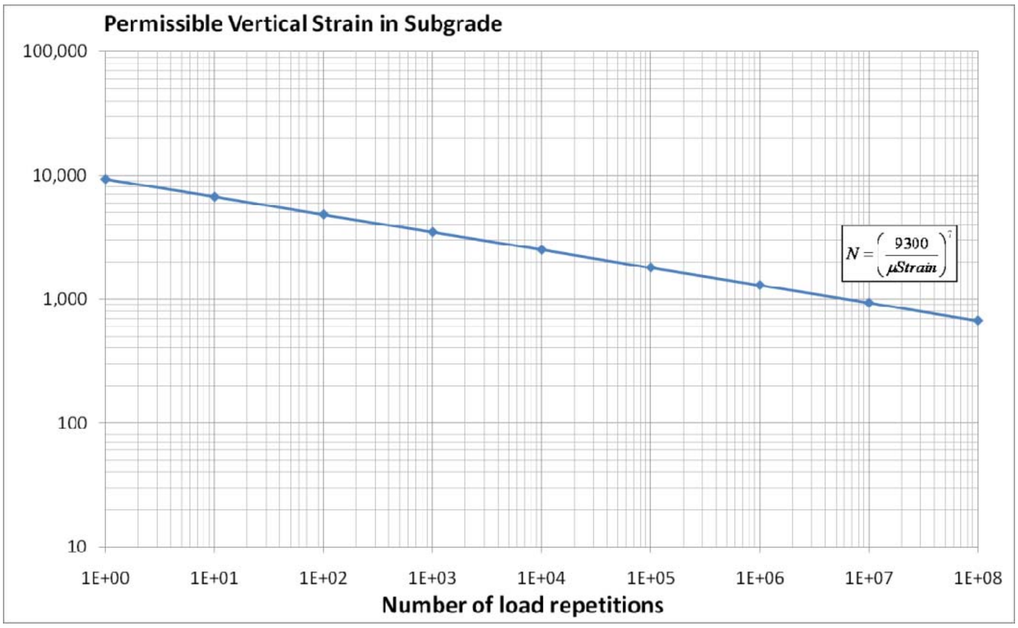


Figure 3 - Austroads fatigue relationship for anisotropic subgrades

Using the deflection bowl from a Falling Weight Deflectometer test, the strain at the top of the subgrade is readily calculated (Austroads, 2008) and the number of ESA to a terminal rutting condition is calculated from *Figure 3*. This would provide the simplest method of estimating rutting life using Austroads principles. The creation of an appropriate transfer function (see step 4 in *Figure 2*) is described in the following section.

The structural indices generated are calibrated to the range of SNP to minimise the effect on existing regression relationships already obtained with the NZ LTPP Programme (Henning, 2008) or HDM-4 (Hoque et al, 2008) so that SNP is simply substituted with the relevant SI for the distress mode under consideration. Because network management systems are focused on maintaining and updating deterioration models based on regression analyses, the new structural indices are readily assimilated to whatever extent is found to be significant when reviewing the independent variables, functions and calibrations used for any specific network.

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 PPM needs closer examination.

3.2 STRUCTURAL INDICES FOR SPECIFIC DISTRESS MODES

3.2.1 RUTTING

The above method of generating a structural index for rutting, 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 (2006). 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.

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 below in *Figure 4*

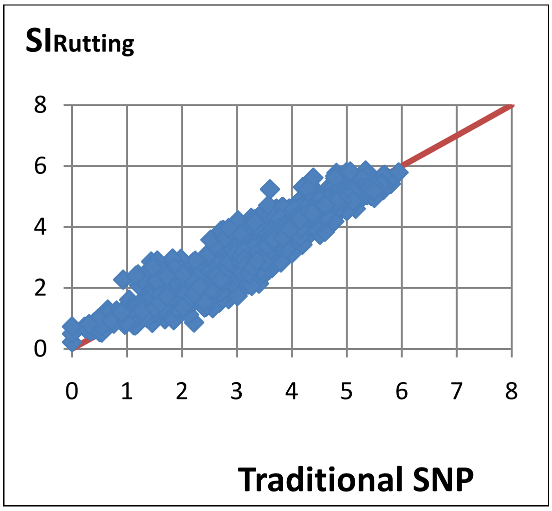


Figure 4 - $SI_{Rutting}$ vs SNP for LTTP site data

To illustrate the difference between the old and new parameters, it may be noted from *Figure 4* that, for example, 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. However, the network average pavement life could be very similar for the two cases.

The current form of the transfer functions are given in the main research report in preparation, along with the inverse functions that could also be used as a basis when a new regression is being explored.

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 is on the basis of the current roughness of the LTPP sites with necessarily approximate assumptions on the original conditions (immediately after construction). It is likely to take several more years of observation of the LTPP sites before the roughness progression model will have good reliability (Henning, et al, 2007). The interim model is 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 is an annual roughness progression based on environmental impacts.

Roughness progression at the LTPP sites shows, 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 LTPP sites least susceptible to roughness progression appear to have progressed at a rate of about one NAASRA count per one mm of rut depth, while those most susceptible to roughness have progressed at about five NAASRA counts per one mm 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 have been 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 (i.e. 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 0 mm at the start of the treatment length to 20 mm at the end). The standard deviation of rutting would be substantial over that treatment length, but because the rut depth decreases so steadily, roughness would be expected to be relatively low, compared to a treatment length where rut depth fluctuated repeatedly up and down by 10 mm over the full treatment length. The concept of the proposed measure is illustrated below in *Figure 5*.

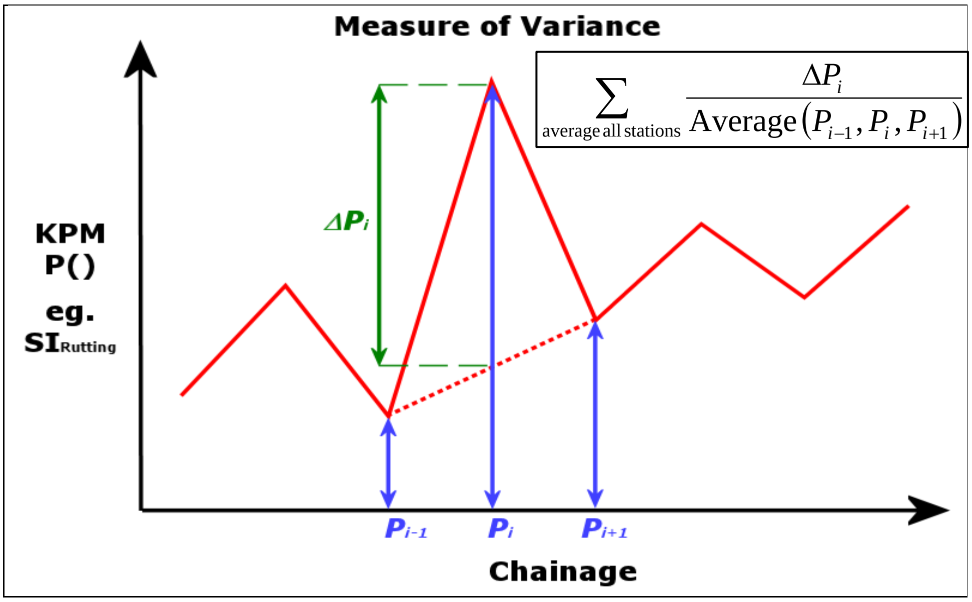


Figure 5 **- Measure of Local Variance for any Given Parameter P**

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 have been 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 can be made of where in the scale of roughness susceptibility (above) the performance of given treatment length can 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.

As an interim measure, a combined local variance (CLV) has been determined from trial weightings of 3 structural parameters normally evaluated for all FWD test points.

$$\textbf{CLV} = \textbf{LV}(\textbf{SI}_{\textbf{Rutting}}) + \textbf{0.8}^*\, \textbf{LV}(\textbf{1-2n}) + \textbf{0.9}^*\textbf{LV}(\textbf{NMR}) \qquad \textbf{Equation 1}$$

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 - the ratio of moduli between successive granular layers (Salt & Stevens, 2007) compared with that expected by the Austroads Guide (Austroads, 2008)

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 sites show more marked changes than have been exhibited so far. Of course, another overriding consideration is that roughness prediction is inevitably thwarted by unrecorded maintenance or disturbance (e.g. 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 will still require ongoing calibration and adjustment as more data become available. In addition, indices for cracking (flexure) and shoving (shear) have been developed in concept only. A brief description of the potential make-up of these indices is provided in the subsequent sections. Significant refinement of these indices will be undertaken as part of the ongoing research.

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 (2008) 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 the 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 at a later 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:

- (i) Vertical compressive strain in the centre of the uppermost layer (from back calculated modulus);
- (ii) Dissipated energy in the layer (using energy lost during the FWD test); and,
- (iii) Residual deflection (permanent deformation) after the FWD impact

A testing programme is underway to investigate occurrences of shear in order 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 chip-seal 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.

Details of the structural indices are provided in the main research report in preparation.

4. TESTING THE INDICES BASED ON NETWORK AND LTPP DATA

4.1 REPLACING SNP IN PAVEMENT DETERIORATION MODELS

Originally it was anticipated that the SNP could be simply replaced by the structural indices and give better prediction of pavement performance. This did not yield satisfactory results as the NZ pavement performance models are strongly empirical based models. Therefore, all independent variables are included to the prediction models in order to provide the best correlation with the actual data. It has been demonstrated that the structural indices are significantly different from the SNP.

The best way of introducing the structural indices in the pavement models is actually to include the indices in a full regression process that will deliver a new model for each defect. This process was tested on preliminary structural index values and promising results were obtained. Not only were the structural indices more significant predictors than the SNP, but models also resulted in an overall better correlation with the actual data.

It is therefore recommended to re-analyse the prediction models on the finalisation of the structural indices.

4.2 DIRECT USE OF THE STRUCTURAL INDICES AS MAINTENANCE DECISION TOOL

Far better maintenance options could be developed if the field investigator has an advanced understanding of the failure mechanism of the road. For example, if a pavement is displaying rutting, it could be as a result of unstable surface layers, or it could be that the pavement has carried its design life and the subgrade is getting over-stressed. Obviously the rehabilitation solutions for the two activities could be vastly different. Because the structural indices are developed based on the individual failing criteria, it would naturally be a strong indicator of rehabilitation needs.

Schlotjes (2009) tested this theory at network level during an analysis of data from Southland District Council. One of the interesting findings was that the indices will also be useful network level reporting measures. For example, Figure 6 illustrates the distribution of the respective indices on the Southland District Council network. It shows that most pavement failures would be due to cracking, rutting and roughness rather than shear instability. It would therefore be possible to monitor these outputs over time in order to establish the effectiveness of certain maintenance regimes on a particular network.

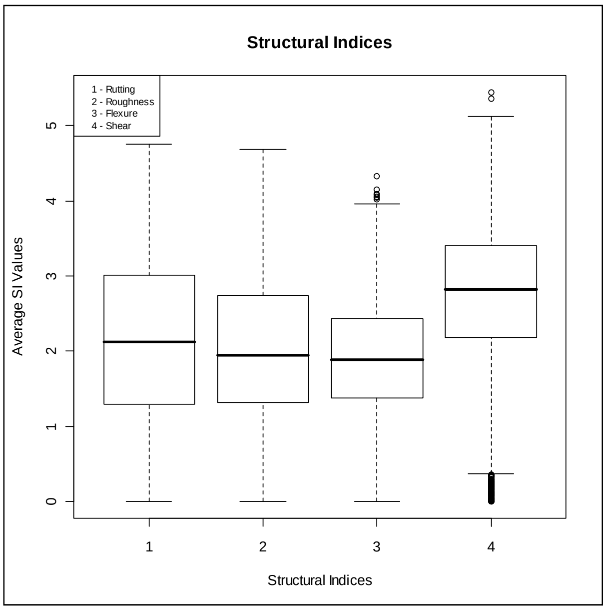
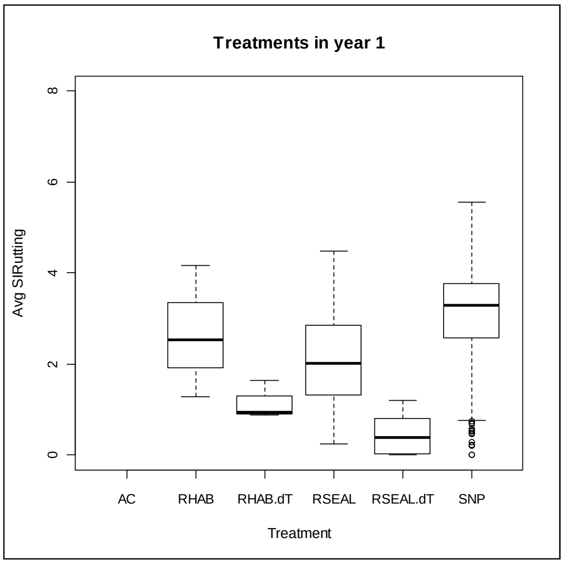
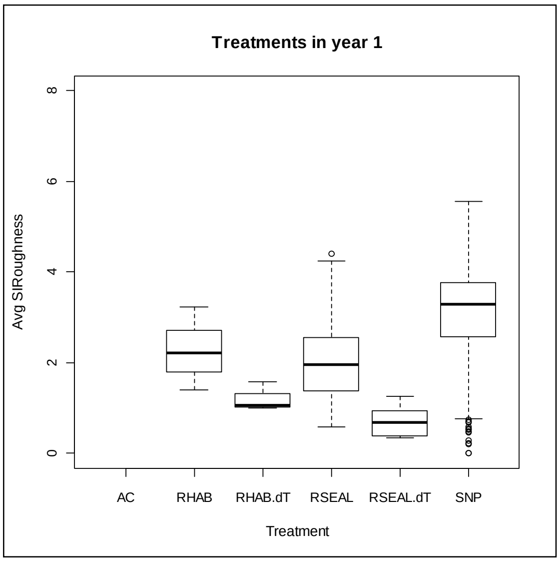


Figure 6 - Distribution of Indices for the SDC Network (Schlotjes, 2009)

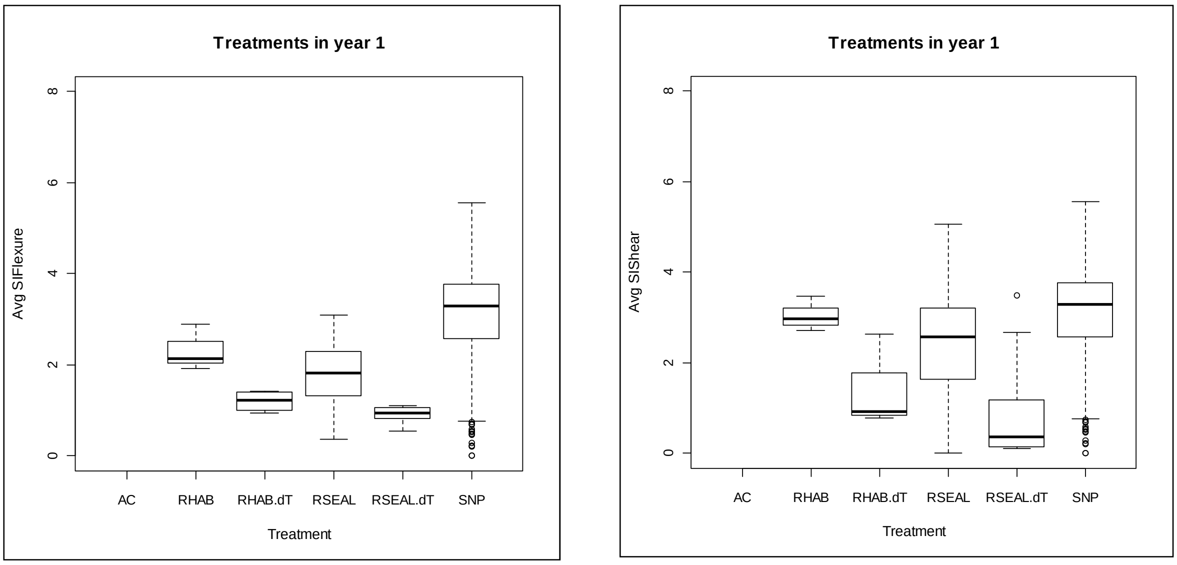
The structural index values were compared with the actual maintenance decisions for the network. Note that the network maintenance decisions were solely based on the existing condition data, and no input from either dTIMS or the structural indices were utilised. A summary of these comparisons is presented in Figure 7. For these comparisons, reseals and rehabilitation identified during the first year were compared with the corresponding four structural indices. 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.



Maintenance Compared with Rutting Index



Maintenance Compared with Roughness Index



Maintenance Compared with Flexure Index **Maintenance Compared with Shear Index**

Figure 7 - Comparing Maintenance Decisions and Structural Indices

The following observations are made from the above comparisons:

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

These results were encouraging since it confirmed that the field decision process can 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.

Lastly, as indicated by Schlotjes (2009), the structural indices also show real potential in the definition of over-all failure risk of pavements. In order to develop such a risk index, it is necessary to first establish the most likely failure mechanism prior to calculating the probability to failure. In her research, she has demonstrated that there is a strong relationship between the indices and actual defect development.

5. CONCLUSIONS

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 it 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 layer 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 (SI), has been proposed. For each of the currently recognised structural distress modes (i.e. 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 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 SI 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 remain stored in RAMM, updated structural indices may be readily generated at any future time for any network.

6. ONGOING DEVELOPMENT: REFINING THE STRUCTURAL INDICES

The key to refinement of the strength indices is to first test the concept at project level rather than at network level; 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 reason for focusing on individual treatment lengths is that networks are greatly complicated by unknown as-builts, uncertain recording of maintenance, and uncertain history of performance for many of their component roads.

The recommended process for project level calibration 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 (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 the predicted life for each distress mode.
4. Review the predicted condition with the observed condition in conjunction with the pavement engineer familiar with its historic performance.

After collating the data from a series of such project level sites, validate or refine the indices to ensure consistency of observed performance with each structural index.

As an example, see Figure 8 for 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 state highway. 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 (i.e. do the chainages where greater severity of distress is expected coincide with actual observed performance), and also would allow consideration of historic knowledge that may not be included in specific data collected.

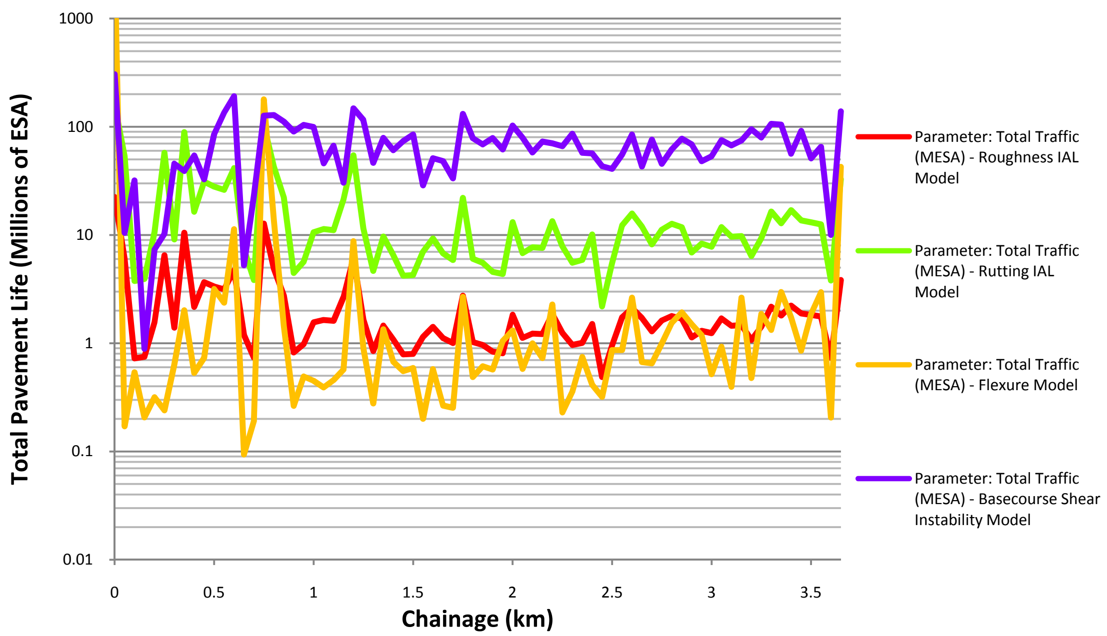


Figure 8 - Example distress mode analysis for validation of the PPM

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

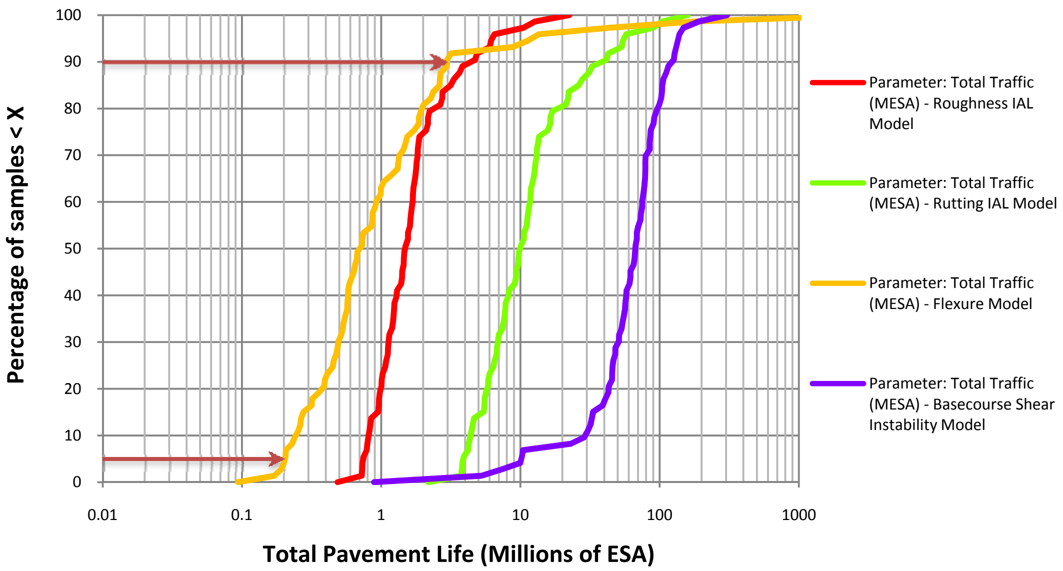


Figure 9 - Example of life predictions (the leftmost graph is most critical)

In this case it has been reported that at least 90 percent 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.

7. CLOSURE

The focus now is to expand the number of specific treatment lengths for calibration of the indices at project level. Each needs to be in a terminal condition with known as-builts, known trafficking and no disturbance (minimal maintenance). Suitability is greatly enhanced where there is a pavements engineer who has been closely involved with its historic performance. Ideal cases are premature failures of new or fully reconstructed pavements. Information on any such candidates for calibration would be welcomed by the writers.

Further development and refinement work required are summarised in Table 1.

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 ESA).
$SI_{cracking}$	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: (i) beneath AC surfacings (ii) instabilility beneath thin seals, and (iii) 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.

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