REPORT

RIMS Group
Good Practice Guide

Collection and Interpretation of Pavement Structural Parameters using Deflection Testing

Part I: Network Asset Management

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Abbreviations

AC	Asphaltic Concrete
AASHTO	American Association of State Highway and Transportation Officials
ARRB	Australian Road Research Board
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
DRP	Decreasing Route Position
dTIMS	Deighton Total Infrastructure Management System
ESA	Equivalent Standard Axles
FBS	Foamed Bitumen Stabilisation
FWD	Falling Weight Deflectometer
FWP	Forward Work Programme
HDM	The Highway Design and Maintenance Standards Model (for roading investment developed by the World Bank)
HDM4	Highway Development and Management (an extension of HDM)
IRP	Increasing Route Position
KML	Keyhole Markup Language
LTTP	Long Term Pavement Performance
NCHRP	National Cooperative Highway Research Program
NZTA	NZ Transport Agency
OGPA	Open Graded Porous Asphalt
QA	Quality Assurance
RAMM	Road Assessment and Maintenance Management
RWD	Rolling Wheel Deflectometer (RWD)
SI	Structural Index
SNP	Adjusted Structural Number (or variations such as the modified structural number SNC or original AASHTO adjusted structural number, SN)
TSD	Traffic Speed Deflectometer

Table of contents

1 Introduction

1.1 General

Maintaining accurate and current information about the condition and remaining service life of pavements is fundamental for its efficient maintenance. It also provides the information needed for planning network forward work programmes for pavement rehabilitation or reconstruction. From a management perspective, it is desirable to obtain pavement structural data to give a basis for informed decisions. The data collection is typically distinguished as being at either Project or Network Level.

This document (Part I of a set of two reports) provides a good practice guide for road controlling authorities about "Network Level" pavement deflection testing for Asset Management. Part II of this report series addresses "Project Level" testing and interpretation for specific road lengths' rehabilitation treatment, or their quality control during construction. Typical structural parameters from past and ongoing studies on New Zealand roads, including the NZ Transport Agency's (NZTA) Long Term Pavement Performance (LTPP) benchmark sites, are presented.

Pavement structural performance can be determined through it surface condition, behaviour under load, and material properties. Some aspects are readily observed (such as surface condition), whereas subsurface information concerning the basecourse, subbase and subgrade is costly to gather and interpret with destructive testing; this is why non-destructive methods, particularly deflection testing with the Falling Weight Deflectometer (FWD), are commonly used. Results are generally accessible in the Road Assessment and Maintenance Management (RAMM) database.

Obtaining deflection data at highway speed has been attempted seriously since about 2000 - with the Rolling Wheel Deflectometer (RWD) in the United States, and more recently the Danish Traffic Speed Deflectometer (TSD). As the FWD is regarded as the benchmark for structural testing in view of its inherently greater accuracy, it is the focus of this guide.

Predicting a roading network's future performance (in programs such as dTIMS or HDM) has traditionally used the empirical structural number concept (or variations such as the modified structural number or adjusted structural number, SNP). This concept proved a good starting point when it was introduced in the late 1950's as the only structural information available was the pavement layering and subgrade strength or, alternatively, just the central deflection from the Benkelman Beam. Structural number provides a "one size fits all" empirical parameter which results in values close to zero for weak pavements up to about 8 for strong structural capacity. Details are given in Appendix C (equations 1 and 2). SNP includes an additional term for structural capacity of the subgrade and this term can be as low as -1, hence the SNP range can be from -1 to +8, with median often at about 4.

While SNP is straightforward to calculate, it is of limited reliability in associated predictions (such as Forward Work Programmes) because it does not account for the massive developments in mechanistic analysis since its introduction. In particular, it does not distinguish, as newer models do, between the various modes of distress that can bring a pavement to a terminal condition.

For this reason, this report focuses on a set of "structural indices". Each of these indices is similar to the structural number, but for a specific distress mode: rutting, roughness, flexure, cracking and shear. Explanations about the structural indices concept and its application to road network asset management is also included in this report.

1.2 Network versus Project Level Evaluation

Network level management focuses on of the road asset's present and future condition and establishing performance models for pavement life expectancy and associated maintenance, or FWPs for rehabilitation. The network level tests are set further apart than the ones used at the project level. They are also spread across the entire network, which results in their having a lower level of scrutiny than the project level tests. Reporting for network level surveys normally includes the basic data format given in Appendix A, structural indices, and a preliminary mechanistic analysis with layer moduli.

Part II of this report series is focused on project level assessment of individual lengths of pavement that have reached a terminal condition and require rehabilitation. Treated pavements are often tested at project level too, because structural evaluation is increasingly being specified as a quantitative QA tool, both during and immediately after construction. This is because numerous case histories have provided the data needed to predict the stresses and strains after construction that result from testing during construction at either subgrade or subbase level. This is why a greater depth of analysis and more closely spaced test points are required for project level evaluation. Reporting typically includes the basic data format given in Appendix A, a preliminary mechanistic analysis with layer moduli, and nominal rehabilitation options.

The distinction between Network and Project Level testing is discussed further by Austroads[1] in Section 2, Part 5 of the Austroads Guide to Pavement Technology.

2 Basic Principles and Data Collection

2.1 Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD) is currently the most practical system for accurately measuring the deflection response of a pavement subject to a dynamic load. It uses a set of weights, which may be dropped from various heights onto a circular loading plate with load cell incorporated, and a number of geophones (deflection sensors) that are incrementally spaced in a line radiating out from the point of impact. Test results are recorded electronically and the whole setup is usually mounted on a trailer towed behind a specially equipped vehicle.

[1] Austroads is the association of Australian and New Zealand road transport and traffic authorities.

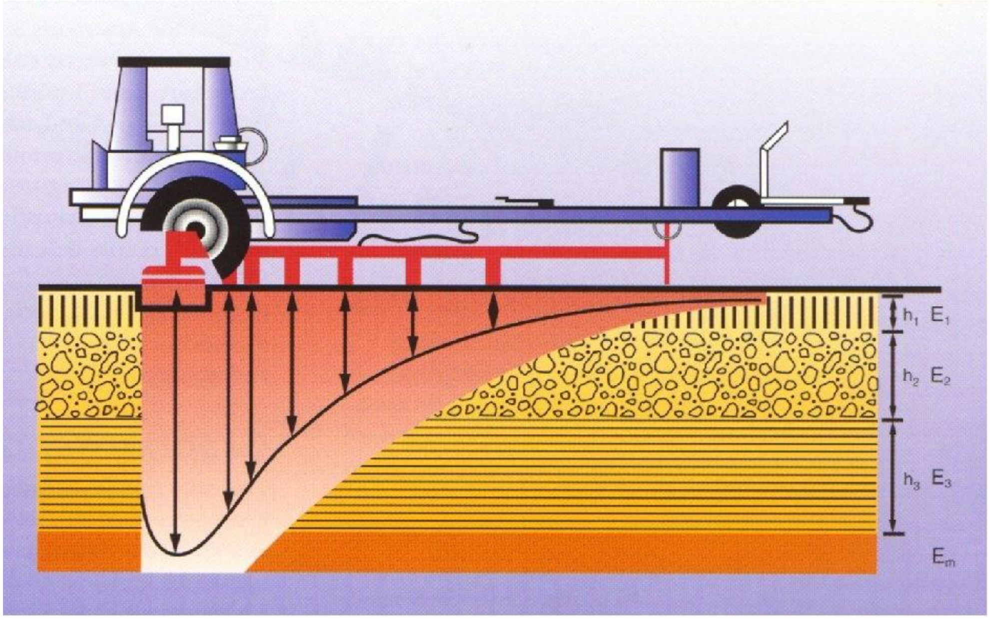


Figure 2.1 - The FWD trailer showing deflection bowl recorded at geophone offsets

The geophones are used to measure the deflection bowl produced by the impulse of the falling weights on the pavement surface. Having an array of geophones in a radial line away from the loading plate allows the maximum deflection (vertical displacement of the pavement surface, usually in the range of 0.2 to 2 millimetres) to be measured as a function of time during the load impulse. A typical full-time history recording is shown in Figure 2.2.

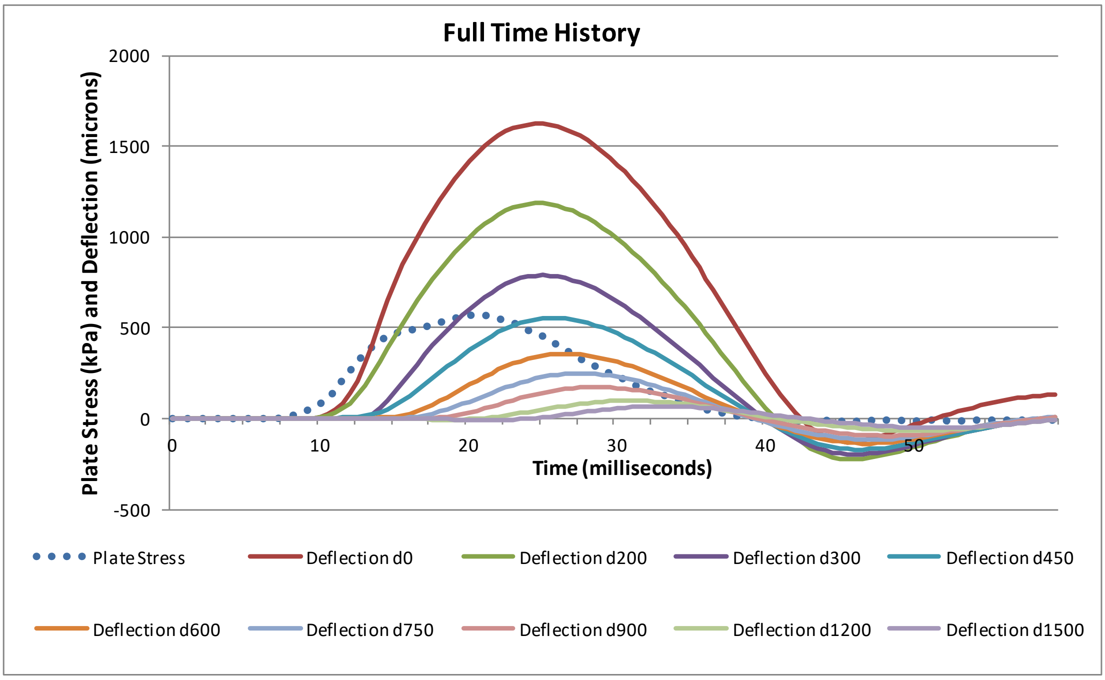


Figure 2.2 Typical FWD Full Time History (FTH) showing displacements over time at each geophone

The peak deflections, combined with the measured impact load, may be back-analysed (using layered elastic theory) to determine the stiffnesses (dynamic moduli – E_1, E_2 etc.) of the various layers and the subgrade (E_{SG}).

The moduli provide a model for the type of pavement, distinguishing between unbound granular and bound layers, as each layer type has characteristic properties. Further explanation on analysis and alternative software packages is given in Part II of this guide.

Applying the surface stresses from a standard wheel load (an Equivalent Standard Axle or ESA) allows a subsequent forward-calculation of the in-service stresses and strains throughout the pavement. It may be used in conjunction with empirical strain criteria to predict pavement life.

More recently, the stresses and strains in each layer have been used to determine all potential distress mechanisms and the pavement's remaining life for each potential distress mode. This mechanistic-empirical approach (mathematically calculating moduli, stresses and strains and relating them to past experience of pavement performance) is progressively replacing former empirical methods based on bowl parameters. Examples include central deflection (standardised to a 40 kN load) and curvature (the difference in deflection between the central geophone and that of the geophone at the 200 mm offset).

A major advantage of analytical or mechanistic structural design methods over more empirical methods is that the former may be used with any type of material and structure, and under all climatic conditions (provided that fatigue criteria are established for each material type). The latter, on the other hand, may be applicable only under the conditions for which the empirical relationships were developed. The mechanistic model then provides for continuous improvement as more case histories or other information for calibration becomes available; these are providing increasingly reliable predictions about the pavement's remaining service life. When results for a network are compiled, a picture can be built up of which sections need attention, which distress mechanisms apply (and hence which solutions are most appropriate) and how soon remedial work will be required. The result offers asset managers a greater understanding of future performance and where to most effectively direct the increasingly limited budgets and resources available for remedial treatments. Further discussion is given in Appendix B.

2.2 High Speed Structural Data Collection

The RWD and TSD have two advantages; speed and continuous readings. Initial models could not, however, accurately define the full deflection bowl; furthermore, they could only produce useable accuracy for peak deflection by averaging a large number of readings along the wheeltrack.

Recent TSD's have been built with up to 10 Doppler lasers. These measure pavement movement velocities at various offsets from the centre of the dual wheel print; displacement is then integrated from the result.

2.3 Planning data collection surveys and sampling

2.3.1 Network Level Survey Planning

Network testing is primarily for asset management. It may be used for determining whether current expenditure will at least maintain the standard of the network, or for determining the future budget and most effective Forward Work Programme (FWP) to meet and maintain a given level of service.

Where budget permits, managers of large networks may allocate funding (often over about five years) to progressively collect structural data across the full network. In the initial years, for cost-effective testing and modelling with dTIMS, the network may be subdivided into various categories on the basis of terrain, geology and road type. This enables representative data to be collected from each of these categories (see Section 3.5 for further discussion). Once patterns of pavement wear are known and understood across the network (such as in areas with heavier traffic), they can be prioritised for testing and remedial works.

2.3.2 Project Level Survey Planning

Project level testing (discussed in Part II of this Guide) is carried out for either pavement structural rehabilitation or for quality assurance during construction and/or during the maintenance period (after bedding in from trafficking). Planning for rehabilitation is centred on feedback from field inspections and records of roads known to be approaching the later phases of their service lives.

In many cases, high speed condition data may indicate trigger levels for intervention, or maintenance costs may be excessive. More detailed information is required for project level surveys (such as traffic loading, layer thicknesses and material types) compared to network level surveys as more reliable quantification is necessary.

2.3.3 Network Level Testing

Network level testing is carried out in the left wheeltrack at usually 100 or 200 metre intervals in each lane, depending on the extent to which treatment lengths need to be defined.

Where the budget for data collection is constrained, savings can be made by having the FWD operator undertake progressive appraisal of the condition of each road during testing; the increasing lane (IRP) is tested initially and the residual life is calculated in the field to determine if the road has a substantial remaining service life. If the initial results indicate little remaining life, the decreasing lane (DRP) is also tested to provide greater reliability for the Forward Work Program; otherwise the budget is allocated to roads elsewhere in the network. A suitable trigger level needs to be adopted (usually 10 or 15 years remaining life).

Table 2.1 Network level FWD testing regime

Centreline Length	FWD Test Spacing based on Field Calculation of Residual Life	
	Life > 15 Years	Life < 15 Years
0 m - 200 m	5 Tests (3 in IRP lane, 2 in DRP lane)	
200 m - 500 m	100 m intervals in each lane	
500 m - 2 km	10 tests in IRP lane only	10 tests in each lane
2 km - 5 km	200 m intervals in IRP lane only	200 m intervals in each lane
> 5 km	200 m intervals in each lane, or 400 m intervals if geologically uniform terrain	

The systematic approach above will usually allow a greater portion of the network to be tested for a given budget while not impacting on the predicted 10 year FWP, as less testing is done on roads which are unlikely to require structural improvement in the short or medium term. Variations of Table 2.1 can readily be tailored to address specific information required on any given network. Alternatively, the roads can be further selected based on high speed condition data and historic traffic. If the budget permits, closer and more regular spacing can be adopted for initial coverage. This will enable improved demarcation of transition points when establishing structurally homogeneous sections for treatment lengths.

Costs for either project or network level testing generally range from $10 per test point for large surveys where minimal traffic control or mobilisation is required, to over $15 per point for a single remote treatment length where only a few test points are to be collected. Using the spectrum of test spacings in table 2.1 gives costs ranging from $25 to over $100 per lane-km for FWD. A similar order of cost can apply for traffic control. Costs are inclusive of standard processing with all data files ready to be input into RAMM in their specified format.

Standard outputs can also be generated as KML files for viewing in Google Earth. An added advantage is these can be readily inspected on a GPS-enabled tablet or smartphone. These facilitate site inspections network inspections are being undertaken, as they show the road, terrain, current location, FWD test location, remaining life and predicted terminal distress mode).

2.3.4 Project Level Testing

Standard project level testing typically involves tests at 50 m centres in each lane (staggered 25 m between left and right lanes), with additional tests recommended on any unusually distressed locations - this enables distress mechanisms to be identified from the test results by the pavement analyst. Standard testing is in the left wheeltrack, with a minimum of 30 tests (down to a minimum of 10 m spacings for short roads) which provides good reliability for assessment of the 5 or 10 percentile parameters usually adopted for design or acceptance testing.

2.3.5 Procurement of Data Collection Services

NZTA carries out regular testing of its LTPP sites with an annual contract that also provides for network deflection data collection and for project level sites where rehabilitation is planned or has been completed. Testing is procured by an up to five-year performance-based tender. Other road controlling authorities tend to procure services on an annual basis; a similar approach is taken by network contractors. Quality assurance during construction often necessitates short notice for mobilisation of testing equipment and execution, but because the time taken to test typical lengths of rehabilitation is short (usually less than an hour), testing can often be accommodated for sites close to the main centres. Costing is traditionally determined on a per point basis rather than an hourly rate, so budgets are usually simple to establish. Austroads standard specification for FWD testing is located on their internet site[2].

2.4 FWD data storage in RAMM

NZTA data collection contracts prescribe a specific data format for RAMM storage (Appendix A). This is compiled and provided in the form of a Microsoft Excel spreadsheet. The data stored for each test point are primarily: the location, peak reading from each deflector, the peak pressure from the load cell, temperatures, and SNP.

The initial full-time history for each sensor recorded in the field is normally available directly from FWD providers. This can be used to obtain additional information, particularly comparing the loading and unloading cycles to determine whether the deformation is purely elastic, or if some plastic deformation is evident. Detailed analysis of the full-time history allows more information to be obtained, particularly in relation to basecourse shear instability.

Data obtained directly from the FWD provider will normally be provided on a spreadsheet with the same fields as in RAMM, plus additional columns for the processed data (in particular, layer moduli and critical strains, and structural indices, if required).

2.5 Seasonal effects

Seasonal variation of pavement deflection generally has two major external controls (temperature and water content) as some layer moduli can be dependent on either. Temperature ranges over diurnal and seasonal timescales can have a minor effect on pavement condition, but they can also be substantial if moisture sensitive materials are subject to freeze/thaw conditions.

[2] AUSTROADS 2008. Specification AG:AM/S002 Pavement Deflection Measurement with a Falling Weight Deflectometer

The back analysis of a deflection bowl provides results for the specific water content at the time of testing. Seasonal variations in moduli must therefore be considered prior to calculating residual life and overlay requirements. Software packages vary in the way seasonal effects are incorporated. One option is to increase deflections by a multiplier in the range of 1.1 to 1.6 if measurements are not carried out during a wet period. Another approach is to assume an annual sinusoidal variation in moduli between a maximum and minimum value. (Usually, the subgrade modulus alone would be varied but the factor could be applied to all unbound layers, with a similar end result).

In a long-term study of deflection changes with seasons in Australia, Rallings & Chowdhury (1995) found a generally sinusoidal variation in peak deflection each year and concluded that a seasonal adjustment factor of 1.1 would be appropriate for deflection measurements made between mid-summer and the end of autumn. The data they obtained include both “wet” and “dry” rainfall areas and there is clearly more seasonal fluctuation of deflection in the case of the dry areas. If the design condition for the subgrade is taken towards the wetter state rather than at the median condition, then an adjustment factor of about 1.3 would be indicated by the data.

Another similar study undertaken at Delft University (Van de Pol et al, 1991) produced comparable sinusoidal seasonal fluctuations in subgrade moduli deduced from FWD measurements taken over a two-year period, but no specific guidelines for assessing seasonal effects generally were indicated.

A considerable degree of judgment will be required to assess seasonal adjustment factors for specific sites. Factors listed in Table 6.1 of Part II (Project Level) are suggested as provisional guides for temperate climates such as New Zealand. This table draws on the above references and is supported by studies in progress. The subgrade water content at the time of testing should be assessed relative to expected ranges in that locality. In practice, a site-specific evaluation of any seasonal effects that results in a relevant subgrade modulus correction, would entail more study but is likely to be more realistic than a nominal correction of deflections.

3 Structural Indices for Modelling of Pavement Performance

3.1 Data requirements

The various categories of data for evaluating network structural performance and their relative importance are given in the following table. Ideally, the network manager should provide the pavement structural analyst all the items in italics after validating the database, although the analyst can also obtain or deduce some as indicated below.

Table 3.1 - Categories of information for pavement structural analysis

1. Essential	• FWD peak deflection data, peak plate stress • Pavement temperature at time of test (only used for asphaltic layers)
2. Essential, but can be assessed by FWD structural analyst	• *Subgrade type (volcanic ash or otherwise)*
3. Important, but can be addressed by FWD operator	• *Surfacing type* • *Top structural layer type* • Full time history (dynamic record of all sensors while the FWD load is applied)
4. Preferable (but can often be deduced by FWD analyst)	• *Nature and thickness of any bound stabilised layers* • *Depth to subgrade* • Weighted mean annual pavement temperature (WMAPT °C)
5. Preferable (should be readily available in RAMM)	• *Pavement age and traffic carried (ESA) since constructed or last rehabilitated (whichever is the lesser)* • *Age of present surfacing* • *Traffic (ESA/lane/year) and intended design life*
6. Desirable to ensure model consistency	• *Rut depths after bedding in* • *Current rut depths* • *Roughness after bedding in* • *Current roughness* • *Recent visual survey(s) especially cracking*
7. Desirable for improved calibration	• *Historic rut depth progression* • *Historic roughness progression* • *Historic maintenance costs*

If the first four categories above are used in the modelling, structural indices can be determined and an adequate appreciation of pavement life should be obtained based on typical nationwide performance.

By including the 5th category as well, some account of local conditions will be acknowledged. By adding the 6th and 7th categories, maximum reliability will be obtained as it allows each specific road's past performance to be used for effective calibration. Consequently, the network's FWP can be much more realistic and focus on only those treatment lengths that are essential, simply through supplying the structural analyst with comprehensive road condition data.

A full as-built profile (or test pit log and penetrometer results) and/or particle size distributions are not essential for network analysis. It is helpful if as-built information is on hand, but any destructive testing is not usually warranted or cost effective. In any case, it is normally preferable to carry out the non-destructive testing (FWD) and structural analyses first, and then decide whether there is a case for some test pitting, and if so, which locations are critical so costs can be minimised. In many cases, for network testing, sufficient information can be deduced by the analyst with no destructive testing. However, for project level interpretation, test pit information is normally required – particularly if there is a shortage of information on historical performance.

3.2 Structural Evaluation

FWD data collection has now become standard practice, and the raw data files are stored in RAMM along with all the other pavement condition data. Processing the FWD data to establish a mechanistic structural model is normally carried out and allows the network manager to maximise the benefit obtained from the testing programme.

Structural number concepts originated well before mechanistic analysis procedures became readily available to practitioners. The reason the SNP or similar notional parameters can give an approximate indication of possible structural deterioration for a large network is that it is essentially a measure of the subgrade's ability to resist deformation. However, SNP is not able to give any indication of how a particular pavement structure would behave if any layer other than the subgrade exhibits distress. For example, a road consisting of a cement stabilised base overlying a thin weak layer on a hard subgrade will have a high SNP but might well crack after minimal trafficking.

SNP has been a fundamental parameter for network analysis, but its limitations are evident when monitoring predicted versus observed deterioration rates.

SNP can be used as an approximate indicator for structural life of pavements, provided:

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
iv. Rather than simply taking the average SNP, an appropriate percentile for SNP is determined corresponding to the percentage of road in a terminal condition (ie the relevant percentile should be nominated by the road controlling authority)

All four of these conditions must be satisfied before the adjusted structural number can be considered reliable. However, as the first condition may not apply to many roading networks (Henning et al, 2006), this substantial limitation to the structural number concept 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. For this reason, considerable emphasis is given in this Guide to the case for abandoning SNP for those networks where it is proving unreliable. Appendix C provides a comprehensive explanation on the basis, determination and use of SNP.

Slightly better modelling can be achieved by using empirical parameters such as standard deflection and curvature, or semi-empirical "Layer Index" or "Pavement Number" values as adopted in South Africa (Horak, 2008). High standard deflections do provide an approximate indication of the likelihood of rutting, and high curvature promotes cracking of any stiff layers near the surface. However, these approaches are still limiting in terms of future improvements and allow less definitive appreciation of distress modes. The use of mechanistic analysis is promoted instead, with alternative structural parameters (termed structural indices) provided as a rational substitute for SNP that will still enable traditional predictive relationships to be utilised with no major restructuring apart from minor recalibration. For each of the currently recognised structural distress modes (ie. rutting, roughness, cracking, flexure and shear) a corresponding structural index can be quantified. The structural indices are intended primarily for network studies rather than for use at rehabilitation sites.

To determine the governing distress mode, deterioration models need to examine all potential distress modes using relevant parameters for each individual mode (for example, when predicting cracking, one needs to use an index that reflects the pavement's stiffness and the fundamental strain conditions that will lead to cracking).

The additional work, beyond that required for assessing SNP, involves analysing the structural data to ensure a rational multi-layer elastic pavement model is obtained. This will enable layer moduli and the critical stresses or strains that would be induced by an equivalent standard axle (ESA) to be calculated. The same mechanistic approach is now the basis of the Austroads design guide for pavement rehabilitation, although an empirical component (establishment of fatigue criteria from observed performance of pavements) remains essential.

3.2.1 Structural Distress Modes

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 these 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 cracking of the surfacing, consequent water infiltration, pumping and potholing. On New Zealand highways, if shear instability develops, it is most likely to be within the unbound basecourse layer, but it may also occur within an unbound subbase or within multiple chip seal layers. It is important to identify the source, and this can sometimes be evident in the FWD data.

Roughness – loss of shape longitudinally along each wheel path. This is primarily governed by structural non-uniformity (longitudinally) leading to variations in rut depth. Roughness is also a secondary effect of shear instability.

Cracking – cracking of thick (structural) bound layers, traditionally characterised (eg. in the Austroads Guide) as being initiated by excessive horizontal tensile stresses at the base of the layer leading to bottom-up crack propagation – although cracking may also initiate elsewhere in the thick layer.

Flexure – this term is used here to denote only surficial top-down cracking caused by the imposition of both shear stresses from wheel traction and horizontal strain cycles within a thin asphaltic 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 thin surfacing and generally the reverse sequence at the top of the surfacing. The tractive stresses applied by the driving wheels of a truck travelling at 90 km/h on a flat road (where only wind resistance has to be overcome) is typically about 80 kPa, and at the front of the tyre print, this induces additional tensile strains. The combined strains eventually initiate top-down cracking within chip seal layers and cracking or ravelling of thin AC or OGPA. Additional surfacing may be sufficient for substantial life extension if the existing surfacing is thin (and cracking no more than incipient). However, aged surfacings suffering from flexure are likely to require replacement or other structural rehabilitation. Flexure is differentiated from bottom-up cracking as each has a different significance. This is why different treatment needs to be considered when mechanistic modelling is carried out to determine a FWP.

Some mechanisms for pavement structural deterioration are inter-related. Once flexure or cracking has initiated, and there is water ingress, a pavement can develop accelerated rutting, pumping, shallow shear and potholing, which in turn leads to rapid deterioration of a pavement's roughness. Correctly predicting the occurrence of this first significant failure mechanism can lead to timely and appropriate intervention resulting in significantly prolonged pavement life.

Adopting the above concept of determining pavement structural life for the five distress modes has very similar equivalents in other forms of structural engineering, e.g. the design of a structural column or beam for multiple distress modes (bending moment, buckling, deflection and shear capacity).

3.2.2 Pavement Types

The most common form of rural highway pavement structure is a chipseal surfacing on an unbound basecourse. As a result, for the flexure mode, seal cracking is the prime consideration. However, there are several cracking and flexural modes that are encountered in urban pavements, depending on the type of surfacing and near surface material type(s):

- Chipseal on unbound granular basecourse; cracking of the seal, top only
- Chipseal on modified or cement bound basecourse; cracking may be bottom up within the stabilised basecourse (depending on cement content) or confined to the seal surfacing
- Chipseal on foamed bitumen stabilised (FBS) basecourse; cracking similar to thick structural AC (Austroads)
- All combinations of thin AC, SMA or OGPA over the three categories above
- Thick structural AC

The above provides a total of at least nine permutations of distinct pavement types that may be present on urban networks. Considerable work has been carried out with the development of relevant fatigue criteria applicable to New Zealand conditions and construction practices; however, those with two stiff yet dissimilar layers (ie. thin AC/ OGPA over stabilised basecourses) have yet to be well documented.

The recent NZ research (Gray et all, 2011) into stabilised layers with chipseal is still very preliminary, but it is already clear that the observed performance for the types of stabilisation being carried out in New Zealand is not well characterised by the Austroads relationships (particularly for foamed bitumen stabilisation). So, although preliminary, the models from the New Zealand data are likely to give more reliable results locally because they relate to different mix types and construction practices.

3.3 Conceptual Determination of Structural Indices

The structural indices may be generated either simplistically (from algorithms currently under development, possibly for use within RAMM) or by using a rigorous approach (structural analysis using layered elastic theory). In many cases, the latter will be available directly from the FWD provider. However, any moderately experienced analyst may soon derive the indices for each structural distress mode from following the first principles approach described below:

1. Download FWD data and pavement condition data from RAMM
2. Obtain additional FWD full time history from FWD provider (optional)

3. Use recognised multi-layer elastic analysis software to back-analyse the deflection bowls and generate layer moduli. Because most New Zealand pavements have subgrade moduli that are non-linear (ie. stress dependent), software that accommodates this characteristic properly is preferred. (Further discussion is given in Part II of this guide.)
4. Use the forward calculation routine (in the same program used for back-analysis) to calculate at each FWD test location, the maximum stresses and strains under a 1 ESA load at critical points in each layer and at the top of the subgrade
5. Use performance monitoring of local pavements subject to known traffic (including findings from NZTA's LTPP sites and CAPTIF) to determine fatigue relationships. These need to relate the number of ESA to a terminal condition (lifetime traffic) for each of the five main structural distress modes to the layer moduli, stresses and strains under 1 ESA loading. Appropriate mechanistic criteria have historically been subject to considerable revision, and refinements are continuing; therefore there is no intent in this Guide to prescribe the specific transfer function that must be used for each distress mode, although functions which have been used and are likely to be appropriate for some parts of New Zealand are given in section 3.4. Elsewhere, relatively minor calibration may be required.
6. Determine the traditional SNP values for all available data.
7. Determine a transfer function to convert the lifetime traffic into a structural index (for that specific distress mode) so the structural index has the same range and general distribution as the traditional SNP. The purpose of adopting the same range is to simplify the changeover from SNP, as the dTIMS and HDM4 systems can readily accommodate the minimal changes. (However, the distress-specific lifetime traffic itself should, in time, become directly incorporated in the modelling as a more fundamental parameter.)
8. Repeat the above steps to further refine (or totally revise) each model as more data come to hand from the LTPP sites and other case history sources. However, it is important to ensure the calculations for all structural indices for any one network use the same set of transfer functions.

3.4 Mechanistic Criteria

A starting point for all models was to explore the existing fatigue relationships promoted by Austroads, or those widely used elsewhere. For example, rutting performance is often in terms of the allowable subgrade strain for a given traffic loading (ESA), as shown in the following figure.

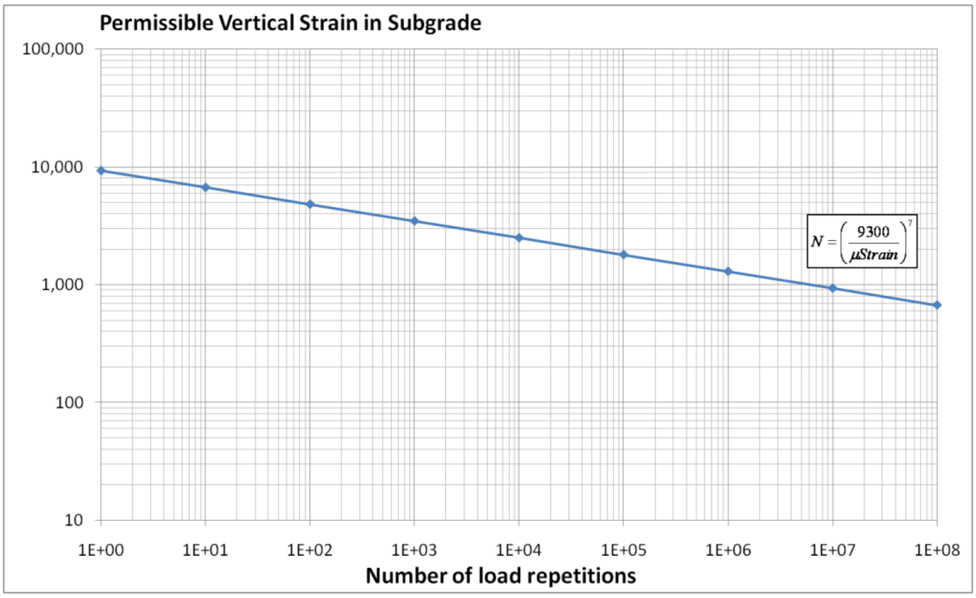


Figure 3.1 – Austroads fatigue relationship for anisotropic subgrades.

This provides the number of ESA to a terminal rutting condition ($N_{RUTTING}$) as a function of the microstrain at the top of the subgrade. Using the load and deflection bowl from the FWD test, the strain at the top of the subgrade is readily calculated from layered elastic theory (Austroads, 2008).

Austroads also prescribes a function for cracking life for thick structural asphaltic pavements –a fatigue relationship for $N_{CRACKING}$ in terms of the asphalt modulus and horizontal tensile strain at the base of the bound layer. The application of the Austroads transfer functions to New Zealand conditions, particularly those for cracking, has frequently been debated. Accordingly, locally derived criteria based on observed performance from well documented case histories are preferable. Provided the following steps are followed, and the final model calibration checked anyway, a reliable model should result so long as a soundly based fatigue criterion is adopted throughout the network.

The next step is to determine the distribution of SNP for the network under consideration. Ideally, this should use the fundamental method (AASHTO NDT 1) which goes back to first principles and is calculated from isotropic moduli as described in Appendix C. However, rather approximate correlations are also available, using the "New Zealand Regression":

$$SNP = 112\,(D_0)^{-0.5} + 47\,(D_0 - D_{900})^{-0.5} - 56\,(D_0 - D_{1500})^{-0.5} - 0.4 \qquad \textit{Equation 3.1}$$

where:

> D_0, D_{900} and D_{1500} are the deflections in microns at offsets of 0, 900 and 1500 mm respectively under a standardised 40 kN FWD impact load.

The New Zealand Regression and two other Australian regressions are already available for automatic calculation in RAMM.

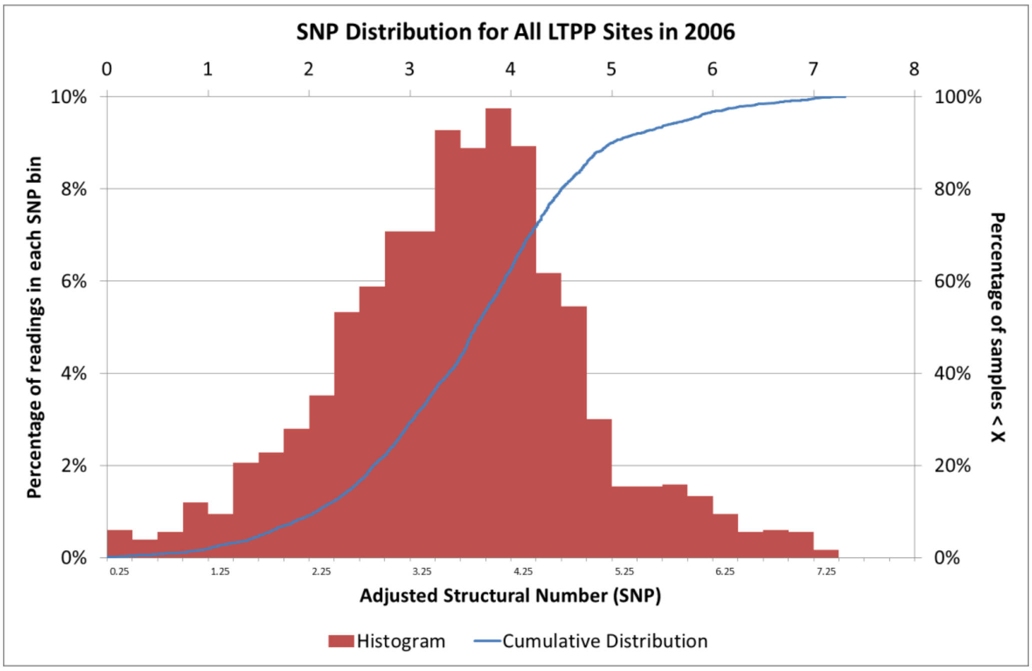


Figure 3.2 – Distribution of adjusted structural number (SNP) for all national LTPP sites.

The pavement structural life (N_{MODE}) determined for each distress mode from the mechanistic analysis is converted to the corresponding structural index (SI_{MODE}) using a transfer function. The form that proved suitable for this purpose (the Lorentzian cumulative function) has the following structure:

$$SI_{MODE} = a + \frac{b}{\pi}\left[\tan^{-1}\left(\frac{Log_{10}(N_{MODE}) - c}{d}\right) + \frac{\pi}{2}\right] \qquad \textit{Equation 3.2}$$

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 the following table.

Table 3.2 Function coefficients for structural indices based on NZTA LTPP sites

Mode	A	B	C	D
Rutting	-1.216	8.086	6.5	1.789
Roughness	-1.062	10.100	6.5	1.200
Cracking *	N/A	N/A	N/A	N/A
Flexure	-0.408	9.750	8.0	0.661
Shear	-1.368	10.440	8.11	2.231
The LTPP sites do not have any basecourse stabilisation or thick AC, therefore the cracking parameter is not applicable.				

The transfer functions have been derived from the complete set of national state highway LTPP sites. While they are evidently not following a normal (Gaussian) distribution, the end result is straightforward in concept. For local authority networks, where SNP has been used historically, the LTPP SNP distribution should be replaced with the local SNP distribution to generate the relevant function coefficients. This is straightforward in practice, and then minimal (if any) calibration is likely to be required with the changeover from SNP to the relevant set of structural indices. Because of the way SI values are derived, they are already calibrated to have the same range and distribution as the traditional SNP value for the network as a whole. It is only the relative ranking of individual roads that changes when the relevant governing distress mode of each road is evaluated.

The new approach will also readily allow other rational methods of determining the number of ESA to a specific terminal condition to be adopted. This allows the network model to be very simply upgraded at any future time when actual performance is observed and compared with predicted performance. The following example from one treatment length of an LTPP site on SH1 illustrates the type of variation between traditional and new parameters.

Table 3.3 Example structural indices from LTPP site BM01

Chainage	SNP	$SI_{RUTTING}$	$SI_{ROUGHNESS}$	$SI_{FLEXURE}$	SI_{SHEAR}
0.10	3.9	4.5	4.9	3.1	3.6
0.15	3.9	4.6	4.7	3.5	3.8
0.20	3.5	4.0	3.9	3.3	3.6

0.25	3.6	4.1	3.8	4.0	3.9
0.30	1.8	2.6	2.7	4.2	4.2
0.35	2.2	3.1	3.1	3.8	3.8

For the transition period (as the new approach is implemented), all structural indices can be readily generated from FWD data. If using SNP alone meets the accuracy required for a given network (when assessing structural deterioration and FWPs), then clearly no change is necessary. However, where the traditional approach is found to be limiting (ie. anywhere that rutting is not the dominant trigger for rehabilitation, or the "hit rate" for the dTIMS model is not suitably realistic), the upgrade can be made by substituting the relevant structural index in place of the SNP value for that form of distress. (Any number of indices may be used but using all of them should invariably give the best predictions.).

The following equation gives the inverse function for Equation 3.2:

$$Log_{10}(N_{MODE}) = d \cdot \tan\left[\frac{\pi(SI_{MODE} - a)}{b} - \frac{\pi}{2}\right] + c \qquad \textit{Equation 3.3}$$

It is important to appreciate that lifetime traffic predictions (N_{MODE}), as with SNP determinations, relate to the state of the pavement at the time of field testing. If a pavement is in its stable rut progression phase (Figure 3.3), layer moduli, stresses and strains under 1 ESA loading will be relatively invariant.

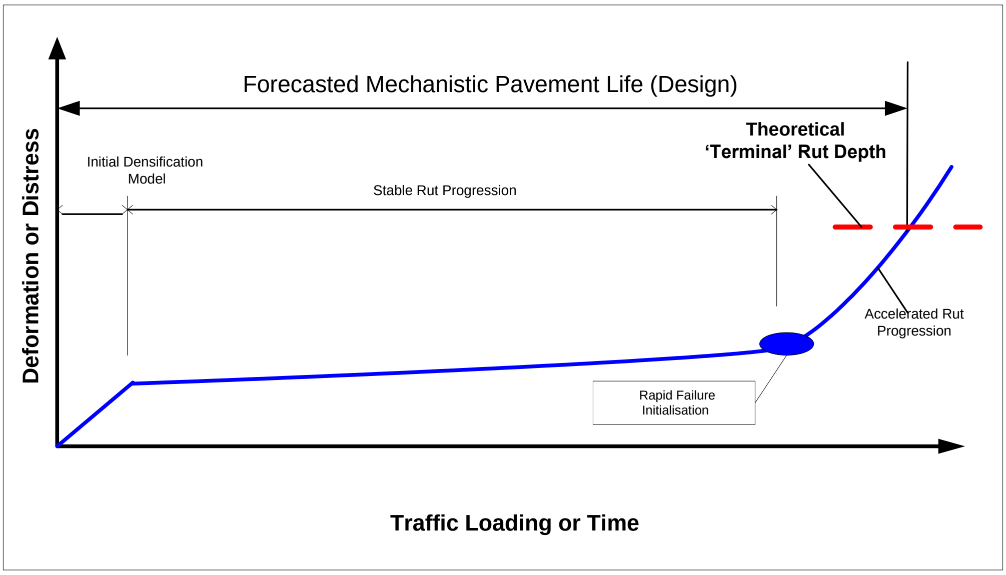


Figure 3.3 – Pavement Life Phases

In the stable phase, the indices for rutting, roughness, cracking and flexure are normally assumed to relate to the expected total life of the pavement. This is because these values are derived from the deflection bowl shape (which is expected to remain relatively constant for most of the service life). The values for the shear index are largely derived from indications of in-elastic (plastic) response in the full-time history of the FWD record at the time of test. These characteristics may change progressively and are therefore considered to relate to the remaining life.

For modified or bound materials, considerable study has been carried out (Gray et al, 2011) to understand the way in which layer moduli may change with temperature, time or traffic loading.

Further studies on these aspects are in progress to refine the calculation of the indices and improve predictions of pavement life. (Further explanation is given in Part II of this Guide.)

If the pavement is in the terminal stage, it should be visually apparent, so life prediction is not important. The same assumptions apply as for the stable phase, but the calculated total life will be a lower bound.

If the pavement is bedding in (initial densification phase), correction factors to increase the indices may be appropriate, or they may be taken as a conservative lower bound, but the best approach to avoid the issue is to delay structural testing until at least three (but preferably six) months of trafficking has been experienced on any new or rehabilitated pavement.

3.5 Determining Structural Indices for Incomplete Datasets

Using the results of FWD testing, SI values (or SNP if subgrade rutting is the dominant distress mode) can be determined at each test location. The values within a treatment length can be used to calculate a specific percentile (characteristic) value for that treatment length.

To provide interim structural capacity estimates over the rest of a network where there are no deflection measurements or other knowledge of layer properties, the dTIMS manual gives two options:

i. ARRB method – this assumes that the pavement is correctly designed for its intended traffic and estimates the SNP from design charts.
ii. Typical Pavement method – this consists of a series of typical pavement designs and subgrade strengths.

The ARRB method places considerable faith in the past standard of design as being appropriate for the future traffic, while the Typical Pavement Method is essentially a best guess of the pavement layering. Both methods have substantial limitations; however, where factual data are available from elsewhere on a given network, a more informed approach can be adopted for the unknown part of the network. The basic principle is to combine both the ARRB method and the Typical Pavement method and incorporate trends for structural capacity determined by deflection testing elsewhere on the network. A network is suited for this process if it has a substantial amount of FWD data already collected over the full range of pavement types.

The first step is to categorise all roads into firstly urban or rural, and secondly into the different pavement structural types (up to nine as described above in Section 3.2.2). While this gives potentially a total of 18 categories, in practice there may only be two or three rural categories and five or so urban.

Each category is individually partitioned. An effective means of doing this is to assemble a matrix for binning (classifying) the network according to Cumulative (Design) ESA on one axis, and Pavement Age on the other.

By dividing the number of FWD tests in each bin by the total km length for that bin, a simple but effective graph can be generated that shows the portions of the network that are under-represented and would be ideal for future network data gathering. An example of the data from a large city network is shown below.

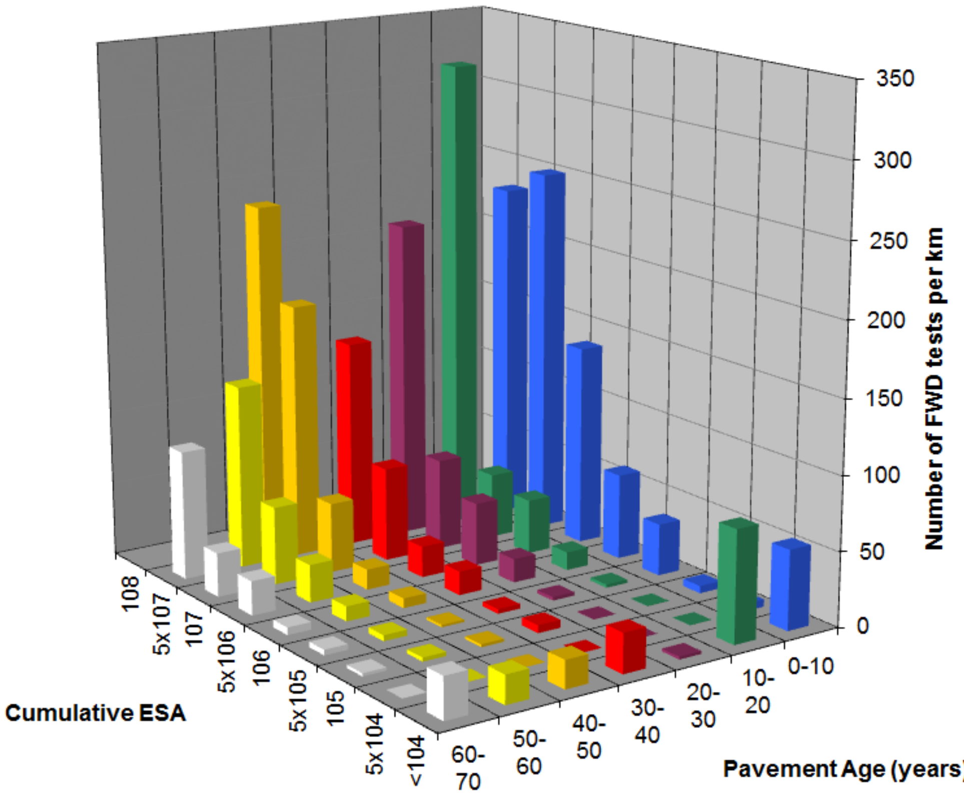


Figure 3.4 – FWD test distribution for a large city network

All of the FWD testing available is then batch reprocessed using the latest treatment length table (based on carriageway) from RAMM imported directly into the models. This ensures all of the data are determined using the same procedures. The calculated SI (or SNP) values are assembled, sorted into each bin, and the characteristic SI determined for each bin. Some data smoothing may be required to enable trends to be assessed.

A moving average filter passed over each data set will reduce or eliminate any anomalous peaks. Within any given time interval, the pavements with low design traffic would tend to have lower SI values than pavements with higher design traffic, so a custom-designed filter is then passed over the top to ensure that this condition is met by all data sets. The logic of this is that while design standards might change over time, at any particular year in the network's history, designers would be quite unlikely to design thicker roads for less traffic (ie. this incorporates to some extent the assumption made in the ARRB method).

The section of the network that has not been tested is then binned in the same manner as for that for the tested section, and the "characteristic" SI value for each bin is then used in the FWP covering the whole network.

It is important to note that the estimated parameters would give the probable value for any given treatment length. They can therefore only be used to assess the network's average performance <u>for a large number of roads</u>. The structural parameter (and therefore predicted performance) of any individual treatment length will generally fluctuate considerably either side of the mean value for the bin.

The following graphic presents a case history for a network where this procedure has been undertaken. Using the known pavement age (in years) and the cumulative design ESA, the contour plot can be used to determine a representative SI for whichever failure mode for which the plot is created.

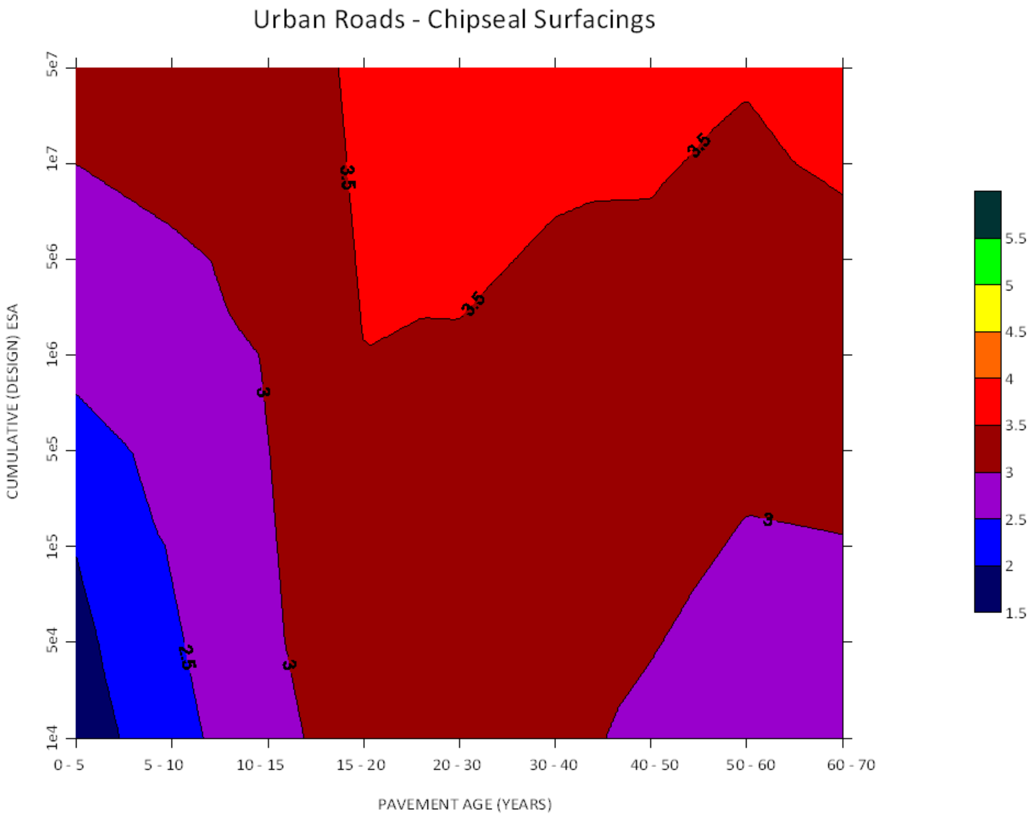


Figure 3.5 – Contour plot showing $SI_{Rutting}$ distribution

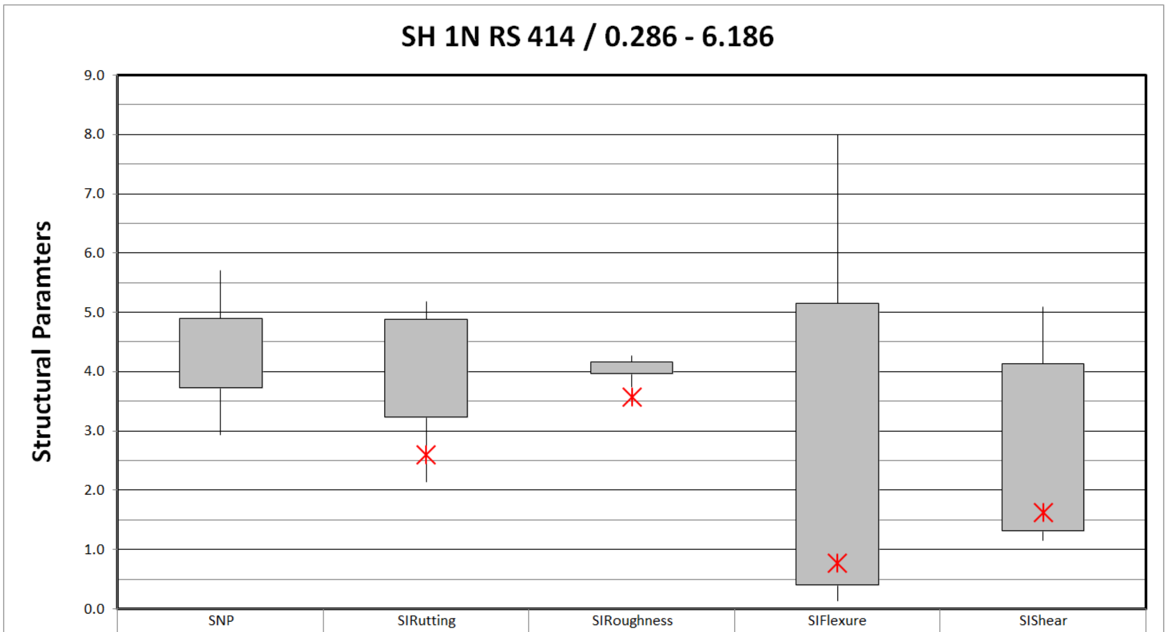
For this network, it is interesting to note that for any given current ESA, the trend for building roads historically appears to be a slight improvement in structural capacity with time up until a peak about 25 years ago, while more recently constructed roads show a marked reversal. Changing traffic loadings and increasing urbanisation will have contributed to this.

This procedure (here termed the "Characteristic Structural Index Method") should give markedly better extrapolation from limited data for general appraisal of the future performance of a roading network than either of methods 1 or 2 above.

3.6 Comparative SNP and Structural Indices Plots

Where applicable Structural Indices have been obtained from the section of road being tested, a box-and-whisker plot may also be used to present the results comparatively for each section as displayed in Figure 3.8 below.

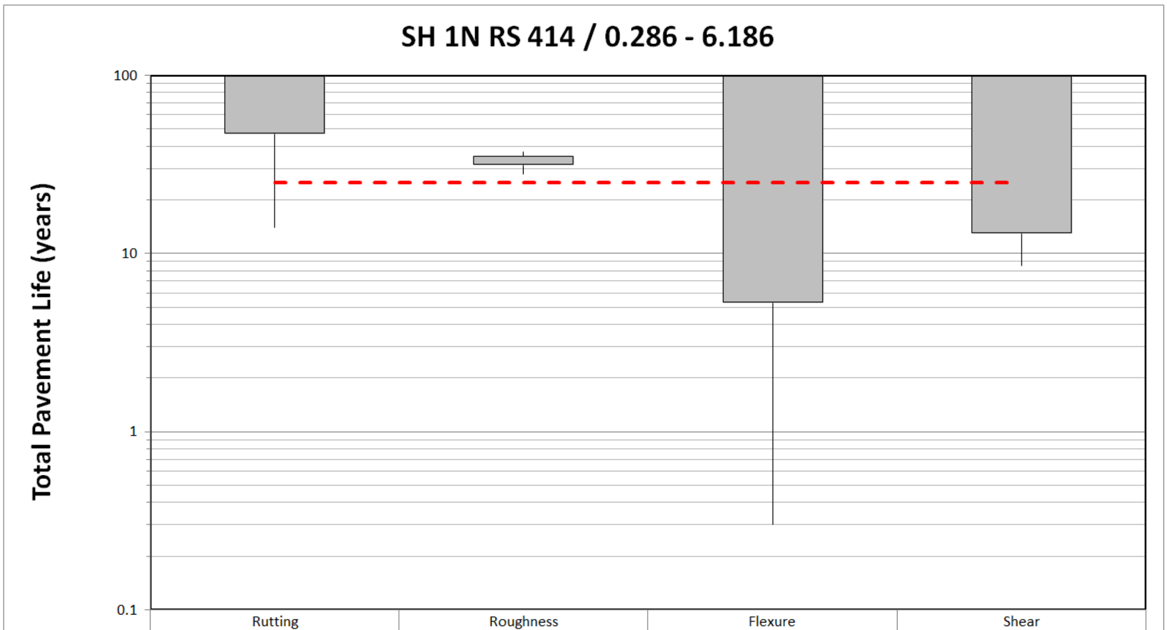
A point to note is that these plots will require differing interpretations depending on the road's traffic loading. For instance, a median SNP or SI of two may be quite adequate for a minor residential street with an AADT of <100, but for a busy urban arterial road, the same numeric values would indicate substantial structural inadequacy with improvements warranted. Therefore, for ease of application, the required SI (ie. the 10th percentile required if the 25-year life is to be obtained) for each distress mode can readily be calculated from the known lifetime ESA using equation 3.2. In general, the set of required SI values for a specific road will differ (as illustrated by the red asterisks in the following diagram).



	SNP	SIRutting	SIRoughness	SIFlexure	SIShear
10th%ile	3.72	3.24	3.98	0.40	1.32
Minimum	2.93	2.14	3.74	0.14	1.15
Maximum	5.70	5.18	4.26	8.00	5.10
90th%ile	4.90	4.87	4.17	5.15	4.14
SIMODE		2.60	3.57	0.77	1.62

Figure 3.8- Comparative plot of adjusted structural number (SNP) and pavement structural indices (SI) from an FWD survey of a section of SH1 near Auckland.

An alternative representation is to plot total life for each distress mode in place of the structural index, as shown below.



	Rutting	Roughness	Flexure	Shear
10th%ile	47.38	31.80	5.32	13.10
Minimum	14.00	27.80	0.30	8.50
Maximum	99.00	37.10	99.00	99.00
90th%ile	99.00	35.27	99.00	99.00
25 Years	25.0	25.0	25.0	25.0

Figure 3.9- Comparative plot of the total pavement life for each structural index distress mode

4 Application

4.1 The RAMM Database and Performance Modelling

RAMM data stores only peak FWD deflection data, rather than the full-time history of each test. Since 2010, it has been made standard practice to retain the full-time history for all FWD data collected in New Zealand. This involves no additional cost and has the advantage that for pavements with thin surfacings, the relevant distress mode can be predicted more reliably.

It is important to identify on the database whether SNP/SI values results have been generated from regression equations, or more rigorously through mechanistic analysis, to appreciate the reliability of performance predictions.

The following is a link showing where SNP can be determined in RAMM:

http://www.cjntech.co.nz/manuals/Working%20with%20RAMM/index.htm#12670.htm

Structural indices may be:

i. Supplied by the FWD provider after mechanistic analysis
ii. Calculated by dTIMS analysts after mechanistic analysis as outlined in this Guide
iii. Generated by regressions currently being developed so they can be generated within RAMM.

The regression method is recommended only for preliminary studies. However, it is important that a consistent (single) method is always used throughout any individual network under consideration, to facilitate any subsequent calibration.

4.2 Network Calibration

Each structural index is mechanistically derived and has the same range and general distribution as the traditional SNP. This allows straightforward implementation (substituting the relevant structural index for SNP) with minimal additional calibration needed for any model that is based on Structural Number (ie. 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.

Reliability of any FWP for a network can be improved by using observations of precedent performance. The relevant proportions of the five different structural distress modes experienced historically should be at least similar to that predicted in the future. This may come from records of past experience, or by systematic survey of the currently exhibited distress modes. Assuming the triggers used in the model are appropriate, then if past and future proportions of distress differ significantly, rates of deterioration adopted for each distress mode may need adjustment to give consistency.

4.3 Distress Modes and their Treatments

The prime advantage of SI over SNP is that the deterioration model will identify the structural distress mode that will trigger rehabilitation. This, in turn, will allow the most cost-effective treatment to be determined for the FWP. Where flexure causes the terminal condition, only the surfacing will need to be addressed. Where cracking is the trigger, it will require the top structural layer to be rehabilitated. Shear instability usually originates in the top structural layer, but the subbase is occasionally a source also. Roughness and rutting are often sourced in the subgrade, but sometimes there is significant contribution from thick aggregate layers.

The SIs provide a simple means for designers to consider more than just the treatment to address the expected terminal distress in the current lifetime. By considering all five structural distress modes, designers can also appreciate what treatment would optimise the next life cycle.

5 Summary

- Top surface type, FWD peak deflection data, peak plate stress and top surface temperature at the time of testing are the principal prerequisite fields required for network level analysis. However, more effective performance prediction for each distress mode will be obtained by recording the full-time history of the FWD tests.
- To complement the FWD data, other information is also preferable for comprehensive mechanistic analysis (see Table 3.1). Much of this can often be inferred from the FWD data, operator observations or the RAMM database.
- Recommended sampling with FWD for network evaluation is indicated in Table 2.1.
- Destructive testing is not essential. It is seldom warranted or cost effective for network level evaluation but is usually required for project level evaluation where rehabilitation is imminent.
- Pavement deflection data should be collected progressively across the full network. In the initial years, for cost-effective testing and modelling, the network may be subdivided on the basis of terrain, geology and road type to enable representative data to be collected. Techniques for extrapolating the representative samples to enable preliminary performance monitoring of the full network is given in section 3.1.
- Mechanistic evaluation of the roading network is recommended, unless simple empirical methods are already providing suitably reliable FWPs. The advantage of mechanistic structural evaluation and design methods over more empirical methods is that the former may be used with any type of material and structure and under all climatic conditions (once fatigue criteria are established for each material type), whereas the latter may be applicable only under the conditions for which the empirical relationships were developed.
- A simple way in which users of dTIMS or HDM models can readily make the shift from the empirical SNP to mechanistically derived structural indices is given section 3.4.
- Adopting a mechanistic approach for the structural evaluation of long-term pavement performance provides the setting for continuous improvement as more case histories become available. This in turn allows better definition of immediate remedial work actions, increasingly reliable prediction of both the remaining service life and terminal distress mode, resulting in more realistic FWPs and the most effective use of a limited roading budget.

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Appendix A – RAMM FWD Data Formats

- **Data Format Table**
- **Header Format Table**

Falling Weight Deflectometer – Data Format				
Field	**Description**	**Field Information**		**Remarks**
1.	Road ID	Integer	(6)	Unique identifier of the road
2.	Road Name	Char	(35)	Name of road
3.	Survey Number	Integer	(5)	
4.	Latest	Char	(1)	
5.	Region	Smallint	(2)	SH Region No (1-14)
6.	Network Contract Area	Char	(15)	SH Network Management Area
7.	State Highway	Char	(3)	SH Number
8.	Reference Station	Char	(4)	SH Reference number
9.	Direction	Char	(1)	Flag for divided carriageways. I, D or Null
10.	Ramp	Char	(8)	Motorway On or Off
11.	Date of Test	Date		Date of test
12.	Location	Integer	(5)	Displacement from the start of the road where the reading was taken in metres
13.	Lane	Char	(2)	Lane indicator. Format is Ln, Rn where n is a numeric
14.	Offset	Decimal	(3,1)	Offset from the centreline
15.	NZMG North	Decimal	(12,4)	NZMG north value of the reading site
16.	NZMG East	Decimal	(12,4)	NZMG east value of the reading site
17.	Elevation	Integer	(4)	Elevation (m above sea level) of the reading site
18.	Pressure	Integer	(4)	Pressure reading (in kPa)
19.	Temperature	Integer	(2)	Temperature reading (in Celsius)
20.	Sensor Reading d0	Integer	(4)	Sensor reading at displacement 0 (in microns)
21.	Sensor Reading d1	Integer	(4)	Sensor reading at displacement 1 (in microns)
22.	Sensor Reading d2	Integer	(4)	Sensor reading at displacement 2 (in microns)
23.	Sensor Reading d3	Integer	(4)	Sensor reading at displacement 3 (in microns)
24.	Sensor Reading d4	Integer	(4)	Sensor reading at displacement 4 (in microns)
25.	Sensor Reading d5	Integer	(4)	Sensor reading at displacement 5 (in microns)
26.	Sensor Reading d6	Integer	(4)	Sensor reading at displacement 6 (in microns)
27.	Sensor Reading d7	Integer	(4)	Sensor reading at displacement 7 (in microns)
28.	Sensor Reading d8	Integer	(4)	Sensor reading at displacement 8 (in microns)
29.	Sensor Reading d9	Integer	(4)	Sensor reading at displacement 9 (in microns)
30.	Distress Code	Char	(1)	Distress code
31.	Surface Description	Char	(15)	Description of the surface
32.	SNP	Decimal	(4,2)	Adjusted Structural Number
33.	SNP Method	Char	(5)	Method used to calculate the SNP
34.	SNP Organisation	Char	(3)	
35.	Notes	Char	(255)	

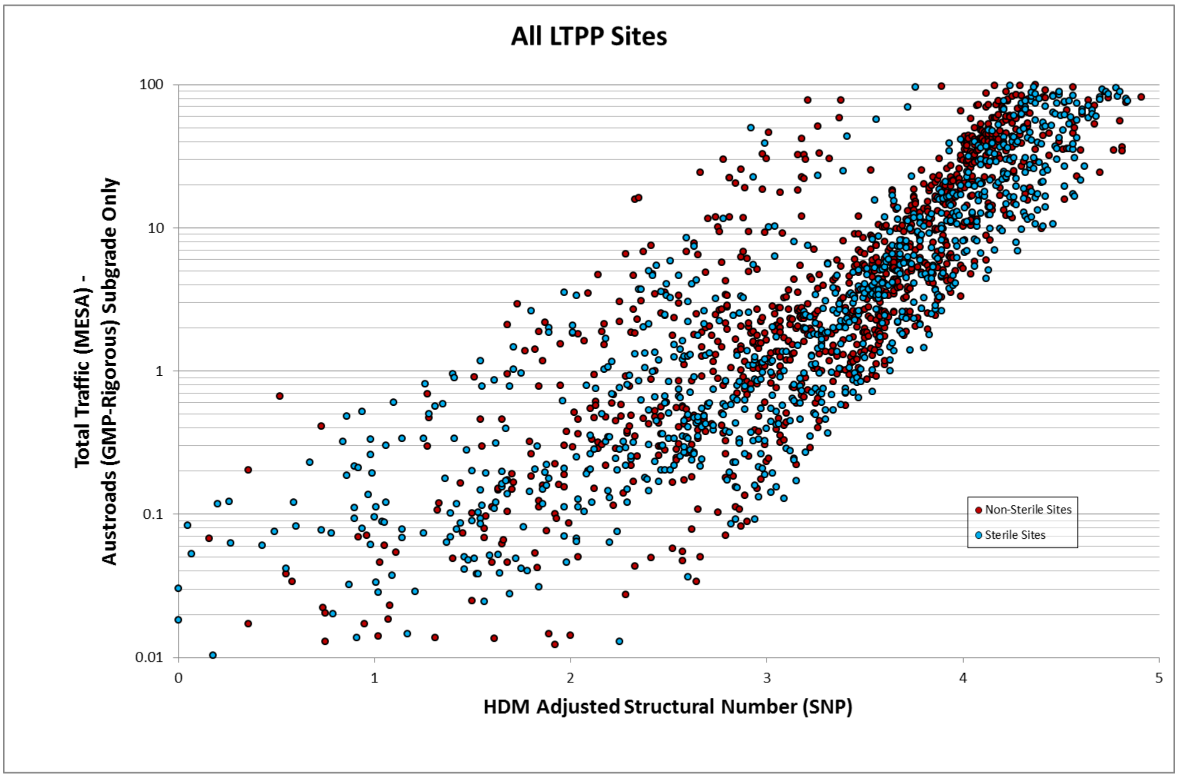
Falling Weight Deflectometer - Header Format			
Field	**Description**	**Field Information**	**Remarks**
1.	Survey Number	Integer (5)	
2.	Survey Description	Char (50)	Description of the survey
3.	Contract Number	Char (18)	Contract number for the survey
4.	Project ID	Char (10)	Project identifier
5.	Project Name	Char (30)	Project Description
6.	Notes	Char (255)	Free format notes
7.	Start Date	Date	Date survey started
8.	End Date	Date	Date survey ended
9.	Financial Year	Char (7)	Financial year of the survey
10.	Supplier	Char (30)	Supplier
11.	Machine ID	Char (30)	Machine ID of the equipment which completed the survey
12.	Plate Radius	Smallint (4)	Plate radius used in the survey
13.	Seasonal Factor	Decimal (3,1)	Seasonal adjustment factor
14.	Sensor Displacement 1	Smallint (4)	Displacement from the centre point of sensor 1
15.	Sensor Displacement 2	Smallint (4)	Displacement from the centre point of sensor 2
16.	Sensor Displacement 3	Smallint (4)	Displacement from the centre point of sensor 3
17.	Sensor Displacement 4	Smallint (4)	Displacement from the centre point of sensor 4
18.	Sensor Displacement 5	Smallint (4)	Displacement from the centre point of sensor 5
19.	Sensor Displacement 6	Smallint (4)	Displacement from the centre point of sensor 6
20.	Sensor Displacement 7	Smallint (4)	Displacement from the centre point of sensor 7
21.	Sensor Displacement 8	Smallint (4)	Displacement from the centre point of sensor 8

Appendix B – SNP Limitations

SNP Limitations

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 (AASHTO, 1986). In the 1980s and 90s, structural number or modified structural number (SNC) 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 and dTIMS still retain the concept as either modified structural number (SNC) or 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 highway sites in New Zealand is shown in the following Figure 1.



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. For example, reading off the range of life for a mid-scale (SNP=3) value from the above graph, gives values ranging from about 0.1 years to 40 years. 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 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, cracking 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 of its introduction more than 50 years ago, but its nature precludes any progression of the state of the art. The above figure is clear evidence that it is inappropriate to use SNP alongside Austroads mechanistic principles. A search for any literature (from New Zealand or elsewhere) to the contrary was unsuccessful. The use of SNP does not acknowledge all the advances in pavement engineering in general and mechanistic analysis in particular since 1990.

Structural number was promoted in New Zealand in 1999 solely to support the dTIMS system at a time when many other countries were discarding it. The introduction of SNP was a large backward step for New Zealand pavement engineering because it has precluded any advancement of the state-of-the-art, and made calibrating predicted to observed performance a difficult task. To enable an immediate transition for dTIMS or HDM users to obtain the benefits of mechanistic principles, the simple concept of a set of structural indices has been established, and these parameters are made available freely for all FWD testing currently carried out in New Zealand. Other mechanistic approaches are also available.

References

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Appendix C – SNP Determination