Collection and Interpretation of Pavement Structural Parameters using Deflection Testing

PART I: NETWORK ASSET MANAGEMENT

DECEMBER 2012
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  Structural number concept [or variations such as the modified structural number or adjusted structural number]
TSD  Traffic Speed Deflectometer
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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 on-going 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 its 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. While SNP is simple to calculate, it is of limited reliability in associated predictions (such as Forward Work Programmes – FWPs) 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 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.

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\(^1\) Austroads is the association of Australian and New Zealand road transport and traffic authorities.
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.

![Image of FWD trailer showing deflection bowl recorded at geophone offsets.](image)

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.
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 (ESG).

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.

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

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.

2.4 **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.

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).

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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.

<table>
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<tr>
<th>Category</th>
<th>Information</th>
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</thead>
<tbody>
<tr>
<td>1. Essential</td>
<td>FWD peak deflection data, peak plate stress, Pavement temperature at time of test (only used for asphaltic layers)</td>
</tr>
<tr>
<td>2. Essential, but can be assessed by FWD structural analyst</td>
<td>Subgrade type (volcanic ash or otherwise)</td>
</tr>
<tr>
<td>3. Important, but can be addressed by FWD operator</td>
<td>Surfacing type, Top structural layer type, Full time history (dynamic record of all sensors while the FWD load is applied)</td>
</tr>
<tr>
<td>4. Preferable (but can often be deduced by FWD analyst)</td>
<td>Nature and thickness of any bound stabilised layers, Depth to subgrade, Weighted mean annual pavement temperature (WMAPT °C)</td>
</tr>
<tr>
<td>5. Preferable (should be readily available in RAMM)</td>
<td>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</td>
</tr>
<tr>
<td>6. Desirable to ensure model consistency</td>
<td>Rut depths after bedding in, Current rut depths, Roughness after bedding in, Current roughness, Recent visual survey(s) especially cracking</td>
</tr>
<tr>
<td>7. Desirable for improved calibration</td>
<td>Historic rut depth progression, Historic roughness progression, Historic maintenance costs</td>
</tr>
</tbody>
</table>

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:

- Rutting is the governing distress mechanism (i.e. no other trigger for rehabilitation applies)
- The majority of the rutting occurs in the subgrade rather than the overlying layers
- The treatment length is correctly defined and relates to a uniform sub-section
- Rather than simply taking the average SNP, an appropriate percentile for SNP is determined corresponding to the percentage of road in a terminal condition (i.e. 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 [i.e. 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 [e.g. 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 (i.e. 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:

- Download FWD data and pavement condition data from RAMM
- Obtain additional FWD full time history from FWD provider (optional)
- 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 (i.e. stress dependent), software that accommodates this characteristic properly is preferred. [Further discussion is given in Part II of this guide.]
- 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
- 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.
- Determine the traditional SNP values for all available data.
- 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.)
- 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.](image)

This provides the number of ESA To a terminal rutting condition (NRUTTING) 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 NCRACKING 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":

\[
\text{SNP} = 112 \left( D_0 \right)^{-0.5} + 47 \left( D_0 - D_{900} \right)^{-0.5} - 56 \left( D_0 - D_{1500} \right)^{-0.5} - 0.4
\]

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.

*Equation 3.1*

The New Zealand Regression and two other Australian regressions are already available for automatic calculation in RAMM.
The pavement structural life (N \text{MODE}) determined for each distress mode from the mechanistic analysis is converted to the corresponding structural index (SIM \text{MODE}) using a transfer function. The form that proved suitable for this purpose (the Lorentzian cumulative function) has the following structure:

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

Equation 3.2

where:

a, b, c and d are constants derived from the optimisation of the distribution of N \text{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.

<table>
<thead>
<tr>
<th>Mode</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rutting</td>
<td>-1.216</td>
<td>8.086</td>
<td>6.5</td>
<td>1.789</td>
</tr>
<tr>
<td>Roughness</td>
<td>-1.062</td>
<td>10.100</td>
<td>6.5</td>
<td>1.200</td>
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<tr>
<td>Cracking *</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Flexure</td>
<td>-0.408</td>
<td>9.750</td>
<td>8.0</td>
<td>0.661</td>
</tr>
<tr>
<td>Shear</td>
<td>-1.368</td>
<td>10.440</td>
<td>8.11</td>
<td>2.231</td>
</tr>
</tbody>
</table>

*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.

<table>
<thead>
<tr>
<th>CHAINAGE</th>
<th>SNP</th>
<th>SIRUTTING</th>
<th>SIROUGHNESS</th>
<th>SIFLEXURE</th>
<th>SISHEAR</th>
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<tbody>
<tr>
<td>0.10</td>
<td>3.9</td>
<td>4.5</td>
<td>4.9</td>
<td>3.1</td>
<td>3.6</td>
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<tr>
<td>0.15</td>
<td>3.9</td>
<td>4.6</td>
<td>4.7</td>
<td>3.5</td>
<td>3.8</td>
</tr>
<tr>
<td>0.20</td>
<td>3.5</td>
<td>4.0</td>
<td>3.9</td>
<td>3.3</td>
<td>3.6</td>
</tr>
<tr>
<td>0.25</td>
<td>3.6</td>
<td>4.1</td>
<td>3.8</td>
<td>4.0</td>
<td>3.9</td>
</tr>
<tr>
<td>0.30</td>
<td>1.8</td>
<td>2.6</td>
<td>2.7</td>
<td>4.2</td>
<td>4.2</td>
</tr>
<tr>
<td>0.35</td>
<td>2.2</td>
<td>3.1</td>
<td>3.1</td>
<td>3.8</td>
<td>3.8</td>
</tr>
</tbody>
</table>
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 (i.e. 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:

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

**Equation 3.3**

It is important to appreciate that lifetime traffic predictions \( N_{\text{MODE}} \), 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.](image)

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 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:

- **ARRB method** – this assumes that the pavement is correctly designed for its intended traffic and estimates the SNP from design charts.
- **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.
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 (i.e. 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 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 for a large number of roads. 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.

Figure 3.4: FWD test distribution for a large city network.
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 SIRutting 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 (i.e. the 10 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).
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.

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.cjintech.co.nz/manuals/Working%20with%20RAMM/index.htm#12670.htm

Structural indices may be:
- Supplied by the FWD provider after mechanistic analysis
- Calculated by dTIMS analysts after mechanistic analysis as outlined in this Guide
- 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 (i.e. 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.
6. Bibliography


Sparks, GH & Potter, DW 1982, An investigation into the relationship between California bearing ratio and modulus for two clays, Internal Report AIR 295 1, Australian Road Research Board, Vermont South, Vic.


# Appendix A

**RAMM FWD Data Formats**

Data Format Table

<table>
<thead>
<tr>
<th>Field</th>
<th>Description</th>
<th>Field Information</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Road ID</td>
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<td>Unique identifier of the road</td>
<td></td>
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<tr>
<td>2. Road Name</td>
<td>Char (35)</td>
<td>Name of road</td>
<td></td>
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<tr>
<td>3. Survey Number</td>
<td>Integer (5)</td>
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<td></td>
</tr>
<tr>
<td>4. Latest</td>
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<td>Smallint (2)</td>
<td>SH Region No (1-14)</td>
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<td>Char (15)</td>
<td>SH Network Management Area</td>
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<tr>
<td>8. Reference Station</td>
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<td>SH Reference number</td>
<td></td>
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<tr>
<td>9. Direction</td>
<td>Char (1)</td>
<td>Flag for divided carriageways. I, D or Null</td>
<td></td>
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<tr>
<td>10. Ramp</td>
<td>Char (8)</td>
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<tr>
<td>11. Date of Test</td>
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<td>Date of test</td>
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<td>Integer (5)</td>
<td>Displacement from the start of the road where the reading was taken in metres</td>
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<td>13. Lane</td>
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<td>Lane indicator. Format is Ln, Rn where n is a numeric</td>
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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 ESAs 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 \(\text{SNP}=3\) 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


APPENDIX C
SNP DETERMINATION
PAVEMENT PERFORMANCE PREDICTION
Determination and Calibration of Structural Capacity (SNP)

Graham Salt & David Stevens, Tonkin & Taylor Ltd
20th ARRB Conference Melbourne, March 2001
ABSTRACT
Realistic prediction of pavement performance is a critical component of asset management. Performance in terms of structural capacity is generally measured by the Adjusted Structural Number SNP, which is why a review has been carried out testing alternative methods for deriving this parameter for unbound granular pavements. Simplified methods may work reasonably well when calibrated to typical local conditions, but they are less likely to provide the same reliability in different regions or with different pavement structures. The rigorous methods (which can be readily applied) are recommended wherever deflection bowl information has been recorded. However, it is important to note the standard equations are based on isotropic moduli for each layer. This is different from the anisotropic moduli, which the Austroads Pavement Design Guide has adopted for mechanistic analysis. Typical field measured moduli for unbound granular pavements are presented and appropriate calculation procedures suggested.

Calibrating or correcting SNP may also be required in a range of circumstances when refining the structural capacity for use in long-term pavement performance prediction. Where representative benchmark sites are set up and actual performance is monitored, appropriate calibration factors, based on specific modes of distress can be developed. These result in effective SNP values for a given network, giving due regard to the range of locally available materials and construction techniques.

INTRODUCTION
To implement the long term planning of road management (using systems such as dTIMS), parameters are required to define the structural capacities of the various pavements which make up each road network. The Modified Structural Number (SNC) or SNP are widely used for this purpose. There have been numerous studies to find simple methods for calculating these parameters using both destructive and non-destructive tests. As most studies relate to thick structural asphaltic pavements, a literary review of completed projects has been carried out. The review aimed to determine the validity of alternative procedures used for pavements containing unbound granular basecourses with chip-seal surfacing.

Because structural capacity is determined as a single parameter, it does not always characterise pavements which do not exhibit “typical” properties. Calibration is essential for these atypical pavements so meaningful long-term pavement performance prediction can be obtained. Studies carried out over the last decade have identified several specific pavement types where calibration is essential; as a result, approaches have been developed to provide suitably adjusted measures of structural capacity.
BACKGROUND

The Modified Structural Number is defined as linear combination of the layer strength coefficients \( a_i \) and thicknesses \( H_i \) of the individual layers above the subgrade, and a contribution from the subgrade denoted by SNSG (Paterson, 1987; Watanatada et al, 1987) for the Highway Design and Maintenance Standards (HDM) model.

\[
\text{SNC} = \left( \frac{1}{25.4} \right) \sum_{i=1}^{n_{\text{layer}}} a_i H_i + \text{SNSG} \tag{1}
\]

Where \( a_i \) is the strength coefficient of the \( i^{th} \) layer as defined by Watanatada et al (1987)
\( H_i \) is the thickness in millimetres of the \( i^{th} \) layer provided that the sum of thicknesses \( H_i \) is not greater than 700 mm
\( n \) is the number of pavement layers

SNSG is the modified structural number contribution of the subgrade, given by:

\[
\text{SNSG} = 3.5 \log_{10} \text{CBR} - 0.85(\log_{10} \text{CBR})^2 - 1.43 \tag{2}
\]

CBR is the California Bearing Ratio of the subgrade at in situ conditions of moisture and density. If the Falling Weight Deflectometer (FWD) has been used then the CBR is usually calculated from:

\[
\text{CBR} = \frac{E_s}{10} \tag{3}
\]

where \( E_s \) is the isotropic subgrade modulus in MPa.

In the 1986 AASHTO Guide, layer coefficients may be determined from CBR, modulus and other parameters. Non destructive testing was soon introduced for pavement rehabilitation design. Paterson (Figure 1) found that Benkelman Beam readings (standardised to a 40 kN wheel load) gave an approximate relationship between central deflection and SNC, of the form:

\[
\text{SNC} = 3.2 D_B^{-0.63} \tag{4}
\]

if the base is unbound, or

\[
\text{SNC} = 2.2 D_B^{-0.63} \tag{5}
\]

if the base is cemented. \( D_B \) is the Benkelman Beam reading in mm.

Paterson's test data were limited to pavements with deflections generally less than 2 mm.

It was found that Paterson's equation for SNC over predicted the capacity of pavements with thicknesses over 700 mm. Hence the Adjusted Structural Number (SNP) is used in HDM4. SNP applies a weighting factor which reduces with increasing depth, to subbase and subgrade contributions so the pavement strength for deep pavements is not over predicted. (Parkman and Rolt, 1997). However, as adjustments of a similar nature have commonly been used for at least the last decade when calculating SNC, the terms are used synonymously in this paper.
After Paterson's study, many relationships were developed using the FWD, and these are summarised by Rohde & Hartman, (1996). Various alternative methods, which use the deflections from specific offsets on the deflection bowl, were examined for unbound granular pavements. It may be expected that these methods will work well when calibrated to typical local conditions, but they are less likely to provide the same reliability in different regions or with different pavement structures.

The most rigorous of the FWD methods is where the layer coefficients are determined from a full analysis of the deflection bowl. Known or reasonable inferred layer thicknesses are used to back-calculate the layer moduli. The latter are then compared with values for materials used in the AASHO Road Test using Table 1 (Rohde & Hartman, 1996). Referred to in this document as the “AASHTO NDT 1” Method, this is used as the baseline in the current study. The procedure fits well with layered elastic theory because the same rational concept relating layer stiffnesses to the cube root of the layer modulus is applied, ie:

\[ a_i = a_g \left( \frac{E_i}{E_g} \right)^{0.33} \]  

\( a_g \) is the layer coefficient of standard material (from the AASHO Road Test) as listed in Table 1
\( E_i \) is the isotropic resilient modulus of the layer
\( E_g \) is the isotropic resilient modulus of standard material (from the AASHO Road Test)

**Table 1: Layer coefficients and resilient moduli of standard materials from the AASHO Road Test (Rohde & Hartman, 1996).**

<table>
<thead>
<tr>
<th>Layer Type</th>
<th>Layer Coefficient</th>
<th>Resilient Modulus ( E_g ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphaltic concrete</td>
<td>0.44</td>
<td>3100</td>
</tr>
<tr>
<td>Unbound basecourse</td>
<td>0.14</td>
<td>207</td>
</tr>
<tr>
<td>Subbase</td>
<td>0.11</td>
<td>104</td>
</tr>
</tbody>
</table>

Table 1 is based on isotropic moduli for all layers and Poisson’s ratio of 0.35. However, the Austroads Guide (1992) recommends all unbound layers including the subgrade should be modelled as anisotropic layers with horizontal modulus half of the vertical modulus. Granular layers should be assumed to have a Poisson’s ratio of 0.35, while cohesive subgrades should use a Poisson’s ratio of 0.45. Therefore the significance of the alternative assumptions are discussed in the following section.

**IMPLICATIONS OF MODULUS ANISOTROPY**

The majority of software packages for back-analysing FWD deflection bowls (Ullidtz & Coetzee, 1995) assumes isotropic moduli for all layers. The practical reason for this is only vertical deflections are measured with commonly available equipment. The horizontal moduli (transverse and along the wheelpath) and horizontal components of Poisson’s ratio are usually unknown and cannot be readily determined.

A study was carried out using FWD data from a series of new unbound granular pavements constructed of known good quality basecourses and sub-bases. No structural AC layers were present. The objectives were to (a) determine appropriate isotropic moduli currently being achieved in the upper 100-150 mm of good quality unbound basecourses, (b) relate these to reported anisotropic values suggested in the Austroads Guide (1992, Table 6.4), and (c) consider the implications for determination of SNP.

Because the achievable modulus of any unbound granular layer is dependent on the stiffness of the underlying layers (Shell, 1978), basecourse moduli (using ELMOD
Suggested conservative approximations (near the lower bounds of the data as shown in Figures 2 and 3) that represent reasonable design values are:

\[ E_{\text{isotropic}} = 220 \left( \text{SNC} \right)^{0.57} \quad (7) \]

\[ E_{\text{isotropic}} = 330 \left( D_0 \right)^{-0.5} \quad (8) \]

Where \( E \), the back-calculated modulus, is in MPa and \( D_0 \), the FWD central deflection under 40 kN loading (550 kPa plate stress), is in mm. Austroads (1992) recommends modelling unbound granular layers as anisotropic materials with vertical modulus \( E_{v,n=2} \) of twice the horizontal modulus. Applying the relevant correction (from Ullidtz, 1987; Tonkin & Taylor, 1998) gives:

\[ E_{v,n=2} = 290 \left( \text{SNC} \right)^{0.57} \quad (9) \]

\[ E_{v,n=2} = 440 \left( D_0 \right)^{-0.5} \quad (10) \]

These relationships could also be used for the mechanistic design of unusual pavements (ie those not determined more simply through the use of the Austroads Guide (1992, Fig 8.4)). An iterative approach would be needed because both SNC and \( D_0 \) would need to be calculated after a pavement is designed. However, as a starting point a preliminary estimate for a design basecourse modulus can be readily related to design ESA. Using central deflection alone, the Austroads Guide (1992, Fig. 10.3 Curve 1) may be adopted to determine a relationship between ESA and Benkelman Beam deflection of a structurally adequate pavement. After correcting for Beam/FWD ratio (Eqn. 11 below) the resulting design modulus values can then be assessed from Figure 4.

The anisotropic results conform well with the wide ranges of vertical moduli suggested by Austroads (1992, Table 6.4a) but allow the designer a more systematic approach. The relationships given do not take account
of stress dependency of the basecourse modulus. However, because the mean principal stress adopted in these FWD tests was slightly less than the in-service stress under 1 ESA, the values shown should be slightly conservative. Note that beneath a moderately thick structural AC layer, loadspread would result in moduli about 50-75% of the above. The moduli presented, apply only to the top 100 -150 mm of good quality unbound granular basecourse in a chip-sealed pavement designed to Austroads principles.

Different results may well be obtained using other methods of assessment – eg repeated load triaxial testing, particularly if confining stresses adopted are other than those applicable to the field situation. Therefore, it is important to use a consistent approach which in the case of this FWD based derivation of parameters is the use of the same methods for back-analysis as for subsequent forward-analysis during design.

The key steps indicated for determining SNP values from FWD testing and structural evaluation are:

1. Back-analyse for isotropic moduli (or convert appropriately for anisotropic values) before determining layer coefficients from Equation 6.
2. Distinguish whether the top layer is unbound or bound (eg cement stabilised) from Figure 2 or 3 before using Equation 6, (or select between Equations 4 and 5).

**METHODS FOR DETERMINING STRUCTURAL CAPACITY OF UNBOUND GRANULAR PAVEMENTS**

**Benkelman Beam – SNP**

Various studies have been carried out where both FWD and Beam data have been recorded and an approximate correlation between them is given in Transfund Report 117 (Tonkin & Taylor, 1998), ie

\[
\text{If } D_0 < 1 \text{ mm then } D_B = 1.1 \times D_0 \text{ otherwise } D_B = 1.1 \times D_0^{1.4} \quad (11)
\]

where \( D_B \) is the Benkelman Beam reading (mm) under an 80 kN standard axle load

\( D_0 \) is the FWD central deflection (mm) standardised to 40 kN loading (550 kPa stress)

The FWD is a dynamic test while the Benkelman Beam is used in a semi-static mode. For this reason correlation between the two tests is poor in view of the time-dependent and viscous stress-strain effects in the various layer materials. Nevertheless, for preliminary deflection data comparison , the equivalent Benkelman Beam deflections (using Eqn. 11) have been plotted against SNP “AASHTO NDT1” values for a number of newly laid unbound granular pavements where good layer thickness data were available. Most deflections were less than 2 mm (the range applicable to Paterson’s data) and a good correlation was found in this range with Paterson’s relationship, (Eqn 4).
Subsequently a range of older pavements including some on volcanic ash subgrades was included with the data set. Ash subgrades frequently produce Benkelman Beam deflections of 4 mm or more yet still perform adequately on moderate design traffic loadings. The full data set is shown in Figure 5, (using the axes originally shown by Paterson) along with the curve for Paterson’s relationship and also a best fit line to the New Zealand data. Not surprisingly, Paterson’s data do not extrapolate too well beyond his 2 mm data limit and Equation 3 tends to slightly underestimate SNP at high deflections. The correlation appears relatively good ($r^2 > 0.9$), but this is caused by the use of converted FWD central deflections rather than Benkelman Beam readings for which Paterson found a relatively poor correlation ($r^2=0.56$). The best fit to the data collected in the present study is:

$$\text{SNP} = 3.2 \, D_B^{-0.5} \quad (12)$$

where $D_B$ is the Benkelman Beam deflection in millimetres under an 80 kN standard axle load.

Ideally, a field study should be carried out using both the FWD and Benkelman Beam on volcanic ash pavements with high deflections. However, Equation 12 should provide the best estimate in the interim for assigning Adjusted Structural Numbers where organisations hold historic Benkelman Beam data. It must be appreciated that the predictions will not be good, ie $r^2$ about 0.5 to 0.6, with standard error of about 1.2 on SNP. Other disadvantages with Benkelman Beam data are that the distress mode (problems in individual layers) cannot be identified, nor can the presence of stabilised layers be inferred, so some as-built information will be essential to determine which of Equations 4 or 5 is appropriate.

Figure 5: Beam -SNP Correlation for NZ Unbound Pavements (cf HDM III)

SNP $= 3.2 \, BB^{-0.5}$
FWD – SNP Existing Regression Relationships

When determining structural capacity for dTIMS, three methods have been considered for estimating SNP from regression of deflection results. These methods by-pass the need for any mechanistic analysis, as carried out, as with the AASHTO NDT 1 Method. Regression methods are:

- Jameson (1993), using bowl deflections at 0, 900 and 1500 mm offset
- Roberts (1999), using deflections at 0 and 900 mm
- Rohde (in prep.), using deflections at 0 mm and approximately 3 other offsets, depending on the total thickness of the pavement.

The results of applying these methods to a wide range of New Zealand pavements are shown in Figure 6. The data dispersion is similar to that found by Rohde & Hartman (1996).

Individual data points are shown but it should be appreciated that less scatter would be apparent if results were averaged within identified treatment lengths.

- Jameson’s method gives good agreement for stiff pavements (SNP over 3.5)
- Roberts’ method gives good agreement in the mid range (SNP 1.5 to 3.5)
- Rohde’s method does not show good agreement but this may be due to variations between interpolated or inferred as-built pavement thicknesses, or the different pavement profiles in Rohde’s study area compared those of the present study. Where pavements have unknown thickness this method would be of limited applicability.

All the above relationships apply to “typical” pavement configurations in the locality in which they were derived. Therefore calibration to local conditions should be able to provide improved predictions for SNP.

FWD – SNP Regression Relationship for New Zealand Pavements

Data points from a wide range of New Zealand unbound granular pavements were used to determine a local correlation. The parameters used for study were restricted to those identified by Jameson and Roberts. Many trial functions were used (logarithmic, inverse and variable power). It was found that reasonably close relationships ($r^2>0.9$) could be generated by linear regression of the selected functions. Because regression on central deflection alone produced a best fit with an exponent of $-0.5$, this value was adopted for each term. The equation determined (with $r^2=0.94$) was:

$$
SNP = 112 \left( D_0 \right)^{-0.5} + 47 \left( D_0 - D_{900} \right)^{-0.5} - 56 \left( D_0 - D_{1500} \right)^{-0.5} - 0.4
$$

(13)
where $D_0$, $D_{900}$ & $D_{1500}$ are the deflections in microns at offsets of 0, 900 and 1500 mm respectively under a standardised 40 kN FWD impact load.

SNP is calculated inclusively of subgrade component, rather than independently as proposed by Jameson and Roberts. A selection of local pavements was compared using this function and results shown in Figure 7.

**Figure 7:** Deflection – SNC for NZ Unbound Pavements from Regression Analyses

\[
SNC = 112 (D_0)^{-0.5} + 47 (D_0 - D_{900})^{-0.5} - 56 (D_0 - D_{1500})^{-0.5} -
\]
FWD – AASHTO NTD 1 Method for Unbound Granular Pavements

The AASHRO NDT 1 method (or Method A as given by Rohde & Hartman, 1996) is generally intended for use with good as-built information. In New Zealand with many roads of unknown structure, the method has frequently been used with only occasional test pit data and/or inferred layer thicknesses. In the latter case, the surfacing (chipseal or asphalt) must be identified. While pavement temperature measurements are being taken during FWD testing the asphalt thickness can also be measured. Using only the AC thickness and minimal information on the granular layers, the procedure is to carry out a series of back-analyses of the FWD deflection bowl with trial layer thicknesses to develop a rational model for the type of pavement inferred from the surfacing. The method relies on the fact that the subgrade modulus is determined explicitly from the bowl (ie, its value is largely insensitive to the assumed layer thicknesses). Layer thicknesses do, however, influence the moduli for the surface layer moderately and the intermediate layer(s) substantially. The thicknesses are adjusted during the back-analysis so:

- The surface layer modulus is within the recognised range for the material type
- The intermediate layers show a progressive geometric gradation between the surface layer modulus and the subgrade modulus.

The results are generally reliable in unbound granular and structural asphaltic surfacings but are not entirely fool-proof. This is because in some cases a pavement may have developed (or been constructed with) an intermediate layer of lower strength than the subgrade. This form of inversion is rare, in practice. Another case is an “upside-down” pavement with cement stabilised subbase. These can often be identified by the analyst and modelled accordingly.

When a rational elastic model is developed for the pavement type, calculation for SNP continues in the normal manner using Equations 1, 2 and 3.

CALIBRATION FOR STRUCTURAL CAPACITY

The Need for Calibration

When basic SNP values result in predicted performance models which are inconsistent with known or expected performance (eg from long-term monitoring of representative benchmark sites), some investigation is required. Checks need to be made to see whether the assigned SNP gives due regard to the specific site conditions and pavement fatigue mechanisms. If not, calibration or correction of SNP values may be warranted.

Cases most commonly encountered during previous surveys for dTIMS purposes include:

- Changed physical conditions, ie conditions at the time of FWD survey are not appropriate to provide an indicator of structural capacity over a long term.
- Subgrades that exhibit superior (or inferior) performance than would be expected on the basis of the Austroads (1992) subgrade strain criterion.
- Pavements with thin chipseal surfacings and poor quality unbound granular basecourses which exhibit rapidly progressing distress through shallow shear (or shoving) in the upper layer, or in subbase layers.
- Pavements with abnormal structural profiles.

Any of the above can produce inappropriate SNP values leading to performance predictions of residual life which may be in error by an order of magnitude, unless an appropriate correction is applied. To signify some form of correction has been applied, the term SNP' is adopted below.
Changed Physical Conditions

The principal changes between the effective long term SNP and the value calculated from FWD measurement at a previous time, are those due to climate, construction and drainage factors. The appropriate adjustment factors for SNP are discussed in detail by Kerani (1996), Martin (1998) and Roberts et al (1997).

Calibration of Subgrade Strain Criterion

The Austroads (1992) subgrade strain criterion is intended to represent performance of all subgrade types. In practice this works relatively well, but there are some soil types that show much better performance than predicted by the Austroads model for conventional soils. In some cases future performance can be addressed simply by appropriate examination of past performance. If, for an unbound granular pavement, the principal mode of distress is rutting and the past traffic (ESA since constructed or last rehabilitated) can be estimated, then a precedent analysis may be carried out. The method uses the past rate of deterioration to predict the life under future design loading (TNZ, 1989; Tonkin & Taylor 1996; Arnold, 1998). The assumption is made that subgrade strains do not change markedly over the life of the pavement.

The procedure is to back-analyse deflection bowls to determine the subgrade strains for a pavement which has reached the end of its design life. The number of strain repetitions sustained is estimated from the pavement age and historic traffic data. By plotting these parameters onto the diagram showing recognised strain criteria (Ullidtz, 1987; Moffat & Jameson, 1998), the actual strain susceptibility of a specific subgrade may be compared with that expected for “conventional” soils.

If the conditions at the time of testing can be shown to be typical of those occurring historically and the serviceability of the pavement has not been significantly affected by routine maintenance, then a “local precedent” design criteria can be established, as shown in Figure 8. The ordinate for the design relationship can be assessed by graphically selecting an appropriate lower bound, (depending on the proportion of the road exhibiting a terminal condition), and the gradient should be parallel to the recognised strain criterion.

The strain criterion gradient can be shown to be related to the power law for traffic loading equivalence (Ullidtz, 1987). The latter has normally been regarded as approximately a 4th power relationship (derived from the AASHO Road Test). However Austroads has produced a 7th power relationship, as the result of an indirect back analysis of anisotropic CBR-pavement thickness design curves, yet it uses differing power laws for traffic loading equivalence. For this reason, the 4th power law may perhaps be regarded as having a somewhat more consistent and substantive origin. Also, because it leads to more conservative design, it
may be preferable when deriving a local precedent strain criterion. Further explanation is
given in Appendix I.

In practice, to apply a correction to SNP, the parameter $K_{SSR}$ may be defined as the ratio
of the allowable strain in the specific subgrade to the allowable strain in a “conventional”
soil subject to the same design ESA. (Usually the standard would be the current
Austroads subgrade strain criterion.)

Analyses of the past performance of several pavements founded on unweathered
volcanic ashes indicate that subgrade strains 1.5 to 1.75 times higher than that used for
standard soils can often be justified, ie $1.5 < K_{SSR} < 1.75$. It appears that unweathered
volcanic ash provides unusually high resistance to permanent strain accumulation,
probably attributable to the very high shear resistance provided by its sharply angular
grains.

Calculation of the corrected SNP' may therefore use the relationship:

$$E'_{\text{subgrade}} = K_{SSR} \times E_{\text{subgrade}} \quad (14)$$

followed by the standard procedure given by Equations 1, 2 & 3.

**Shallow Shear Correction**

Many unbound granular pavements which have been in service for many years exhibit
distress from shallow shear (lateral shoving in the basecourse layer). As soon as shallow
shear begins, a terminal condition will be achieved very rapidly due to the formation of
cracks within the developing depression which allows ingress of ponding water. This will
occur almost independently of the subbase quality and support (CBR) of the subgrade, if
the basecourse has suffered degradation. Vertical strains in degraded basecourse may
be greater than those in the subgrade. Similarly if a poor quality subbase is used beneath
a thin good quality basecourse, the greatest strains may be in the subbase. Performance
is likely to be dictated by the layer with the greatest strain. It therefore appears essential
to establish a procedure to correct for this form of distress.

Analysis of deflection bowls allows vertical strains to be assessed in all pavement layers.
In a well-designed granular pavement the highest vertical strains under traffic loading
usually occur at the top of the subgrade. An interim simplistic measure suggested for
correction of SNP is to define the effective subgrade as the layer with the greatest vertical
strains, and ignore all deeper layers, ie in practice, Equation 1 becomes:

$$SNP' = (1/25.4) \sum_{j} a_{j} h_{j} + SNSG_{j} \quad (15)$$

Where layer $j$ is defined as the layer with the greatest vertical strains when the pavement
is subjected to a 1 ESA loading.

**Unusual Pavement Structures**

Examples of “unusual” pavement structures are (a) a granular pavement over only 100
mm of soft subgrade overlying hard rock, or (b) one in which both the subgrade and a
marginal quality subbase experience almost the same strain. When compared to
pavements on the same subgrades and designed according to Austroads (1992, Figure
8.4), the former would have extended life while the latter would have lesser life. The
simplicity obtained by representing structural capacity as a single number, has its
drawbacks for unusual forms of pavement and this may be the reason that some
practitioners well acquainted with the fundamentals of pavement performance would
prefer a more rigorous approach. Ullidtz (1987) anticipates a move away from layer
coefficients and structural number towards “an analytical-empirical (or mechanistic empirical) procedure so that the stresses and strains may be calculated using the elastic parameters, and the performance (including the structural distress) may then be determined from damage functions”.

As an interim measure, because the structural number concept is well established, a means of calibrating SNP using a damage function, is being trialled for unbound granular pavements with anomalous structures. The procedure involves comparing the damage (after standard ESA loading) experienced by a given pavement structure with the damage that would occur in an equivalent standard pavement profile. The steps are:

1. Back-analyse the deflection bowl to determine moduli
2. Sub-layer the pavement into thin finite layers
3. Apply a 1ESA loading and calculate the damage in each sub-layer (weighting strains to a standard power law, ie 7th power for Austroads, 1992, Eqn 5.1 or 4th for Shell (1978)).
4. Integrate the total damage over the full depth of the zone of influence of 1 ESA (to say, 2 m)
5. Adopt a “standard” pavement profile and strain criterion eg Austroads (1992, Fig 8.4 and Eqn 5.1) and standard moduli for a basecourse layer (Figure 3)
6. Using the subgrade modulus, and subgrade strain calculated for the study pavement find the equivalent standard pavement, i.e that which experiences the same damage under 1 ESA (using steps 2-4 above applied to the standard pavement structure)
7. Assign SNP’ as the calculated SNP for the equivalent pavement.

The “Equivalent Pavement” method, gives SNP’ identical to SNP for pavements with moduli and structure that are the same as in the standard pavement. However, it may give greater or lesser values for other structures on the basis of the likely damage induced by a given traffic loading. The procedure is general, in that it allows the standard pavement, achievable moduli and equations for calculating damage, to be nominated for any given locality.

In practice, where selected benchmark sites are set up and monitored closely, the Equivalent Pavement method (or others as above that adequately account for the pavement fatigue mechanism) can then be used to calibrate the network to known performance. This basis provides an effective framework for long term pavement performance prediction, which takes due account of the local range of materials and construction techniques.

CONCLUSIONS

• When determining SNP from Benkelman Beam deflections, Paterson’s widely used relationship (Equation 4) is consistent with New Zealand data for deflections up to 2 mm. Equation 12 is the best fit to the full range of New Zealand data for unbound basecourses. However the SNP predicted from beam readings has low accuracy (Paterson indicates a standard error of about 1.2 on SNP).

• Where SNP is determined from FWD results, various methods have been proposed to avoid the need for a standard back-analysis of the bowl. These methods rely on linear regression, treating the deflections at various offsets as independent variables. It is evident that calibration is required for each locality. An appropriate regression relationship for New Zealand unbound granular pavements is given in Equation 13, producing a standard error of about 0.3 on SNP. Regression methods produce useful checks on the AASHTO NDT 1 Method.
The AASHTO NDT 1 Method for determining SNP requires standard back-analysis of the FWD deflection bowl, a procedure that is essential for an informed understanding of pavement performance and likely distress mechanisms. Existing software makes analysis a routine exercise. Therefore the AASHTO NDT 1 Method rather than regression equations should be preferred where FWD field data are available.

The standard equations used for the AASHTO NDT 1 Method are based on isotropic moduli for each layer, rather than anisotropic moduli which the Austroads Pavement Design Guide has adopted for mechanistic analysis. Typical field measured moduli for unbound granular pavements are presented and appropriate calculation procedures suggested.

Calibration or correction of SNP will often be required when refining structural capacity for reliably predicting long-term pavement performance. Various methods using mechanistic analysis of deflection bowls have now been tested and applied, giving more meaningful parameters. Where representative benchmark sites are set up and actual performance is monitored, appropriate calibration factors, based on specific modes of distress can be developed. These result in effective SNP values for a specific network, giving due regard to the local range of materials and construction techniques.

ACKNOWLEDGEMENTS

The authors acknowledge the encouragement of Dr Chris Bennett in the compilation of this review.

REFERENCES


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APPENDIX I – COMPARISON OF ALTERNATIVE SUBGRADE STRAIN CRITERIA

The Austroads subgrade strain criterion appears much less conservative for high traffic loadings than the strain criteria advocated by all other organisations. (See dashed lines on Figure 9). This article investigates the reasons for the differences, and implications for the practitioner wishing to compare experiences or utilise innovations from elsewhere in the world, through mechanistic analysis.

The straight lines on Figure 9 compare various published subgrade strain criteria. The more conservative relationships tend to reflect either greater reliability or more conservative terminal conditions (Ullidtz, 1987; Transit New Zealand, 1989). The Austroads derived values are distinguished as dashed lines (Austroads, 1992; Moffat & Jameson, 1998). When the Austroads subgrade strain criterion was established through back-calculation from the empirical pavement design chart (Fig. 8.4 of Austroads, 1992) significant assumptions were made:

All unbound granular layers and the subgrade were assumed to be anisotropic materials with the vertical modulus equal to twice the vertical modulus (E_v = 2E_h).
1. E_v was taken as 10 CBR and E_h as 5 CBR.
2. Poisson’s ratio was taken as 0.35 for the granular layers, and 0.45 for the subgrade.
3. Maximum anisotropic moduli for the upper granular layer were assigned (Austroads, 1992).
4. All materials were assumed to be linear elastic, with specified sub-layering.

The linear elastic model CIRCLY originally used for deriving the Austroads criterion has one feature (viz. anisotropy) which is not generally available in other packages, and also lacks one feature (viz. true non-linear modelling of the subgrade). In view of these differences it is useful to carry out the derivation (back-analysing Austroads, 1992, Figure 8.4) using the features of other software packages that are widely used internationally.

Anisotropy cannot be readily measured with any system routinely used by practitioners, and some have recently queried its use (Rallings, 1998; Rodway, 1998). Complications it introduces are four additional unknown parameters (modulus components transverse to and along the wheelpath, and Poisson’s ratios in these directions). It is also “uncertain whether this would bring you closer to or further away from the actual stresses and strains in the pavement” – Per Ullidtz, personal communication. Adoption of anisotropy places the Austroads procedures out of step with the rest of the world. By making the assumption that E_v (MPa) is 10CBR and in the 2 horizontal directions that E_h is 5CBR, the implicit (perhaps inadvertent) assumption made in the derivation of the Austroads anisotropic strain criterion is that E = 6.7CBR in isotropic terms. The latter value is
significantly different from $E=10\text{CBR}$ used internationally. Further explanation is given by Tonkin & Taylor, 1998.

Non-linear subgrade moduli are readily back-calculated from the FWD test and provide the practitioner with information on soil type and a more realistic model. “Many subgrade materials are highly non-linear, and if this is neglected very large errors may result in evaluation of the moduli of the pavement materials.....It should be noted that in a non-linear material the modulus increases with distance from the load, both in the vertical and in the horizontal direction. If one of the linear elastic programs is used to calculate the pavement response then the vertical increase in modulus may be approximated by subdividing the layer into a number of layers with increasing modulus, or by introducing a stiff layer at some depth. But this will not imitate the horizontal increase in modulus, and the deflection profiles derived will be quite different from those found on a non-linear material” Ullidtz (1987). For these reasons, use of a program which correctly models subgrade non-linearity is preferable even though the facility for inclusion of anisotropy is not provided.

For this study, back-analyses of a small number of pavement profiles were carried out using the relevant design charts (Austroads, 1992 – Figure 8.4 and Austroads 1998 – Figures 13.8.2A & B). Only isotropic materials were modelled with Poisson’s ratio of 0.35 and moduli for the upper unbound granular layer taken as the median FWD back-calculated values given in Figure 2. Median rather than lower bound results were used, as the latter would generate an unconservative strain criterion. The traditional isotropic relationship $E=10\text{CBR}$ was adopted, with linear moduli. The results are shown in Figure 9 (labeled Austroads $E_v = E_h$). The departure from a straight line is caused by the change in charts used at a loading of 100,000 ESA, ie between the light traffic design charts (Austroads, 1998) and the main guide (Austroads, 1992). If non-linear subgrade moduli are used with typical coefficients, (ie subgrade moduli increasing with decreasing stress) the result is to generate a strain curve which is concave downward in Figure 9. This concept is consistent with the isotropic criteria back-calculated in this study. Note that only strain has been considered here but an equivalent isotropic stress criterion (with practical advantages – Ullidtz, 1987) could be developed in the same way.)
Implications of the isotropic back-calculated strain criterion:

1. There is little difference between many of the recognised strain criteria for loadings of about 100,000 ESA.
2. The Austroads (1992,1998) anisotropic strain criteria are more optimistic at high traffic loadings than all of the other relationships located. This effect is partly apparent, due to the difference in assumptions made. However after appropriate adjustment for assumptions, the Austroads design chart (Austroads, 1992; Figure 8.4) still appears the most optimistic for traffic loadings greater than about 1 million ESA.
3. Most other strain criteria have gradients which correspond to about a 4$^{th}$ power law for traffic load equivalence. The Austroads anisotropic strain criterion implies a 7$^{th}$ power law while a variable power law is produced by the Austroads isotropic strain criterion.
4. The Austroads design charts for light traffic (Austroads, 1998) appear very conservative in relation to other commonly used strain criteria. This conclusion remains irrespective of which confidence level is used (Austroads, 1998).
5. It is important that forward analyses using the Austroads (1992,1998) anisotropic subgrade strain criterion use the same assumptions used in its derivation.
6. Because evaluation with the Falling Weight Deflectometer is being used increasingly by practitioners who seek to understand distress mechanisms and quantify pavement behaviour, there is a need for a recognised isotropic strain (or stress) criterion for use with the FWD. The isotropic criterion should be fully consistent with parameters measured on in-service pavements which have experienced adequate traffic compaction. The criterion used for forward analysis needs to be derived from back-analysed in situ (field) moduli.
7. The fundamental behaviour of unbound granular pavements is most readily appreciated in terms of applied stresses and strains. Their measurement in situ (using the FWD) leads to refinement of strain criteria for design. In the interests of sharing experiences with new forms of pavement design carried out elsewhere, universal procedures for mechanistic analysis should be targeted. Superfluous parameters (such as anisotropy), which are immeasurable with commonly available equipment, should be avoided unless there is clear benefit to practitioners. It is suggested that for FWD evaluations at least, the use of isotropic moduli and an isotropic strain (or stress) criterion should generally be adopted for mechanistic analysis, design and determination of structural capacity (SNP).
## ABBREVIATIONS

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>AC</td>
<td>Asphalt concrete</td>
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<tr>
<td>CBR</td>
<td>California Bearing Ratio</td>
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<td>dTIMS</td>
<td>Deighton Total Infrastructure Management System</td>
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<tr>
<td>FWD</td>
<td>Falling Weight Deflectometer</td>
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<tr>
<td>HDM</td>
<td>Highway Design and Maintenance Standards</td>
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<tr>
<td>NDT1</td>
<td>Non Destructive Test (Method 1)</td>
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<tr>
<td>SNC</td>
<td>Modified Structural Number</td>
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<tr>
<td>SNP</td>
<td>Adjusted Structural Number</td>
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<tr>
<td>SNSG</td>
<td>The modified structural number contribution of the subgrade</td>
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