
PAVEMENT PERFORMANCE PREDICTION

Determination and Calibration of Structural Capacity (SNP)

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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), hence 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 that the standard equations 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 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 local range of 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 Adjusted Structural Number (SNP) is widely used for this purpose. There have been numerous studies to find simple methods for calculating these parameter using either destructive or non-destructive tests. However, most studies relate to thick structural asphaltic pavements. Therefore, a review of completed projects has been carried out to determine the validity of alternative published procedures when these are adopted 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. Therefore there are numerous cases where calibration is essential before meaningful long-term pavement performance prediction can be obtained. From studies carried out over the last decade, several specific pavement types have been identified where calibration is essential, and for these, 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 HDM model.

$$SNC = (1/25.4) \sum_{i=1}^{n_{\text{layer}}} a_i h_i + SNSG \quad (1)$$

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:

$$SNSG = 3.5 \log_{10} CBR - 0.85(\log_{10} CBR)^2 - 1.43 \quad (2)$$

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:

$$CBR = E_s/10 \quad (3)$$

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 wheelload) gave an approximate relationship between central deflection and SNC, of the form:

$$SNC = 3.2 D_B^{-0.63} \quad (4)$$

if the base is unbound, or

$$SNC = 2.2 D_B^{-0.63} \quad (5)$$

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.

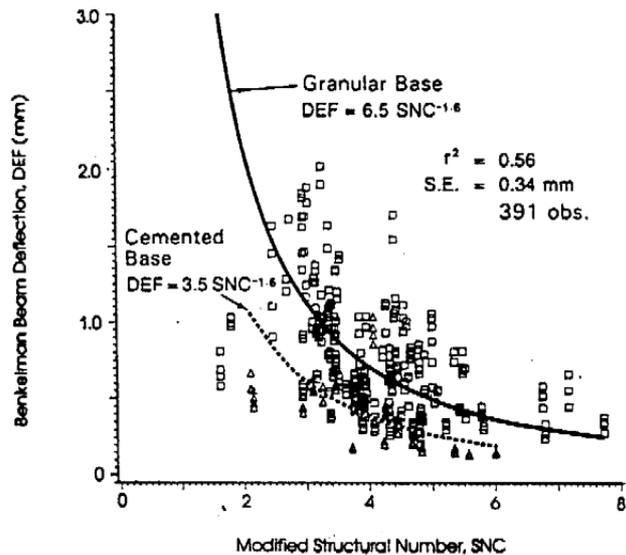


Figure 1: Benkelman Beam - SNC Relationships from Paterson (1987).

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 that the pavement strength for deep pavements is not over predicted. (Parkman and Rolt, 1997). However adjustments of a similar nature have commonly been used for at least the last decade when calculating SNC, hence the terms are used synonymously in this paper.

After Paterson’s study, many relationships were developed using the Falling Weight Deflectometer (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 herein as the “AASHTO NDT 1” Method, this will be 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:

$$a_i = a_g (E_i/E_g)^{0.33} \quad (6)$$

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).

Layer Type	Layer Coefficient	Resilient Modulus E_g (MPa)
Asphaltic concrete	0.44	3100
Unbound basecourse	0.14	207
Subbase	0.11	104

Table 1 is based on *isotropic* moduli for all layers and Poisson’s ratio of 0.35. However, the Austroads Guide (1992) recommends that 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 will be discussed in the following section.

IMPLICATIONS OF MODULUS ANISOTROPY

The majority of software packages for back-analysis of FWD deflection bowls (Ullidtz & Coetzee, 1995) assumes isotropic moduli for all layers. The practical reason for this is that 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 subbases. 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 software) have been plotted against two approximate measures of “support”, ie SNP and D_0 - the standard central deflection (under 40 kN plate loading), **Figures 2 & 3**. Most back-analysed moduli lie between

200 and 1000 MPa, ie with variation by a factor of at least 2 about the mean, a result that is not perhaps surprising given the variations in particle size distribution that will occur from point to point in an assemblage of granular particles.

Suggested conservative approximations (near the lower bounds of the data as shown in Figures 2 and 3) that represent reasonable design values are:

$$E_{\text{isotropic}} = 220 (\text{SNC})^{0.57} \quad (7)$$

$$E_{\text{isotropic}} = 330 (D_0)^{-0.5} \quad (8)$$

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:

$$E_{v,n=2} = 290 (\text{SNC})^{0.57} \quad (9)$$

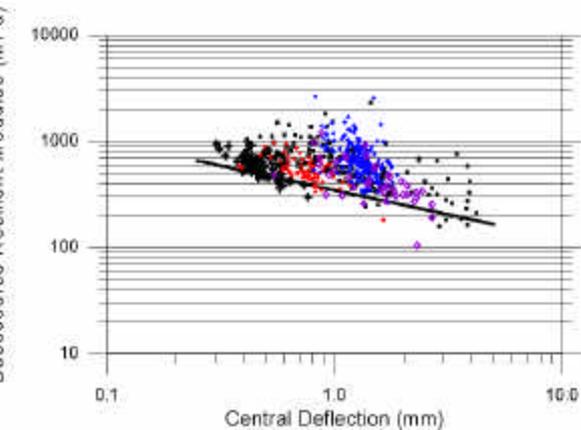


Figure 2: Design (Lower Bound) Basecourse Modulus vs FWD Central Deflection at 550 kPa

$$E_{v,n=2} = 440 (D_0)^{-0.5} \quad (10)$$

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**.

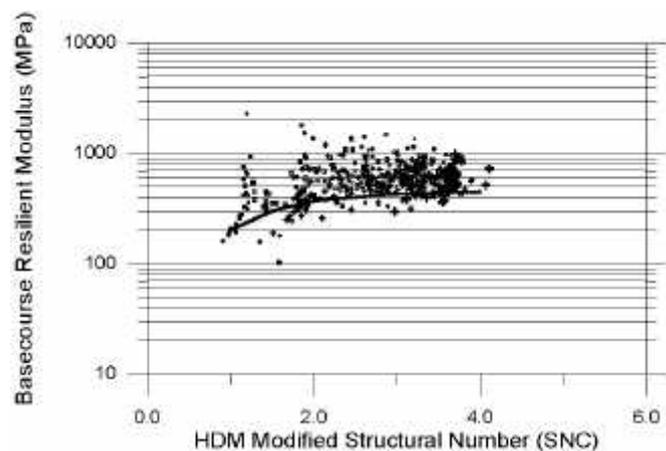


Figure 3: Design (Lower Bound) Basecourse Modulus vs SNC

The anisotropic results conform well with the wide ranges of vertical moduli suggested by Austroads (1992, Table 6.4a) but allow a more systematic approach by the designer. 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

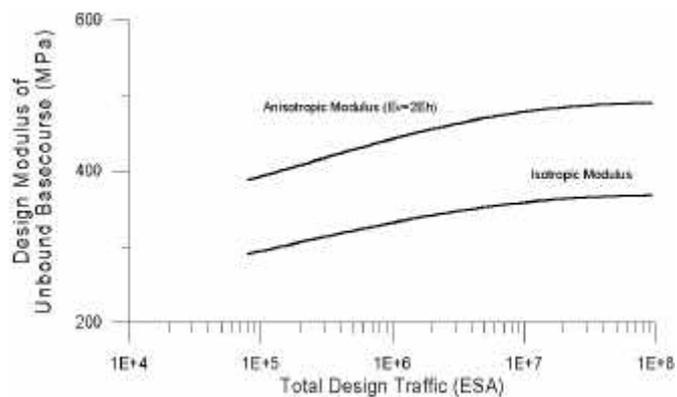


Figure 4: Basecourse Modulus vs Design ESA

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 the determination of 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 comparison of deflection data, 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 NZ 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, hence some as-built information will be essential to determine which of Equations 4 or 5 is appropriate.

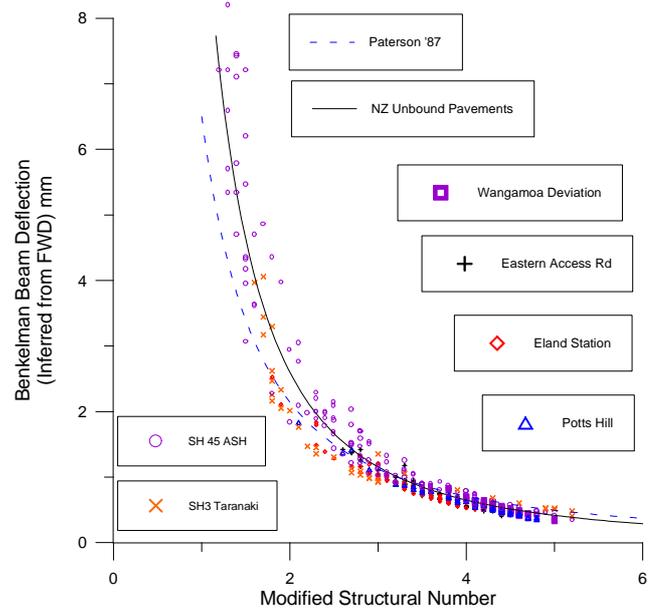


Figure 5: Beam -SNP Correlation for NZ Unbound Pavements (cf HDM III)
 $SNP=3.2 BB^{-0.5}$

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 dispersion of the data 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.

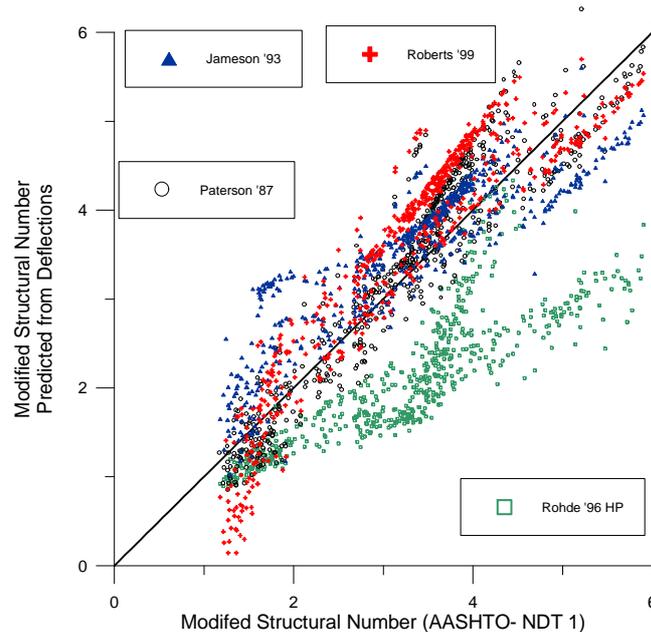


Figure 6: Deflection – SNC Correlation for NZ Unbound Granular Pavements

- 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 NZ 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:

$$\text{SNP} = 112 (D_0)^{-0.5} + 47 (D_0 - D_{900})^{-0.5} - 56 (D_0 - D_{1500})^{-0.5} - 0.4 \quad (13)$$

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**.

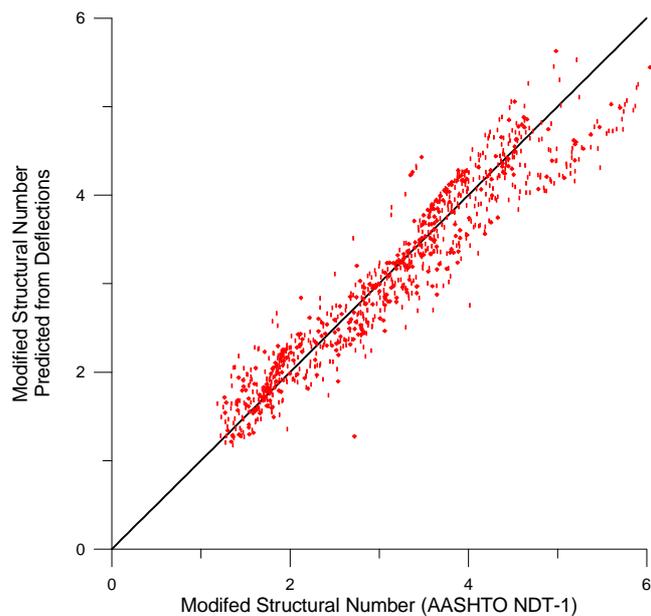


Figure 7: Deflection – SNC for NZ Unbound Pavements from Regression Analyses

$$\text{SNC} = 112 (D_0)^{-0.5} + 47 (D_0 - D_{900})^{-0.5} - 56 (D_0 - D_{1500})^{-0.5} - 0.4$$

FWD – AASHTO NDT 1 Method for Unbound Granular Pavements

The AASHTO 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 that the following conditions apply:

- (1) the surface layer modulus is within the recognised range for the material type
- (2) 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 foolproof 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 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:

1. 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.
2. Subgrades that exhibit superior (or inferior) performance than would be expected on the basis of the Austroads (1992) subgrade strain criterion.
3. 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.
4. 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 that some form of correction has been applied, the term SNP' will be 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 Kerali (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 which 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, may be preferable when deriving a local precedent strain criterion. Further explanation is given in Appendix I.

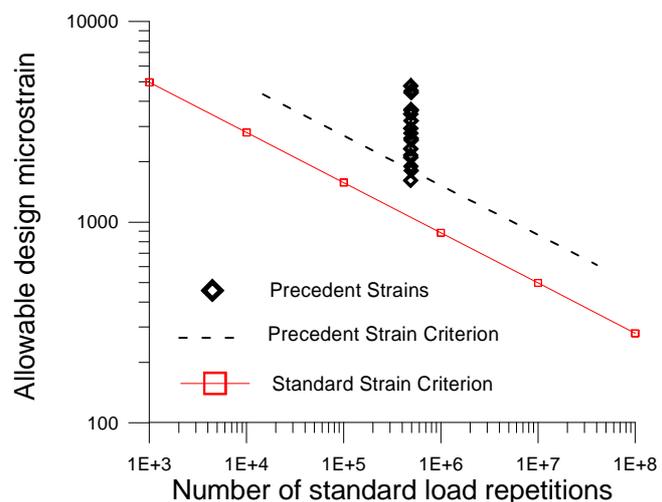


Figure 8: Determining a local "precedent strain criterion" from past performance

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:

$$SNP' = (1/25.4) \sum_{i=1}^{j-1} a_i h_i + SNSG_j \quad (15)$$

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, ie 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 NZ data for deflections up to 2 mm. Equation 12 is the best fit to the full range of NZ 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 Falling Weight Deflectometer 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 NZ 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 anyway 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 the reliable prediction of 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.

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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 NZ, 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:

1. All unbound granular layers and the subgrade were assumed to be *anisotropic* materials with

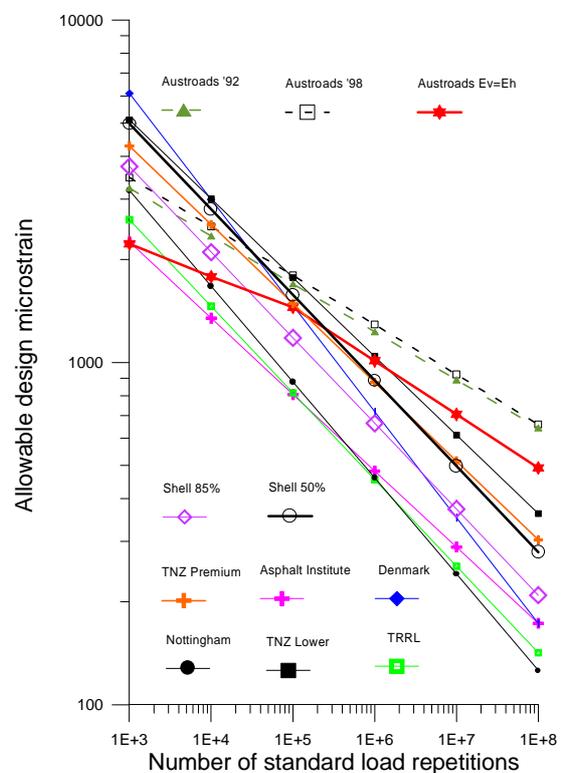


Figure 9: Comparison of alternative subgrade strain criteria

- the vertical modulus equal to twice the vertical modulus ($E_v = 2E_h$).
2. E_v was taken as 10 CBR and E_h as 5 CBR.
 3. Poisson's ratio was taken as 0.35 for the granular layers, and 0.45 for the subgrade.
 4. Maximum *anisotropic* moduli for the upper granular layer were assigned (Austroads, 1992).
 5. 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, pers comm. 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 $E = 6.7\text{CBR}$ 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 4th power law for traffic load equivalence. The Austroads *anisotropic* strain criterion implies a 7th 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).