Methods for Determining Structural Number of New Zealand Pavements

Transfund New Zealand Research Report No. 199

Methods for Determining Structural Number of New Zealand Pavements

J.E. Patrick, D. Dongal
Opus Central Laboratories
Lower Hutt, New Zealand

Transfund New Zealand Research Report No. 199

ISBN 0-478-25057-6
ISSN 1174-0574

Patrick, J.E., Dongal, D. 2001. Methods for determining Structural Number of New Zealand pavements. *Transfund New Zealand Research Report No. 199.* 37 pp.

Keywords: Benkelman Beam, CBR, Californian Bearing Ratio, FWD, Falling Weight Deflectometer, modulus, New Zealand, pavements, roads, strength, structural number

An Important Note For The Reader

The research detailed in this report was commissioned by Transfund New Zealand.

Transfund New Zealand is a Crown entity established under the Transit New Zealand Act 1989. Its principal objective is to allocate resources to achieve a safe and efficient roading system. Each year, Transfund New Zealand invests a portion of its funds on research that contributes to its objective.

Contents

Executive Summary

Introduction

The concept of describing the strength of a pavement in terms of one number, called the Structural Number (SN), was developed from the AASHO road test published in 1962. Since then the Structural Number has become a method for describing the strength of a road pavement, especially in the HDM pavement deterioration models that are currently being calibrated for New Zealand road conditions.

As part of this research, and for the dTIMS software being developed for predictive modelling software for road management in New Zealand, an investigation was carried out in 1999-2000 into the sensitivity and precision of the various methods of obtaining the Structural Number.

The investigation also looked at the differences that can occur in determining the modified SN (SNC, or SNP for thin pavements) by some of the methods, and at the spatial variability of strength, and therefore SNC, of typical pavements.

Definition of Structural Number

The original definition of SN has been modified to ensure that it is used appropriately in pavement design models. It now is as follows:

$$SNC = \Sigma_i^n \, a_I \, h_I + SN_{sg}$$

where

SNC = modified structural number
a_I = layer coefficient of layer i
h_I = layer thickness
n = number of layers above the subgrade
SN_{sg} = structural number contribution from the subgrade

The SNC is adjusted (Adjusted SN or SNP) to allow for the contribution of lower layers and the subgrade in pavements thicker than 700 mm. As most New Zealand pavements are thinner than this, SNP is used in the report as the defining term.

The equation states that the SNP of a pavement is calculated by determining a strength coefficient for each layer and multiplying it by the layer depth. The summation of these layers plus a contribution from the subgrade results in a total SNP for the entire depth of the pavement.

Methods of Calculating SNP

Although direct methods give high levels of precision, they are costly to perform, so cheaper indirect methods have been developed. The two kinds of methods were compared in this investigation to determine their relationships, and to compare their accuracy.

Direct Methods

The two different approaches used to directly measure the SNP are:

1. CBR (Californian Bearing Ratio) method, based on CBR of each pavement layer, their thicknesses, and of the subgrade.

2. Modulus method, based on strength coefficients from the modulus of each pavement layer, their thicknesses, and of the subgrade. This method is normally performed from a full back-calculation of an FWD (Falling Weight Deflectometer) test using known layer thicknesses.

Each of the direct approaches can result in a different assessment of the layer coefficient and SN_{sg}. They require details of the layer depths, and thicknesses.

Indirect Methods

These methods generally are based on relationships between deflection measurements and direct methods (either Modulus or CBR methods). The deflection measurements are obtained by:

1. Falling Weight Deflectometer (FWD), or
2. Benkelman Beam.

The indirect methods do not require the details of the pavement structure such as layer depths. Their precision depends on the accuracy of the initial data, and on calculating the appropriate potential errors.

Comparison of Methods

The investigation has highlighted that, although relationships between CBR and modulus have been proposed, they are not robust and they can result in large differences in the determination of the SNP, especially on volcanic subgrades.

Spatial Variability in SNP

As the SNP is used to assign a value to a road section of pavement, the expected spatial variability was investigated. Results showed that significant changes may occur spatially in SNP, both longitudinally and across wheel paths. Thus SNP data must be analysed for spatial trends, rather than assign a mean SNP to an entire pavement section.

Number of Tests Required to Characterise SNP

Standard deviations of SNP can be used to determine the number of tests required to obtain the mean SNP that is within a required confidence level. Thus the level of precision required will determine the number of tests to be carried out on a pavement.

To obtain an accuracy of ±0.3 which is required for long-term monitoring sites, approximately 20 tests would need to be performed. As the cost of this number of test pits and CBR tests would be prohibitive, the FWD could be used instead. In a network survey, an accuracy of ±0.5 for the mean SNP is required.

SNP of Volcanic Subgrades

Chipseal pavements built on volcanic subgrades in New Zealand exhibit low modulus and high deflections, but perform well. Their excellent performance has been attributed to the high shear strength (CBR) of these soils. This means that the subgrade is more resistant to rutting than the high deflection results suggest.

Recommendations

The investigation has highlighted the difficulty in determining a "true" SNP for a pavement section, and any method used should be relatively easy and inexpensive to carry out yet give consistent results with minimal bias.

- *Direct Methods*

The following three methods in order of decreasing precision are recommended.

The basic method: should consist of the following procedure:

1. Perform an FWD survey with at least 20 points along the pavement section.
2. Calculate the SNP using Tonkin & Taylor's indirect method.
3. Check the results for homogeneity.
4. Pick two points that have strengths near each end of the range of SNP (but not the outer extreme values).
5. At these points dig a test pit, record layer thickness and condition, and perform a CBR test on the subgrade).
6. Use the layer thickness data to perform a full back-calculation of the layer modulus from the FWD bowl shapes.
7. Use the CBR data to estimate the CBR–Subgrade Modulus relationship for that point.
8. Use the above data to re-calculate the SNP for all 20 test points.
9. Calculate the mean and standard deviation of the SNP for the entire road section.

This procedure uses the speed of the FWD to obtain the site variability, and by performing an in-situ shear test of the subgrade, it allows a better estimate of the contribution of the subgrade.

The second method: can be used if a robust CBR–Scala relationship for the subgrade type has already been determined. The basic method is used but the CBR test is replaced with Scala-derived values.

The third method: is the least precise and uses a default CBR–Scala relationship,

- *Indirect methods*

To estimate SNP using the FWD, the "Tonkin & Taylor method" is recommended because it has been derived using typical New Zealand pavements. This method is also recommended for network surveys, and for the determination of the homogeneity of a length of pavement before a full analysis is performed.

- *Pavements with volcanic subgrades*

Volcanic subgrades need to be treated with caution, and the above methods are not always appropriate. Therefore a second SNP should be derived using the back-calculated modulus of the subgrade. This SNP may be a better predictor of cracking and, until the calibration of the HDM model is complete, SNP assessments of volcanic subgrades should be determined by both CBR–modulus and direct FWD methods.

Research is urgently required to derive an indirect method and to adapt the FWD method for determining SNP for these materials.

Abstract

The Structural Number (SN) of a road pavement is a method for describing the strength of a road pavement, in pavement deterioration models that are currently being calibrated for New Zealand road conditions. To assist in the development of these models in New Zealand, and in implementing them in pavement design software, an investigation was carried out between 1999-2000, into the sensitivity and precision of the methods of obtaining the Structural Number.

The methods used are either direct, by CBR or modulus measurements of each layer in a pavement, or indirect, generally based on deflections of the entire pavement. Correlating direct against indirect methods, and the limitations of the correlations, are discussed. Spatial variability of SN on typical pavements in New Zealand, the number of tests required to characterise SNP to different levels of precision, and predicting SNP for subgrades from volcanic materials were investigated. Recommendations are given for preferred methods to be used in New Zealand for routine road network surveys, and for long-term pavement monitoring studies.

1. Introduction

Pavement strength is one variable used to predict pavement performance over time, and thus the rate of pavement deterioration. The concept of describing the strength of a pavement in terms of one number, called the Structural Number (SN), was developed from the AASHO (1962)[1] road test, and Rohde & Hartman (1996) have described its development. Since then the Structural Number has become a method for describing the strength of a road pavement, especially in the HDM[2] pavement deterioration models that are currently being calibrated for New Zealand road conditions.

As part of this research, and for the dTIMS software being developed for predictive modelling software for road management in New Zealand, an investigation was carried out in 1999-2000, by Opus International Central Laboratories, Lower Hutt, into the sensitivity and precision of the various methods of obtaining the Structural Number. The methods are described fully in a report, *Implementation of dTIMS in New Zealand – Establishing Pavement Strength*, by HTC Infrastructure Management (2000).

Structural Numbers can be determined using direct or indirect methods. Identifying the relationships between these two kinds of methods, comparing them and the levels of precision that can be obtained, as well as understanding the limitations of the correlations, were investigated in the project reported here.

The investigation also looked at the differences that can occur in determining the modified SN (called SNC) by some of the methods, and at the spatial variability of strength, and therefore SNC, along and across typical pavements in New Zealand.

Recommendations are given on the preferred methods that should be used in New Zealand for routine road network and for long-term pavement monitoring studies.

---

1 AASHO – Association of American State Highway Organisations (before 1973),
AASHTO – Association of American State Highway & Transportation Organisations (after 1973).

2 HDM – see p.2 of this report.

2. Definition of Structural Number

While carrying out this investigation, it became apparent that a clear understanding of the basis for and definition of the Structural Number (SN) is required to ensure that the SN is applied appropriately in pavement deterioration modelling.

The Structural Number can be deduced by either direct or indirect methods.

- Direct methods use measurements of the strengths of each of the layers in a pavement.
- Indirect methods are generally based on deflections of the entire pavement. However, because they are based on correlations obtained from back-calculations against more direct methods, they have limitations that need to be understood.

The initial definition of SN was proposed when formulating the original AASHO road test published in 1962, and it took the following form:

$$SN = \sum_i^n a_i \cdot h_i \qquad (1)$$

where:
- SN = structural number
- a_i = layer coefficient of the ith layer
- h_i = thickness of the ith layer
- n = number of pavement layers above the subgrade

The above equation does not include a contribution to the SN from the subgrade, and thus it was modified in 1975 by TRRL[3] to derive the modified SN (i.e. SNC) which now includes the subgrade contribution, as in the following equation:

$$SNC = \sum_i^n a_i\ h_i + SN_{sg} \qquad (2)$$

where:
- SNC = Modified Structural Number
- SN_{sg} = contribution to Structural Number from subgrade

The development of HDM4[4] proposed another variation to the calculation to overcome the problem that, in pavements thicker than 700 mm, equation 2 tends to over-estimate the SNC. The so-called Adjusted Structural Number (SNP) applies a weighting factor to the pavement thickness, so that the contribution to the pavement strength from the lower sub-base layers and the subgrade is not over-predicted for pavements thicker than 700 mm. However, as most New Zealand pavements are less than 700 mm thick, SNC and SNP are essentially equivalent. Therefore in this report, SNP is used as the defining term for New Zealand pavements.

Equation 2 states that the SNC (i.e. SNP) of a pavement is calculated by determining a strength coefficient for each layer of the pavement and multiplying it by the layer depth. The summation of these layers plus a contribution from the subgrade gives a total SNC (i.e. SNP).

---

3 TRRL – Transport & Road Research Laboratory (before 1972),
TRL – Transport Research Laboratory (after 1972).

4 HDM-III – Highway Design & Maintenance Version III,
HDM4 – Highway Development & Management Version 4.

The method by which the strength coefficient for each layer is determined, and also how the subgrade contribution is assessed, will affect the outcome.

Initially the subgrade contribution was based on the CBR[5] test.

$$SN_{sg} = -0.85 (\log CBR)^2 + 3.51 (\log CBR) - 1.43 \quad (3)$$

where: SN_{sg} = contribution to structural number of the subgrade
CBR = in situ CBR of the subgrade

The strength coefficients for the different pavement layers were based on a visual assessment, but relationships with CBR were developed in 1965 as follows:

$$a_I = 0.00645\ CBR^3 - 0.1977\ CBR^2 + 29.14\ CBR \quad \text{(for basecourse)} \quad (4)$$

$$a_I = 0.01 + 0.0065 \log CBR \quad \text{(for sub-base)} \quad (5)$$

Since the initial development of the SN concept, methods have been developed to measure the strength coefficients other than using CBR. These have included the use of resilient modulus tests and the back-calculation of layer moduli from FWD[6] tests.

The use of the CBR test implies that the SNP is composed of layers having different shear strengths, while the use of moduli implies that SNP is composed of materials with different load-spreading characteristics.

Although general relationships between CBR and moduli have been proposed, these relationships do not apply on some volcanic subgrades especially. Such subgrades are of low modulus but have relatively high shear strength so that, even though they have high deflections, they still perform well.

In the back-calculation of layer properties from FWD measurements, the SN of the subgrade is nevertheless based on CBR. The relationship between modulus and used in the AASHTO Method 1 (1986) is:

$$Esg = 41.19\ CBR^{0.385} \quad (6)$$

When calculating the CBR from FWD, the above equation is re-arranged. This method appears to be commonly used by most roading practitioners.

In New Zealand, the experience with granular pavement design incorporating volcanic subgrades indicates that SNP should be based on the shear strength of the materials rather than on the modulus. However, where bound pavement layers are involved, SNP based on the CBR of the subgrade will under-estimate the strain on these layers, and therefore may not predict the onset of pavement cracking.

---

5 CBR – Californian Bearing Ratio.
6 FWD – Falling Weight Deflectometer.

3. Sources of Measurement Errors

The accuracy of the Structural Number (SNP) is related to the precision of the measurements that are used in its calculation. Where a number of measurements are made, each has a measure of uncertainty and the accumulation of these "errors" result in an overall total expected error.

For the investigation recorded in this report, the precision of the test or measurement was taken where possible from published information and is given in Table 3.1.

Table 3.1 Precision errors expected in methods used to obtain SN.

Parameters	Precision Error	Source
In situ CBR : Less than 10 10 to 30 30 to 60 Greater than 60	 ±1.5 ±2.5 ±5 ±12.5	ASTM D 4429 – 93 (1993)
Pavement thickness measurements	±10 mm	Assumed
Deflection measurements: – with FWD	 Greater of 2 microns, or 1.25% of the mean +0.5 microns	 FEHRL (1996)
– with Benkelman Beam	0.027 mm	Central Labs Report M2, 86/27 (unpublished report 1986)

The above random errors will affect the precision or the uncertainty in the calculation of the SNP, but in the direct CBR and Modulus methods described in Section 2 the effect will also depend on the pavement structure.

The values in Table 3.1 have been used in a sensitivity analysis given in Section 4 of this report.

4. Direct Methods of Calculating SNP

4.1 CBR Method

4.1.1 Sensitivity of CBR Method

In this method the CBR of individual pavement layers are measured directly or indirectly, and the thickness of each layer is also measured. In order to obtain an estimate of the overall precision error, an analysis was performed for three different pavement configurations, shown in Figures 4.1 – 4.3.

Figure 4.1 Pavement configuration - 1.

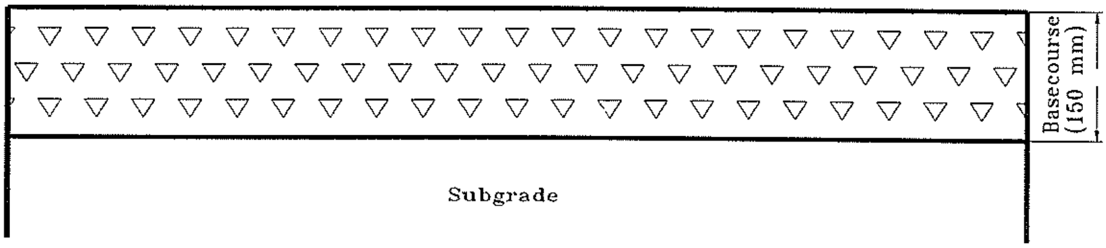


Figure 4.2 Pavement configuration - 2.

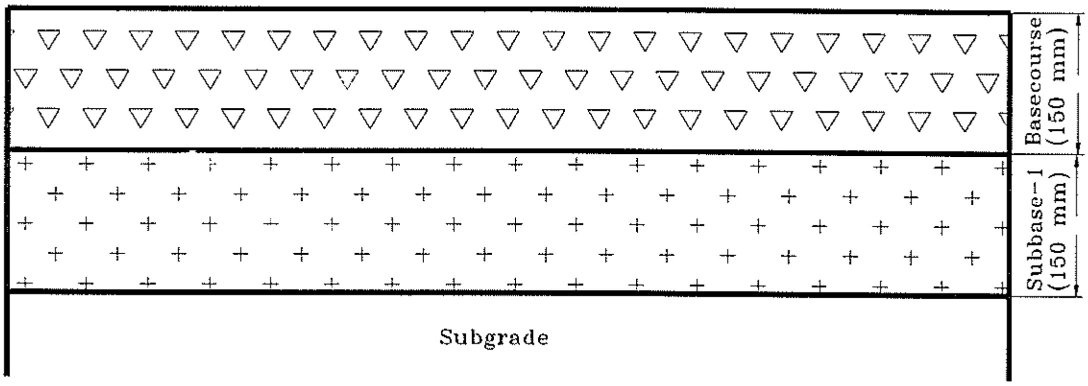


Figure 4.3 Pavement configuration - 3.

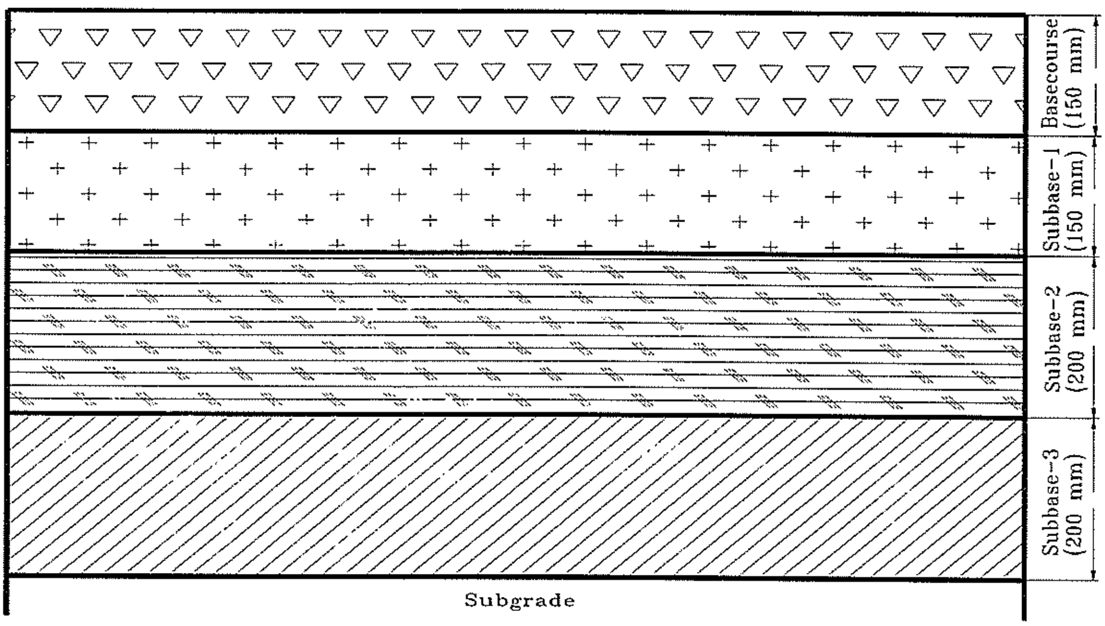


Figure 4.4 Effects of basecourse CBR for pavement configurations 1, 2, 3 on SN, with subgrade CBR of 5 and sub-base CBR of 30.

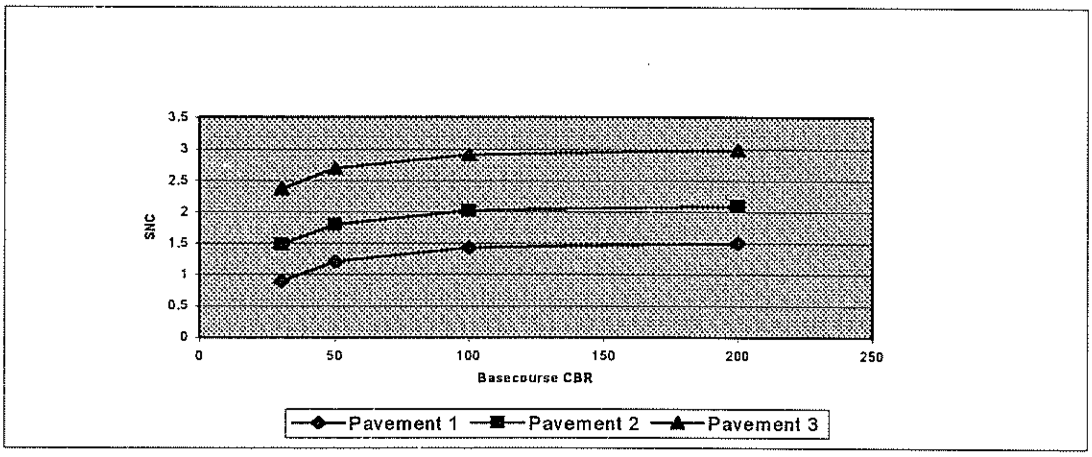


Figure 4.5 Effects of sub-base CBR for pavement configurations 2 and 3 on SN, with subgrade CBR of 5 and basecourse CBR of 100.

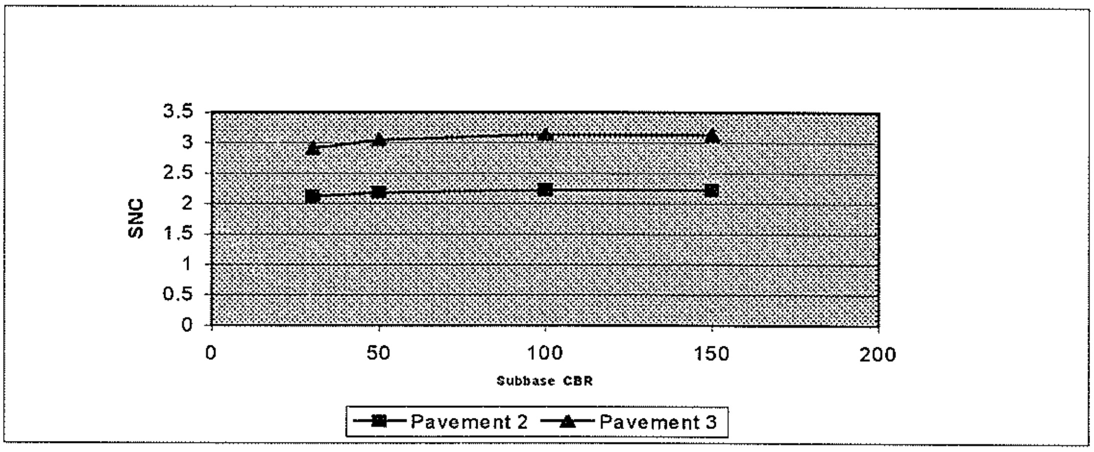
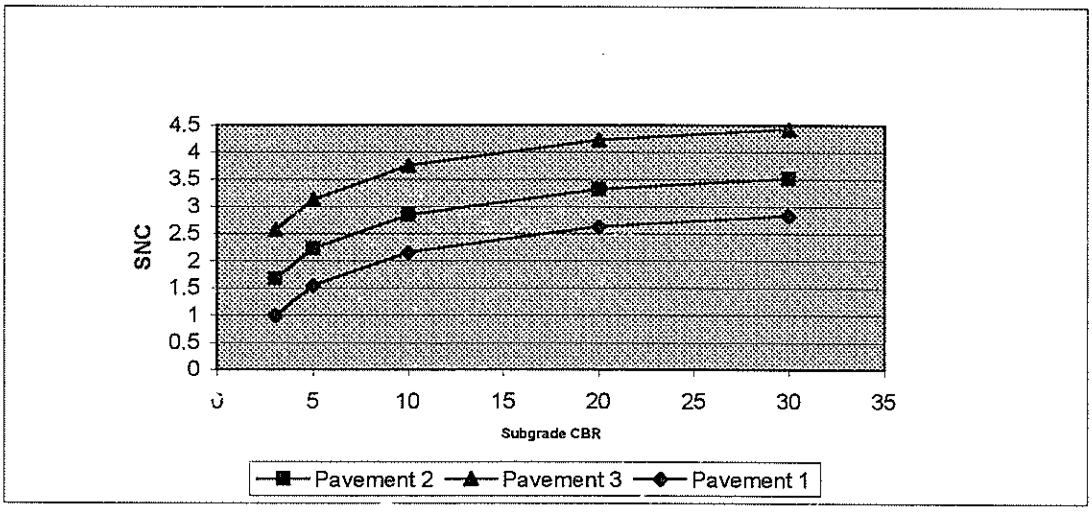


Figure 4.6 Effects of subgrade CBR for pavement configurations 2, 3, 1 on SN, with basecourse CBR of 100, and sub-base CBR of 100.



For each pavement configuration, various values of CBR were assumed and results from the error analysis are shown in Figures 4.4, 4.5 and 4.6. The figures show that errors in the determination of the sub-base and basecourse CBRs have minimal effect, especially if the CBR is above about 80 for a basecourse, or above 50 for a sub-base.

However the subgrade CBR has a more pronounced effect when the CBR is below 20. The effect is illustrated in Table 4.1, using the precision errors given in Table 3.1.

Table 4.1 Change in SNP related to precision of CBR tests on subgrades.

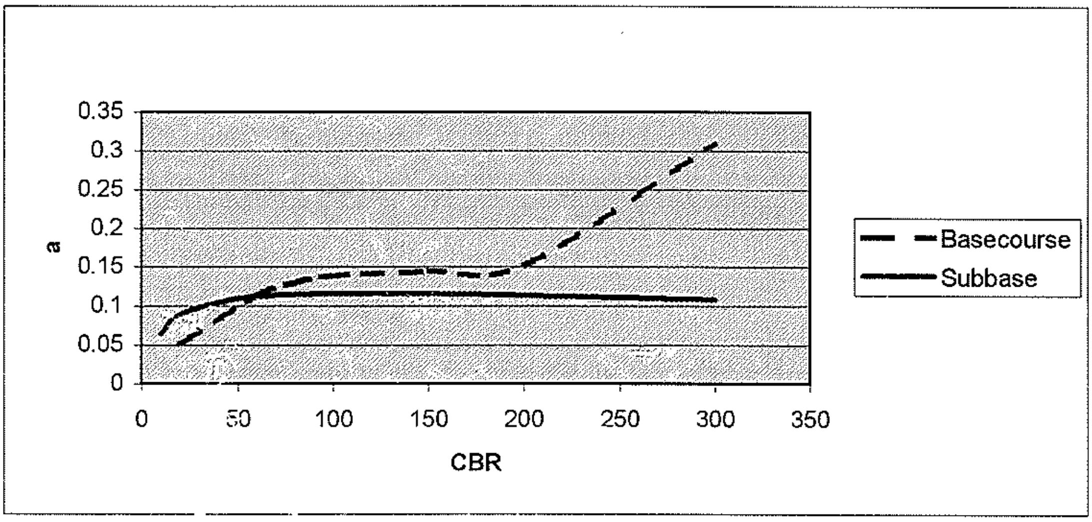
Subgrade CBR	Change in SNP
5	0.40
10	0.16
20	0.08

4.1.2 Strength–CBR Relationship for Granular Layers

In the calculation of SNP, the strength coefficient is multiplied by the layer thickness to obtain an SN for that layer. The strength coefficient can be determined from the CBR of the material, and from the relationships in equations 4 and 5, given by Watanatada et al. (1987) as recommended in the HDM4 (Rohde 1995; University of Birmingham 1998). Normally for New Zealand pavements, in-situ CBR tests will not be performed on each pavement layer. Rather, an assessment of the strength will be made that is based on a visual assessment of the material.

The effects of basecourse and sub-base CBRs on the strength coefficients are shown in Figure 4.7. The basecourse strength coefficient 'a_i' increases steadily with increasing CBR, and reaches almost a plateau value at about CBR = 200. As the CBR increases beyond 200, 'a_i' increases sharply. For practical purposes, an upper limit of 200 of basecourse CBR is recommended.

Figure 4.7 Effect of CBR on the strength coefficient a_i.



A similar effect with sub-base CBR also occurs. The curve shows a sharp increase in strength coefficient 'a_j' with CBR up to 60, and then starts to decrease very gradually. An upper limit of 200 to the sub-base CBR is recommended.

The CBR values are for the in-situ moisture condition (not soaked). Therefore the normal granular basecourse is expected to have a CBR value greater than 80, and only sub-base layers contaminated by a subgrade will have CBR values in the 30 to 50 range. Therefore under most conditions, an error in the estimation of the in-situ CBR of the base and sub-base layers will have a minimal effect, i.e. up to 0.1 SNP.

4.1.3 Strength–CBR Relationship for Subgrades

The basis of the determination of the contribution of the subgrade is the CBR test. This requires specialised equipment and is relatively expensive to perform. The simpler Dynamic Cone Penetrometer test (DCP or Scala test) is often performed instead. The relationship given in the AUSTROADS Pavement Design Manual (1992) is used to convert the values to CBR. Unpublished research performed by MWD Central Laboratories in 1986 has shown that the relationship between CBR and Scala test results is material-dependent and can vary by a factor of 2 or more. For a typical pavement, such as configuration 2 shown in Figure 4.2, the effect of a change in CBR from 5 to 10 on SNP is 0.7. On strong subgrades, a change from 20 to 40 CBR (a factor of 2) changes the SNP by 0.3.

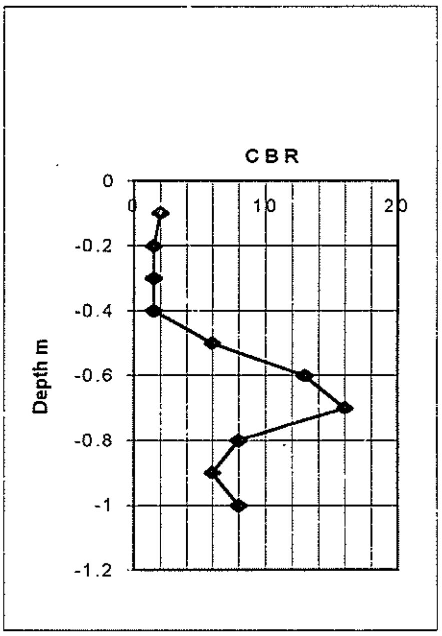
For typical New Zealand pavements, the determination of the subgrade CBR is the critical component in obtaining the SNP. In pavement rehabilitation investigations, a CBR tests and Scala tests are performed in parallel to obtain a site-specific relationship. The Scala penetrometer can then be used to sample the subgrade more extensively.

The use of the Scala penetrometer introduces a bias into the determination of the SNP. The relationship with CBR is very material-specific and the use of a universal relationship means that, for some soil types, the CBR will be over-estimated while for others it will be under-estimated.

A further complication with using the Scala technique is in determining the appropriate estimate of CBR to be used. The CBR test measures the strength at the top of the subgrade, but the Scala test can give an estimate of CBR properties to over a metre in depth, which may reach other layers. To illustrate the difficulty of recording CBR at depth, results from a real site are given below in Figure 4.8.

For the typical pavement configuration 2 (Figure 4.2), i.e. 150 mm basecourse, 150 mm sub-base, an SNP based on the top 400 mm of the subgrade, which has a CBR of 2, gives an SNP of 1.5. However, if the average for the 1.0 m depth is used (which has a CBR of 6.5), the SNP is 2.46. In this pavement configuration, the SNP has changed by 1 unit depending on the interpretation of the test results.

Figure 4.8 CBR inferred for a subgrade of 100 mm depth, using DCP or Scala test.



4.2 Modulus Method

Back-calculation of layer modulus from the FWD test and then assigning strength factors has become a common method to obtain an "accurate" measure of the SNP.

The method is similar to the CBR method in that, for each layer, a strength coefficient is obtained from the modulus of the layer. But the method also requires a relationship between the subgrade modulus and CBR because the contribution of the subgrade in this method is still based on the subgrade CBR. Therefore, where an FWD test is performed, a relationship between modulus of the subgrade and CBR is assumed. Emery (1985) has proposed one relationship (Equation 6) that has been incorporated in HDM4. For comparison, the AUSTROADS relationship (1992) (modified for isotropic conditions) as described by Tonkin & Taylor (1998) are compared in Figure 4.9.

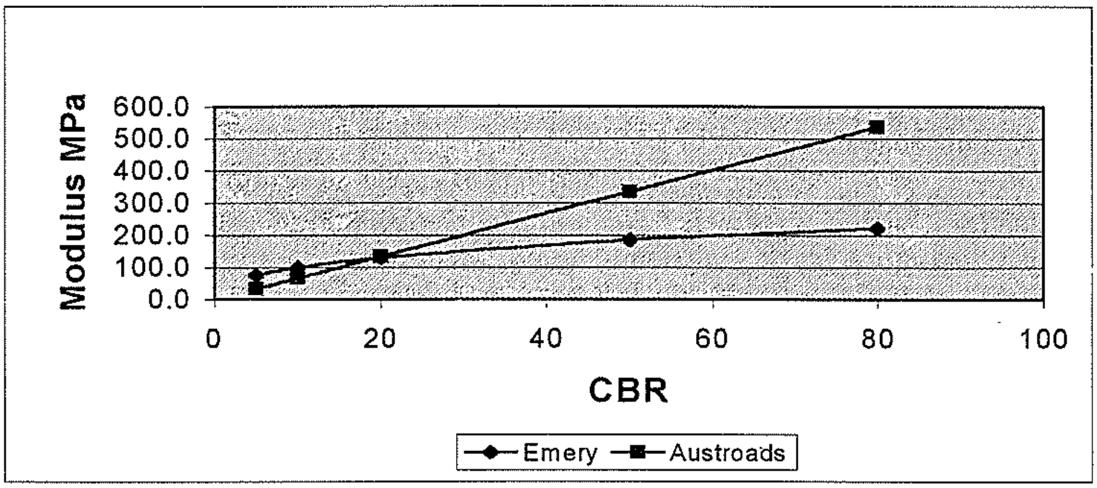


Figure 4.9 Comparison of relationships of subgrade modulus–CBR obtained by two direct methods (from Emery 1985, AUSTROADS 1992).

Correspondence between the direct CBR and FWD methods depends on the assumed relationship between CBR and Modulus. This relationship is not well defined and errors in the order of a factor of 2 can occur (Tonkin & Taylor 1998). This error is of the same order as was described by the CBR–Scala relationship and can result in differences of up to 0.7 in the calculated SNP.

The structural coefficient of the unbound pavement layers is based on the layer modulus. Relationships are given in the HTC report (2000) and Rohde & Hartman (1996). Figures 4.10 and 4.11 compare the shape of the CBR and moduli 'a_2' (basecourse) and 'a_3' (sub-base) relationships.

Figure 4.10 Comparison of three CBR–modulus relationships with strength coefficient a_2 (basecourse strength factor) (based on AASHTO 1986, CBR 10, Ullidtz 1987).

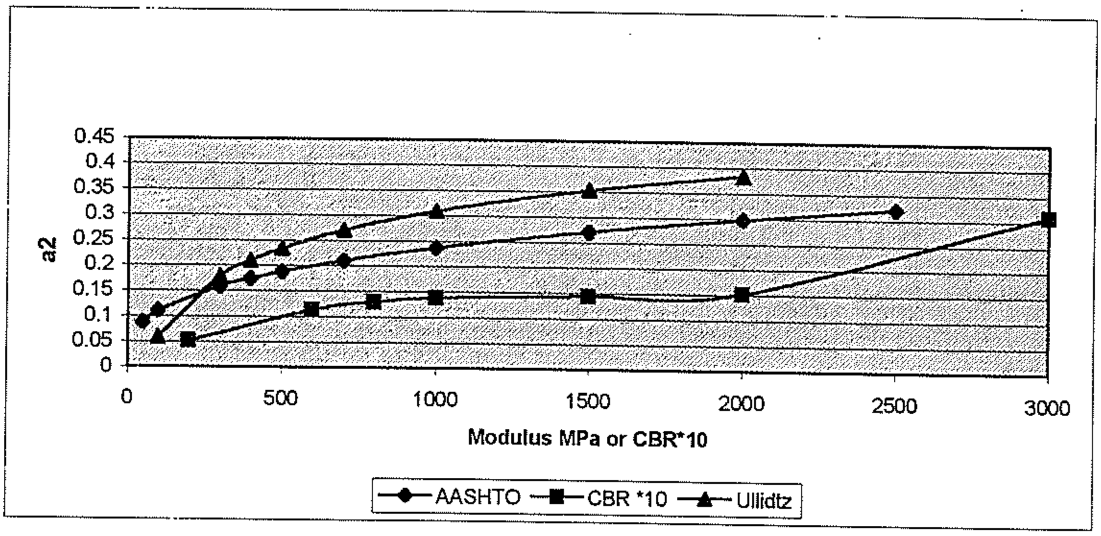
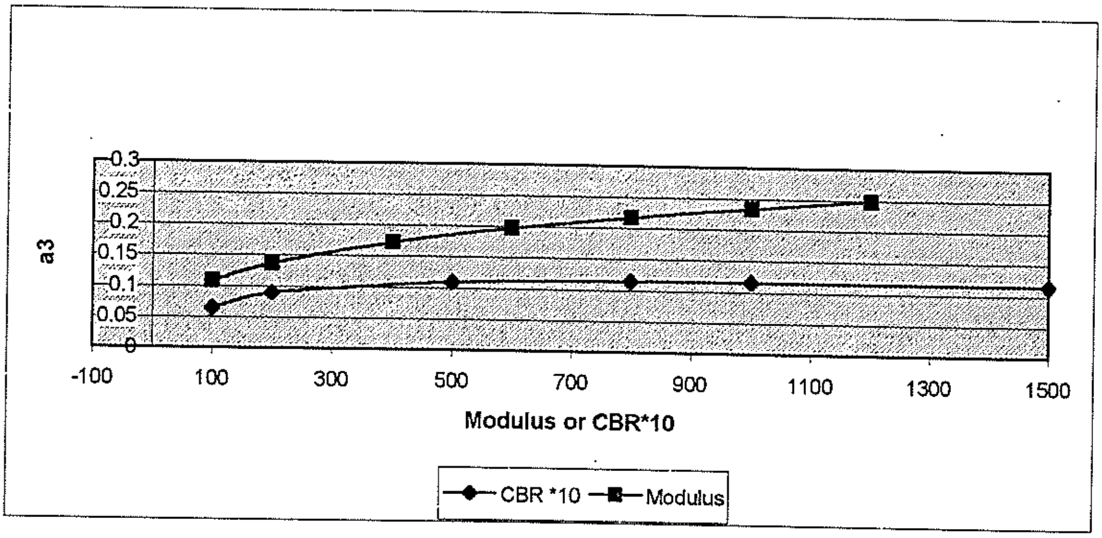


Figure 4.11 Comparison of CBR–modulus relationship with strength coefficient a_3 (sub-base strength factor) (based on CBR 10, AASHTO 1986).



The figures show that the modulus curves are continuous while the CBR curves reach a plateau. This means that, at higher modulus of 600-1000 MPa that are typical for New Zealand pavements, there will be a significant difference in the calculated SNP. The effect on a strong basecourse, as for the pavement 2 example (Figure 4.2) where a basecourse CBR of 200 was assumed, and if the modulus was 800 MPa, then a pavement on a subgrade CBR of 10 would be calculated to have an SNP of 2.78 using the CBR method. However, using the modulus method, an SNP of 3.17 would be obtained.

To obtain an SNP from the FWD test that is similar to that from the CBR method, the modulus of the basecourse would need to be 300 MPa.

There is, therefore, a fundamental difference in the estimation of the structural coefficients for the Modulus and CBR methods. At low moduli the difference is minimal but becomes significant at higher modulus values. A difference of 0.5 SNP can be expected where high basecourse moduli are obtained.

Beside the different forms of the strength coefficient equations, the back-calculation of modulus values is dependent on the skill and experience of the person performing the analysis, even when the layer thickness are known.

Many papers in the technical literature describe the effects that various interpretation techniques can have on the resulting layer modulus, and the effect of the analysis technique has not been further assessed in this study.

4.3 Differences between CBR & Modulus Methods

The combination of the error in the determination of subgrade CBR and in the different shape of the modulus 'a_2' relationship can compound to result in a difference of greater than 1 SNP unit between these two direct methods.

This difference is not a random error but it will vary depending on the subgrade type and basecourse strength. The difference between the two methods is not associated with the errors in measurement but with the relationships used in determining the strength factors.

5. Indirect Methods of Calculating SNP

Various indirect methods of calculating SNP have been derived by different researchers that generally are based on relationships between deflection measurements, obtained either by FWD or Benkelman Beam methods, and direct methods, either by modulus or CBR.

The precision of indirect methods is based on the accuracy of the measurement of the original inputs. So that the effect of the precision of the measurements on the calculated SNP can be assessed, the potential errors arising in each method can be calculated.

5.1 Analysis of Random Errors

Random errors are experimental or precision errors, which occur by chance. The extent of the error is estimated in terms of the standard deviation, Sy of n measurements of a variable y, which is given by:

$$S_y = \left(\sum_{i=1}^{n} \frac{(y_i - \bar{y})^2}{(n-1)} \right)^{\frac{1}{2}} \tag{7}$$

where: n is the sample size.

Inaccuracies inherent in the equipment or its calibration contribute to a systematic error. This type of error is not random and can only be controlled by stringent calibration and quality control of the instrumentation. It has not been included in the analysis.

The total error in y = f(x, z), related to uncertainty in the parameters x and z, can be generally described by:

$$\sigma_y^2 = \left(\sigma_x\right)^2 \left(\frac{\partial f(x, z_{const})}{\partial x}\right)^2 + \left(\sigma_z\right)^2 \left(\frac{\partial f(z, x_{const})}{\partial z}\right)^2 + higher\,terms \tag{8}$$

In equation 8, there is no cross correlation between x and z, and higher terms have insignificant effects on the overall error in y.

The error analysis was performed for each method and described in the subsequent sections of the report. The sources of error for each method were identified and quantified as given in Table 3.1.

5.2 Falling Weight Deflectometer Methods

Two of the FWD methods outlined in the HTC report (2000) are Jameson's (1993), and the Tonkin & Taylor method (Salt & Stevens 2001).

Jameson's (1993) relationship is:

$$SN = \left(b_0 + \frac{b_1}{(D_0 - D_{1500})} + \frac{b_2}{D_{900}} \right) \tag{9}$$

$$CBR_S = 10^{(b_3 - b_4 \log_{10}(D_{900} x 1000))} \tag{10}$$

where: b = parameter of layers 0, 1, 2, 3, 4
D = distance(mm) from point of load application
S = subgrade

The error in this method is associated with the accuracy of the deflection measurement at distances (mm) of D_0, D_{1500} and D_{900} from point of load application.

The Tonkin & Taylor relationship is:

$$SNC = \left(112 D_0^{-0.5} + 47(D_0 - D_{1500})^{-0.5} - 56(D_0 - D_{1500})^{-0.5} \right) \tag{11}$$

The error in this method is associated with the accuracy of the deflection measurements at distances D_0 and D_{1500}.

In terms of the standard deviation associated with the precision of the measurement system of an FWD associated with either relationship are given in Tables 5.1 and 5.2.

Table 5.1 Error (Standard Deviation) in SNP related to uncertainty in various parameters using Jameson's formula (1993) at 95% confidence level.

Measured D_0	Measured D_{900}	Measured D_{1500}	Calculated SNP	$2^* \sigma_{SNP}$
0.489	0.1397	0.091	5.4855	0.0857
1.089	0.2556	0.134	3.6032	0.0286
2.527	0.58033	0.308	2.1437	0.0115

Table 5.2 Error (Standard Deviation) in SNP related to uncertainty in various parameters using FWD (Tonkin & Taylor method), at 95% confidence level.

Measured D_0	Measured D_{900}	Measured D_{1500}	SNP	$2^* \sigma_{SNP}$
0.489	0.1397	0.091	4.3726	0.223
1.089	0.2556	0.134	2.8099	0.060
2.527	0.58033	0.308	1 7044	0.0170

The analysis showed that the errors in calculated SNP which are associated with the accuracy of measurement obtained from an FWD, is in the order of ± 0.2 for the Tonkin & Taylor method, and ± 0.1 for Jameson's method. This low value of error can be attributed to the high precision of the FWD measurement system.

5.3 Benkelman Beam Method

The estimation of SN from a Benkelman Beam reading is:

- for granular pavements
  $SNP = 3.2*DEF^{-0.63}$

- for cemented base pavements
  $SNP = 2.2*DEF^{-0.63}$

The source of error is therefore only in the measurement of the peak deflection.

The effects of this uncertainty on the SNP for various deflections are presented in Table 5.3. The table shows that the measurement precision at high deflections contributes approximately 0.02 to the SNP, but at low deflections it can affect the value by as much as 0.8 in granular material.

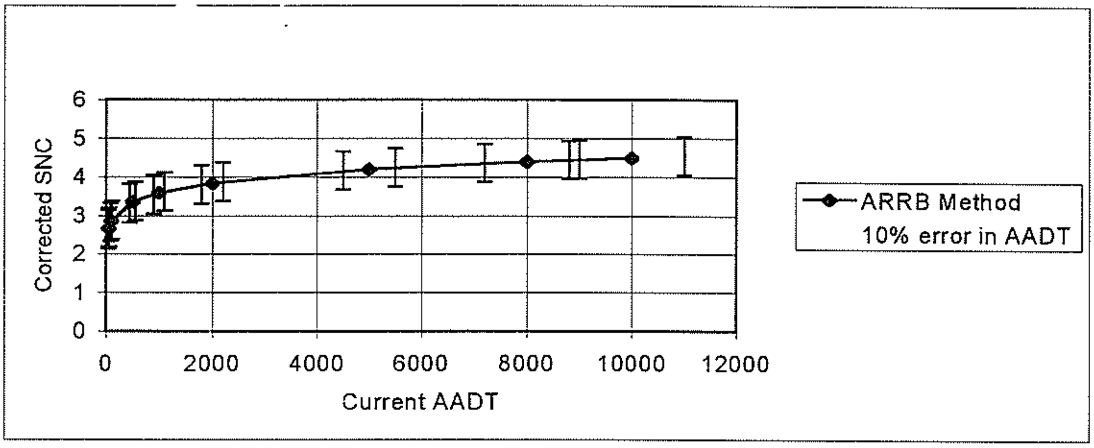
Table 5.3 Error (Standard Deviation) in SNP related to uncertainty in deflection measurements at 95% confidence level using the Benkelman Beam method.

Deflection Values		Granular Material		Bound Material	
Measured (mm)	Uncertainty (mm)	Calculated SNP	Error in SNP 2* σ_{SNP}	Calculated SNP	Error in SNP 2* σ_{SNP}
0.3	0.027	6.832	0.774	4.697	0.532
0.8	0.027	3.683	0.157	2.532	0.108
0.9	0.027	3.420	0.129	2.351	0.089
1.0	0.027	3.200	0.109	2.200	0.075
1.5	0.027	2.479	0.056	1.704	0.039
3.0	0.027	1.601	0.018	1.101	0.012

5.4 AARB Method

The AARB method (Roberts & Roper 1998) is based on the assumption that if a pavement has been designed to the AUSTROADS Pavement Design Guide (1992) that the SNP should be consistent for the design traffic volume. Using the current traffic volume and age of the pavement, its SNP can be calculated. An analysis based on a range of various values of AADT traffic volumes showed that a 10% uncertainty in the measurements has insignificant effects on overall values of SNP. The relationship is shown in Figure 5.1 where the vertical error bars are ±10% of the AADT.

Figure 5.1 ARRB method of evaluating SNP using a range of AADT traffic volumes.



6. Comparison of Methods

Data from 6 sites in the Hamilton region have been analysed. They include CBRs obtained from in-situ Scala penetrometer measurements of the subgrade, and a visual assessment of CBR of the other layers.

At the same sites, FWD tests using two methods were carried out as well. The results were obtained by using assumed thicknesses of the pavement layers, and by the Tonkin & Taylor method.

Table 6.1 compares the SNP obtained from:

a. FWD with assumed layer thicknesses;
b. FWD by Tonkin & Taylor method;
c. Using the subgrade CBR(Scala) with a visual assessment of the strength of the other layers;
d. Using the subgrade CBR (Scala) with an assumed basecourse CBR of 200, and a sub-base CBR of 100.

Comparing results of methods c and d (Table 6.1) shows that using assumed CBR values affect the SNP value by no more than 0.2 of a unit. Results of the FWD methods a and b (Table 6.1) show that SNP values can vary as much as 2 units.

Figure 6.1 compares the SNP calculated from CBR (Scala) (method c in Table 6.1), with the two FWD methods (a, b in Table 6.1). It shows that results from method b are more precise than those calculated with assumed layer thicknesses.

Besides the difference in the average SNP obtained from the FWD methods, some of the pavements exhibit a difference in the variances. This means that statistically the methods are measuring different properties.

Based on analysis using the F test, the SNP based on FWD (by the Tonkin & Taylor method) and on CBR, are different for the Airport (1) and Racecourse (6) pavement test sites at the 5% confidence level, and for the Huntly (4) test site at the 10% confidence level.

Figure 6.1 Comparison of SNP obtained from FWD methods with CBR(Scala) method.

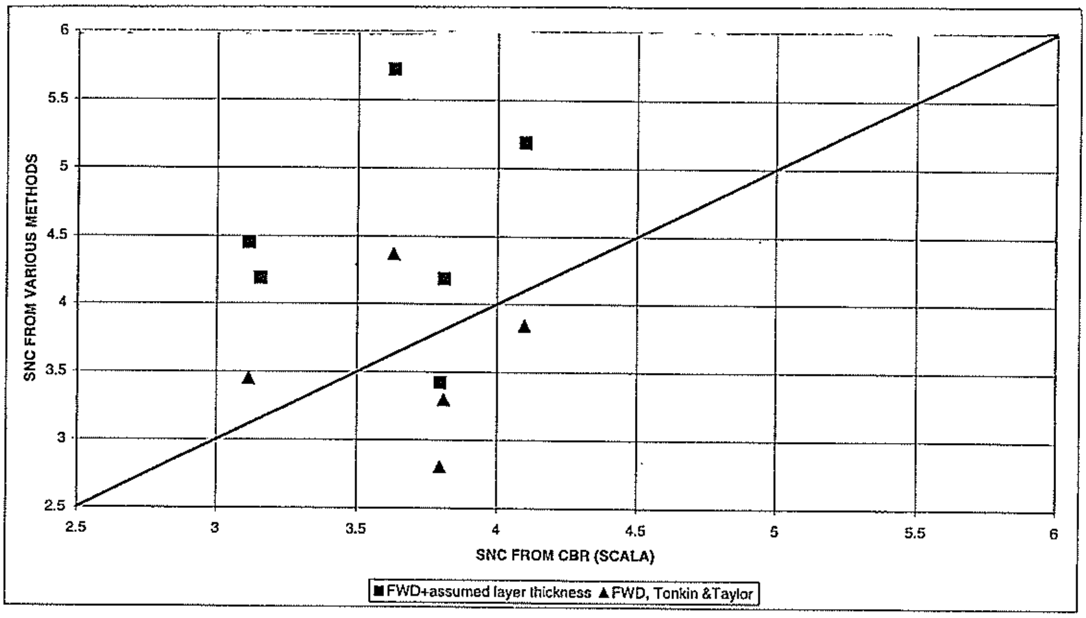


Table 6.1 Summary of pavement test site results.

1. AIRPORT ROAD

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	N
FWD, ASSUMED THICKNESS	4.450	0.732	0.536	8.712	2.644	168
FWD, T&T METHOD	3.448	0.715	0.511	7.295	1.236	168
SG CBR(SCALA). Visual Assess other layers	3.114	0.414	0.171	3.700	2.344	16
CBR, BASE=200, SUB-BASE=100	3.206	0.436	0.190	3.844	2.351	16

2. MARAETAI

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	*N*
FWD, ASSUMED THICKNESS	4.189	1.733	3.002	10.770	2.580	53
FWD, T&T METHOD	2.179	0.717	0.514	5.547	1.369	53
SG CBR(SCALA) Visual Assess other layers	3.154	0.544	0.296	3.814	2.520	5
CBR, BASE=200, SUB-BASE=100	3.212	0.556	0.309	3.906	2.579	5

3. KOPUKU

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	*N*
FWD, ASSUMED THICKNESS	5.187	0.783	0.613	7.460	3.536	79
FWD, T&T METHOD	3.840	0.622	0.387	5.837	2.229	79
SG CBR(SCALA) Visual Assess other layers	4.098	0.538	0.290	5.212	3.388	9
CBR, BASE=200, SUB-BASE=100	4.266	0.571	0.326	5.428	3.544	9

4. HUNTLY

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	*N*
FWD, ASSUMED THICKNESS	3.422	0.904	0.817	5.670	2.111	67
FWD, T&T METHOD	2.800	0.709	0.503	4.490	1.650	67
SG CBR(SCALA) Visual Assess other layers	3.797	0.413	0.171	4.447	3.207	8
CBR, BASE=200, SUB-BASE=100	3.904	0.429	0.184	4.597	3.278	8

5. KAKARIKI

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	*N*
FWD, ASSUMED THICKNESS	5.724	0.623	0.388	7.516	4.650	35
FWD, T&T METHOD	4.368	0.507	0.257	5.863	3.486	36
SG CBR(SCALA) Visual Assess other layers	3.629	0.488	0.238	1.529	2.869	7
CBR, BASE=200, SUB-BASE=100	3.741	0.518	0.268	4.718	2.959	7

6. RACECOURSE

Parameters	*MEAN*	*STD*	*VARIANCE*	*MAX*	*MIN*	*N*
FWD, ASSUMED THICKNESS	4.186	0.671	0.451	6.660	2.750	75
FWD, T&T METHOD	3.296	0.635	0.403	5.400	2.040	75
SG CBR(SCALA) Visual Assess other layers	3.809	0.285	0.081	4.159	3.275	10
CBR, BASE=200, SUB-BASE=100	3.903	0.295	0.087	4.314	3.348	10

T&T Tonkin & Taylor method
SG Subgrade

7. Spatial Variability in SNP

The variability in SNP for the pavement of a site can be random, or can be related to a change in pavement properties along the site. To determine a characteristic SNP for a section of a pavement, the results should be checked to ensure that the pavement properties show no significant changes. The following two examples, at the Huntly and Racecourse test sites for FWD surveys, illustrate spatial variations in properties.

Figures 7.1 and 7.2 show the spatial variation in SNP, for the two test sites. Figures 7.3 and 7.4 show the results where a moving average of 5 points has been used. The use of the moving average has made it obvious that, for the Huntly site from approximately 13.4 km, a distinct difference shows between the left and right wheel paths. A breakdown of the FWD results is given in Table 7.1, in which the difference in SNP between the two wheel paths is about 1.

Figure 7.1 Spatial variation (longitudinal and lateral) in SNP at Huntly pavement test site, using Tonkin & Taylor method.

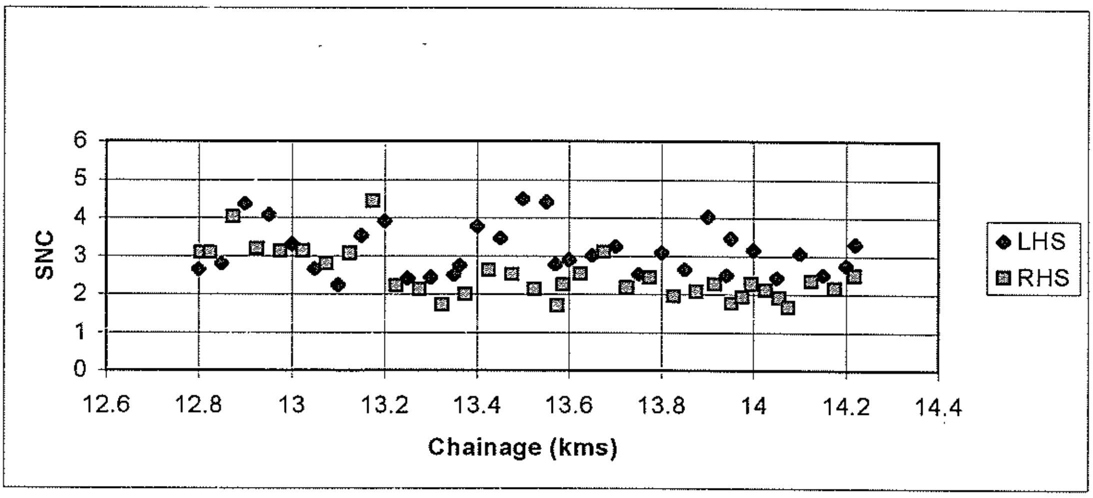


Figure 7.2 Spatial variation (longitudinal and lateral) in SNP at Racecourse pavement test site, using Tonkin & Taylor method.

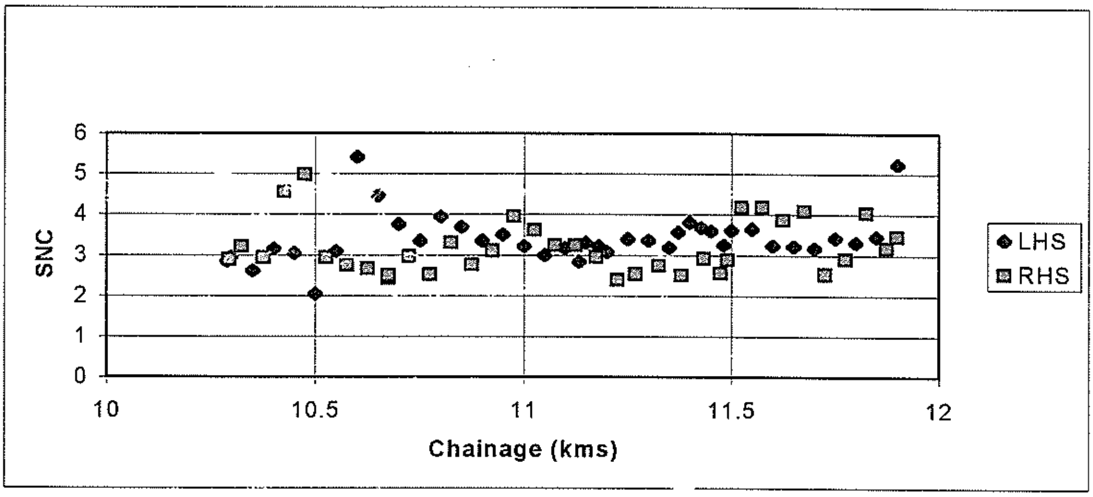


Figure 7.3 Moving average (of 5 points) for SNP at Huntly pavement test site, using Tonkin & Taylor method.

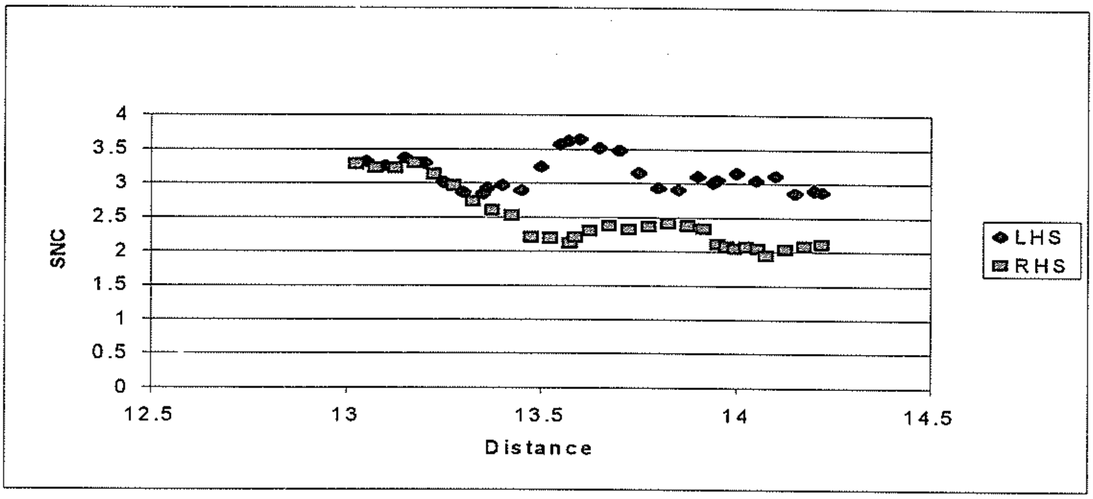


Figure 7.4 Moving average (of 5 points) for SNP at Racecourse pavement test site, using Tonkin & Taylor method.

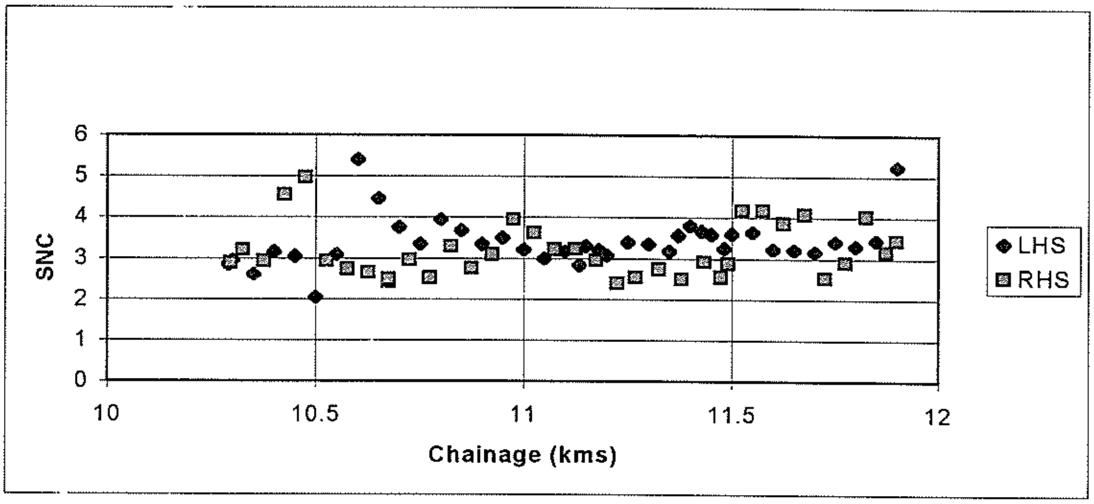


Table 7.1 Details of FWD results from Huntly test site.

	Mean	Standard Deviation
Overall	2.802	0.719
LHS from 13.4 km	3.183	0.616
RHS from 13.4 km	2.208	0.345

These results illustrate the importance of analysing SNP data for trends spatially, both longitudinally, and laterally across wheel paths, rather than assigning a mean SNC to an entire pavement section.

8. Number of Tests Required to Characterise SNP

The results from the 6 test sites around Hamilton (Table 6.1) suggest that the standard deviation of SNP is in the order of 0.4 to 0.7. These STD can be used to obtain the number of tests required to obtain the mean SNP within a required confidence interval. The level of precision required will determine the number of tests to be carried out to characterise the SNP of a pavement.

The standard deviation of the mean of a number (n) of measurements is:

$$S_{y_{mean}} = \left(\frac{S_y}{n^{1/2}} \right) \tag{12}$$

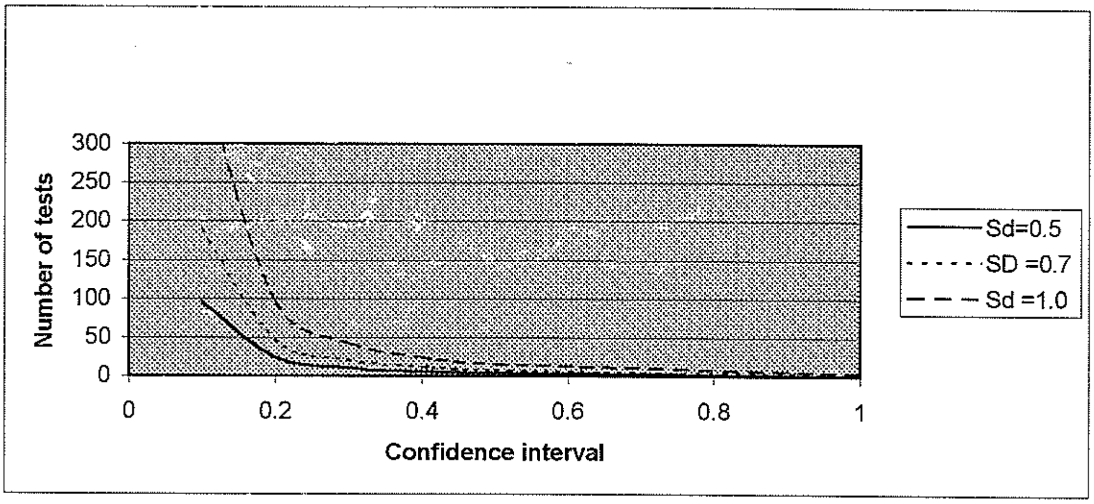
If the number of measurements is large, their deviation from the mean should approach a normal distribution, with the uncertainty of the mean being equal to $1.96Sy_{mean}$ at the 95% confidence level. It follows that, for the 95% confidence level, the range of the measured quantity is equal to $y_{mean} \pm 1.96Sy_{mean}$. Random errors can be minimised by increasing the number of measurements.

Table 8.1 Number of tests required to obtain mean SNP to different confidence intervals.

Confidence Interval	sd=0.5	sd=0.7	sd= 1
0.1	96	188	384
0.2	24	47	96
0.3	11	21	43
0.4	6	12	24
0.5	4	8	15
1	1	2	4

These results are shown graphically in Figure 8.1.

Figure 8.1 Number of tests required to obtain the mean SNP at different confidence intervals.



To obtain a level of confidence for long-term monitoring sites of the order of $\pm$ 0.3, over 20 tests would need to be performed. This number of tests tends to preclude the use of test pits and in-situ CBR tests, as the cost would be prohibitive. However the FWD could efficiently perform this number of tests.

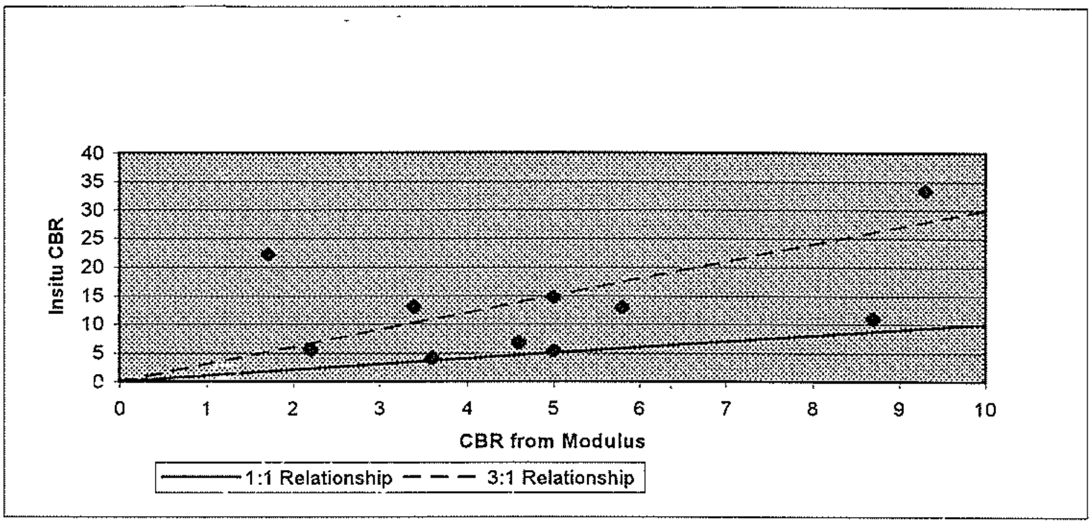
In a network survey where a test is being performed at every 100 metres, the mean SNP over a kilometre section would be within $\pm$ 0.5 of the mean SNP.

9. SNP of Volcanic Subgrades

Pavements built on volcanic subgrades in the North Island of New Zealand are known to exhibit high deflections but perform well. An investigation into the performance of pavements in the Wanganui area built on "brown ash" (Sutherland et al. 1997) showed a significant difference in the predicted performance depending on whether the assessment of the subgrade properties was based on in-situ CBR or back-calculated from Benkelman Beam deflections.

The volcanic subgrades appear to have higher shear strengths than those normally obtained with lower modulus materials. The performance of the subgrade is a function of the shear strength of the material rather than its modulus, and thus the CBR test has been found to be the better predictor of strength.

Figure 9.1 Comparison of CBR derived from modulus, and from in-situ CBR tests on Wanganui Brown Ash.



The relationship between the modulus-derived CBR and the in-situ CBR from the Wanganui project is shown in Figure 9.1. It shows considerable scatter and does not appear to show a direct correlation. Further research would be required to determine if there was a robust correlation that could be used on volcanic subgrades. However, as was discussed in Section 4.2, a factor of 2 can be expected in modulus–CBR conversion on normal soils and this factor is expected to be greater on volcanic materials.

For the Wanganui project, Benkelman Beam tests were also performed. On each of the five sites, two in-situ CBR tests and deflection tests were performed at the same point. The data have been used to calculate SNP based on the in-situ CBR, and on a visual assessment of the layer properties and compared with the Benkelman Beam-derived SNP (shown in Figure 9.2). The average ratio of CBR to Benkelman Beam SNP is 1.9 on this "brown ash" subgrade.

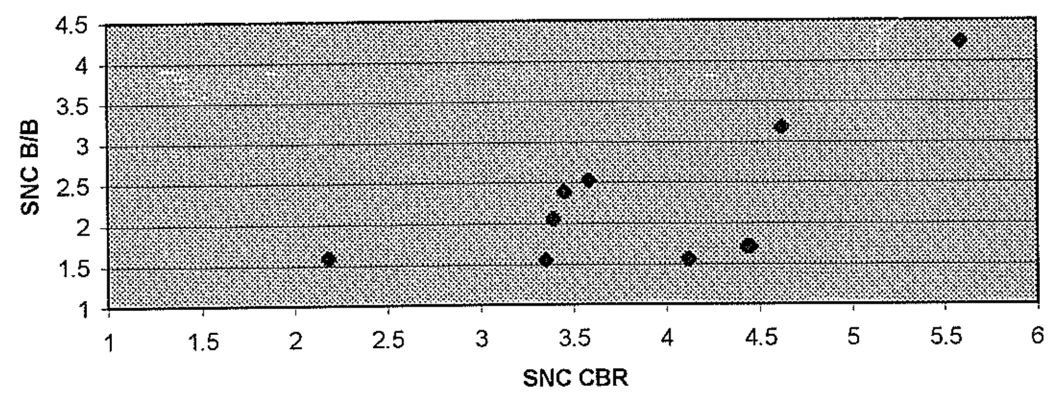


Figure 9.2 Comparison of SNP obtained from CBR and Benkelman Beam methods on Wanganui Brown Ash.

The SNP is used in deterioration modelling as a variable in rutting and cracking models. Where the deformation of the pavement is controlled by the subgrade properties, a CBR-derived SNP would appear to the appropriate input.

In the cracking models the pavement deflection would be expected to be a significant variable. A CBR-derived SNP would then tend to under-estimate the deflection, and an FWD-derived SNP may be a better predictor for cracking.

10. Recommendations

This investigation has highlighted the difficulty in determining a "true" SNP for a pavement section. New Zealand does not have a long history in the use of this concept, so it is important and timely that consensus in the roading industry is gained concerning the methods to be used for determining SNP.

Any procedure should be relatively easy and inexpensive to perform but nevertheless should give consistent results with minimal bias.

- *Direct Methods*

The following three direct methods in order of decreasing precision are recommended.

The basic method: should consist of the following procedure:

1. Perform an FWD survey with at least 20 points along the pavement section.
2. Calculate the SNP using Tonkin & Taylor's indirect method.
3. Check the results for homogeneity.
4. Pick two points that have strengths near each end of the range of SNP (but not the outer extreme values).
5. At these points dig a test pit, record layer thickness and condition, and perform a CBR test on the subgrade.
6. Use the layer thickness data to perform a full back-calculation of the layer modulus from the FWD bowl shapes.
7. Use the CBR data to estimate the CBR–Subgrade Modulus relationship for that point.
8. Use the above data to re-calculate the SNP for all 20 test points.
9. Calculate the mean and standard deviation of the SNP for the entire road section.

This procedure uses the speed of the FWD to obtain the site variability, and by performing an in-situ shear test of the subgrade, it allows a better estimate of the contribution of the subgrade.

The second method: can be used if a robust CBR–Scala relationship for the subgrade type has already been determined. The basic method is used but the CBR test is replaced with Scala-derived values.

The third method: is the least precise and uses a default CBR–Scala relationship.

- *Indirect methods*

To estimate SNP using the FWD, the method proposed by Tonkin & Taylor (Salt & Stevens 2001) is recommended because it has been derived using typical New Zealand pavements. This method is also recommended for network surveys, and for the determination of the homogeneity of a length of pavement before a full analysis is performed.

- *Pavements with volcanic subgrades*

Volcanic subgrades need to be treated with caution, and the above methods are not always appropriate. Therefore a second SNP should be derived using the back-calculated modulus of the subgrade. This SNP may be a better predictor of cracking and, until the calibration of the HDM model is complete, SNP assessments of volcanic subgrades should be determined by both CBR–modulus (using the basic method) and direct FWD methods.

Research is urgently required to derive an indirect method and to adapt the FWD method for determining SNP for these materials.

11. References

AASHO. 1962. The AASHO Road Test. *HRB Special Report 61A-61E.* Highway Research Board (HRB), Washington DC.

AASHTO. 1986. *Guide for the design of pavement structures.* Volume II. AASHTO (Association of American State Highway & Transportation Organisations), HRB, Washington DC.

AUSTROADS. 1992. *Pavement Design: A guide to the structural design of road pavements.* AUSTROADS, Sydney, Australia.

ASTM 1993. Test method for CBR (Californian Bearing Ratio) of soils in place. *ASTM D4429-93.* American Society of Testing & Materials, Philadelphia, USA.

Emery, S.J. 1985. *Prediction of moisture content for use in pavement design.* PhD Dissertation, University of the Wittwatersrand, Johannesburg, Republic of South Africa.

HTC Infrastructure Management Ltd. 1996. Establishing pavement strength for use with dTIMS. Consultants' Report, HTC, Kumeu, Auckland.

HTC Infrastructure Management Ltd. 2000. *Predictive modelling for road management in New Zealand.* dTIMS program, produced by HTC, Kumeu, Auckland.

HTC. 2000. Pavement strength. Unpublished HTC report (ALGENZ 30/06/2000). 22pp.

Jameson, G.W. 1993. Development of procedures to predict structural number and subgrade strength from falling weight deflectometer deflections. *AUSTROADS PRG Report No.3.*

MWD Central Laboratories. 1986. Unpublished report. *MWD Central Laboratories Report M2 86/27.*

Paterson, W.D.O. 1987. Road deterioration and maintenance effects: models for planning and management. The Highway Design and Maintenance Standards Model. (HDM-III) Volume III. *Highway Design and Maintenance Standards Series,* Transportation Department, World Bank, Washington DC. John Hopkins University Press, Baltimore, Maryland, USA.

Roberts, J., Roper, R. 1998. The ARRB integrated project level pavement performance and life cycle costing model for sealed granular pavements. *ARRB Research Report ARR324.* Australian Roads Research Board (ARRB), Melbourne, Australia.

Rohde, G.T. 1995. *The use of FWD deflections in HDM4.* Report to the International Study of HDM4, University of Birmingham, UK.

Rohde, G.T., Hartman, A. 1996. Comparison of procedures to determine structural number from FWD deflections. *Proceedings of Roads 96 Conference, Part 4:* 99-116.

Salt, G., Stevens, D. 2001. Pavement performance prediction. Determination and calibration of structural capacity (SNP). *Proceedings of 20th ARRB Conference,* Melbourne.

Sutherland, A., Dongol, D.M.S., Patrick, J.E. 1997. Applications of AUSTROADS pavement design guide for Wanganui materials. *Transfund New Zealand Research Report No. 128.* 84pp.

Tonkin & Taylor Ltd. 1998. Pavement deflection measurement and interpretation for the design of rehabilitation treatments. *Transfund New Zealand Research Report No.117.* 70pp.

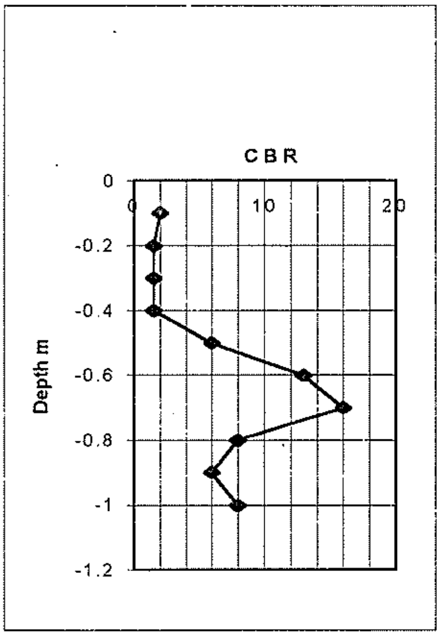
TRRL 1977. Guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical countries. *TRRL Road Note 31.* Transport & Road Research Laboratory, Crowthorne, UK.

Ullitdz, P. 1987. Pavement analysis. *Developments in Civil Engineering 19.* Elsevier, The Hague.

University of Birmingham. 1998. *Specifications for the HDM-4 Road Deterioration Model for bituminous pavements.* Fifth draft, Nov. 1998, University of Birmingham.

Watanatada, T., Harral, C.G., Paterson, W.D.O., Dhareshwar, A.M., Bhandari, A., Tsunokawa, K. 1987. The Highway Design and Maintenance Standards Model. (HDM-III) 2 volumes. *Highway Design & Maintenance Standards Series,* John Hopkins University Press, Baltimore, Maryland, USA.

Figure 4.8 CBR inferred for a subgrade of 100 mm depth, using DCP or Scala test.



4.2 Modulus Method

Back-calculation of layer modulus from the FWD test and then assigning strength factors has become a common method to obtain an "accurate" measure of the SNP.

The method is similar to the CBR method in that, for each layer, a strength coefficient is obtained from the modulus of the layer. But the method also requires a relationship between the subgrade modulus and CBR because the contribution of the subgrade in this method is still based on the subgrade CBR. Therefore, where an FWD test is performed, a relationship between modulus of the subgrade and CBR is assumed. Emery (1985) has proposed one relationship (Equation 6) that has been incorporated in HDM4. For comparison, the AUSTROADS relationship (1992) (modified for isotropic conditions) as described by Tonkin & Taylor (1998) are compared in Figure 4.9.

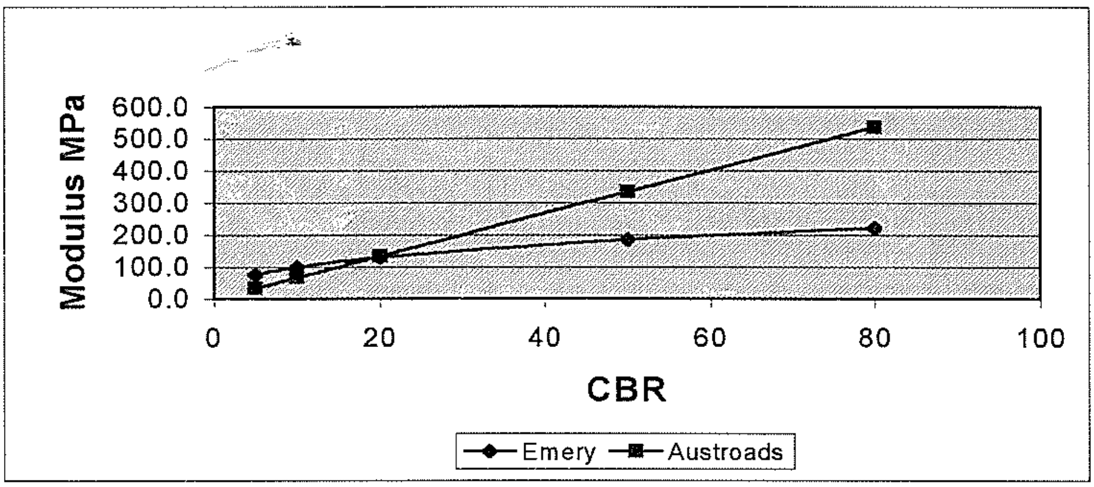


Figure 4.9 Comparison of relationships of subgrade modulus–CBR obtained by two direct methods (from Emery 1985, AUSTROADS 1992).

Correspondence between the direct CBR and FWD methods depends on the assumed relationship between CBR and Modulus. This relationship is not well defined and errors in the order of a factor of 2 can occur (Tonkin & Taylor 1998). This error is of the same order as was described by the CBR–Scala relationship and can result in differences of up to 0.7 in the calculated SNP.

The structural coefficient of the unbound pavement layers is based on the layer modulus. Relationships are given in the HTC report (2000) and Rohde & Hartman (1996). Figures 4.10 and 4.11 compare the shape of the CBR and moduli 'a_2' (basecourse) and 'a_3' (sub-base) relationships.

Figure 4.10 Comparison of three CBR–modulus relationships with strength coefficient a_2 (basecourse strength factor) (based on AASHTO 1986, CBR 10, Ullidtz 1987).

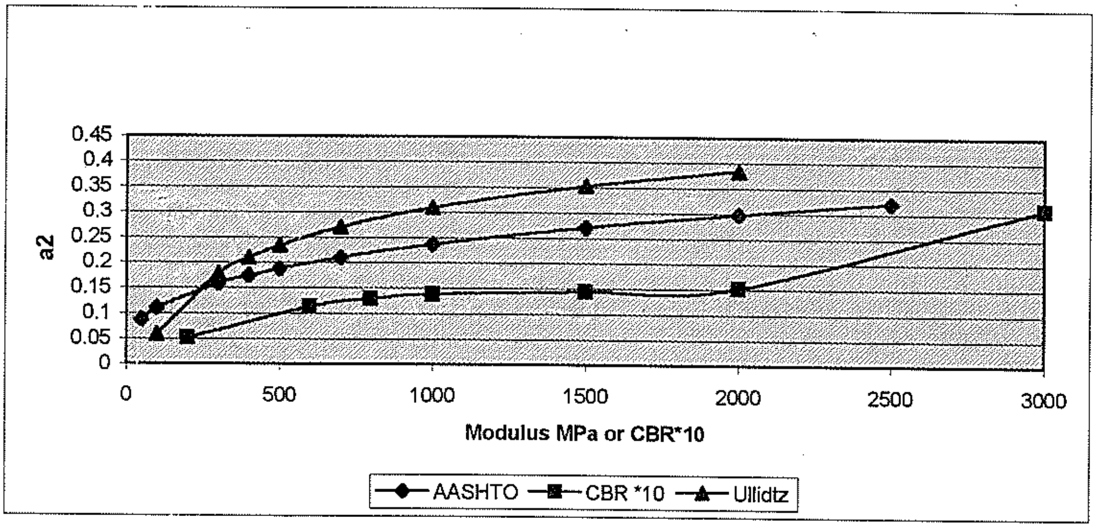


Figure 4.11 Comparison of CBR–modulus relationship with strength coefficient a_3 (sub-base strength factor) (based on CBR 10, AASHTO 1986).

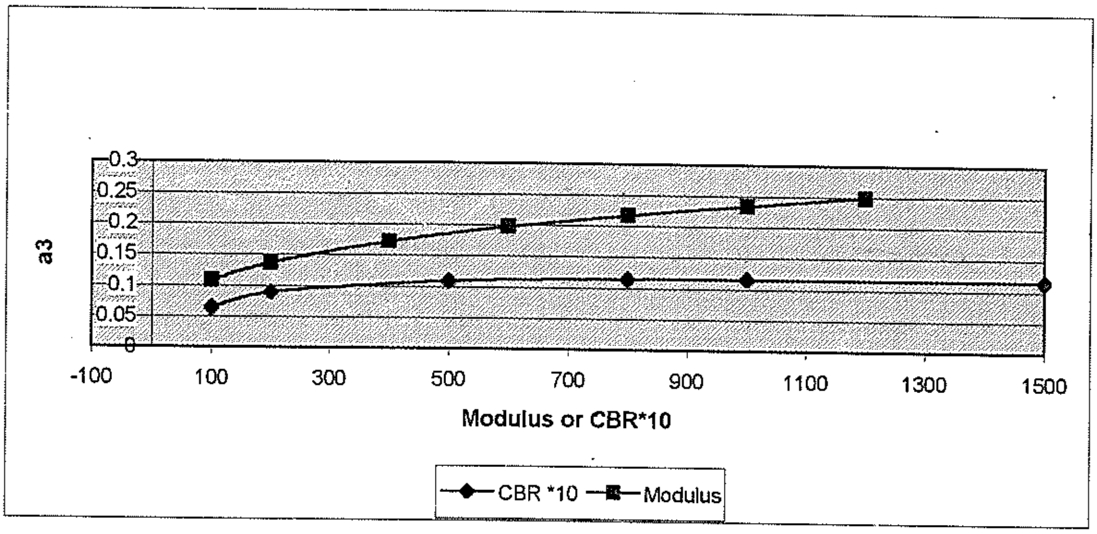


Figure 6.1 compares the SNP calculated from CBR (Scala) (method c in Table 6.1), with the two FWD methods (a, b in Table 6.1). It shows that results from method b are more precise than those calculated with assumed layer thicknesses.

Besides the difference in the average SNP obtained from the FWD methods, some of the pavements exhibit a difference in the variances. This means that statistically the methods are measuring different properties.

Based on analysis using the F test, the SNP based on FWD (by the Tonkin & Taylor method) and on CBR, are different for the Airport (1) and Racecourse (6) pavement test sites at the 5% confidence level, and for the Huntly (4) test site at the 10% confidence level.

Figure 6.1 Comparison of SNP obtained from FWD methods with CBR(Scala) method.