CHARACTERISATION OF PAVEMENT STRENGTH IN HDM-III AND CHANGES ADOPTED FOR HDM-4

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1. INTRODUCTION

The World Bank's HDM-III model contains relationships for predicting road deterioration and maintenance effects as functions of pavement characteristics, traffic and the environment (Watanatada et al., 1987). The relationships in HDM-III were derived primarily from a pavement performance study based in Brazil in the late 1970's (Paterson, 1987) but have been validated using data from a number of other studies over a range of climates. However, it has generally been accepted that there are some limitations to these relationships and so an international study was initiated in 1993 to update the relationships and the software. One of the outputs of the study was a proposal for revised and extended models to be incorporated into HDM-4 (N D Lea International Ltd, 1995).

The N D Lea report was subsequently reviewed and one of the issues that required further work concerned the assessment of pavement strength and the methods by which it can be characterised. Pavement strength is characterised in HDM-III by the use of the modified structural number (SNC) (Hodges et al., 1975). This is a parameter which is calculated from data on the strength and thickness of each layer in the pavement as well as the strength of the underlying subgrade.

This paper considers the accuracy of the modified structural number approach in estimating pavement strength and a revised method is developed which greatly improves the reproducibility of such calculations and largely eliminates the subjective judgements that were often required.

2. BACKGROUND

2.1 Modified Structural Number and its Limitations.

The concept of structural number was first introduced as a result of the AASHO Road Test as a measure of overall pavement strength. It is essentially a measure of the total thickness of the road pavement weighted according to the 'strength' of each layer and calculated as follows:

$$SN = \sum a_i \, d_i \qquad\qquad\qquad (1)$$

   where

           i is a summation over layers
           a_i is a strength coefficient for each layer
           d_i is the thickness of each layer measured in inches

In the original analysis the strength coefficients were treated as model parameters and the pavement performance data analysed on the basis that sections of road with the same structural number should carry the same total traffic before reaching a defined terminal condition at which major maintenance was required. After the performance analysis had been completed and the strength coefficients for the various materials had been derived, correlation studies were undertaken to relate the strength coefficients to the more usual engineering tests of material strength such as CBR for granular materials, unconfined compressive strength for cemented materials and Marshall stability for bitumen bound materials.

The AASHO Road Test was constructed on a single uniform subgrade therefore the effect of different subgrades could not be estimated and the structural number could not include a subgrade contribution. Pavements of a particular structural number but built on different subgrades will therefore not carry the same traffic to a given terminal condition. To overcome this problem and to extend the concept to all subgrades, a subgrade contribution was derived as described by Hodges et al., (1975) and a modified structural number defined as follows:-

$$SNC = SN + 3.51 \left(\log_{10} CBR_S \right) - 0.85 \left(\log_{10} CBR_S \right)^2 - 1.43 \qquad (2)$$

where CBR_S is the in-situ California Bearing Ratio of the subgrade.

This modification has been used extensively and forms the basis for defining pavement strength in many pavement performance models.

It should be recognised that assigning strength coefficients in this way to particular materials can be only an approximate estimate of the relative contribution that the different materials make to the overall structural strength of the pavement. In reality the strength coefficients should depend on the position of the material in the structure, the properties of the other materials, the thicknesses of the layers, the environmental conditions and the types of failure to be expected.

For example, a material which is perfectly acceptable at a low level in the pavement, performing its proper function of spreading traffic loads and reducing the stresses on the weaker layers below, may itself fail if it is incorporated into the structure nearer to the surface where stresses are higher. With a conventional 3-layer pavement comprising a well defined surfacing, roadbase and sub-base, the original AASHO relationships take this partly into account for granular, unbound material. When an unbound material of low CBR is used as roadbase, the strength coefficient (a_2) is less than when it is used for sub-base (a_3), reflecting the poorer performance that would be obtained if such material were to be used at higher levels in the pavement. At high CBR the material is equally acceptable for use as either roadbase or sub-base hence the coefficients are similar.

The strength coefficients therefore depend not only on the inherent strength of the materials but also on the stresses and strains to which the layers are subjected. These stresses and strains depend, in turn, on the in-situ properties of the other layers and, therefore, also on the environmental conditions. Although increasing the strength of any one layer must improve its performance under the same conditions of stress and strain, this is not necessarily true if the layer structure is altered so that the magnitudes of these stresses and strains also change, even though $\Sigma a_i d_i$ remains unchanged. There are numerous possible structures which can have the same structural number but with completely different distributions of stress and strain and therefore their performances are likely to differ.

Despite these difficulties the concept of a single number to indicate pavement performance is valuable and works well for roads similar in structure to those incorporated into the AASHO road test. Outside this range the user is essentially extrapolating empirical data and should do so with caution.

This paper is concerned not with an extrapolation as such, but with an anomaly in the use of the structural number concept when the road apparently consists of a large number of layers. The paper is based primarily on empirical evidence from pavement research around the world and on certain logical consequences derived from the original AASHO model.

2.2 The Problem

Many road pavements cannot be easily divided into three distinct layers with a well-defined and uniform subgrade; frequently, there are numerous layers and the strengths often vary with depth. Hence, when calculating the structural number according to eq.(1), the engineer has to judge which layers to define as roadbase, which as sub-base, and where to define the top of the subgrade. For many roads the subgrades are quite strong, and frequently of sub-base quality. The simple summation of eq.(1) allows the engineer to obtain almost any value of structural number since the value will always increase as the depth at which the subgrade is assumed to begin increases.
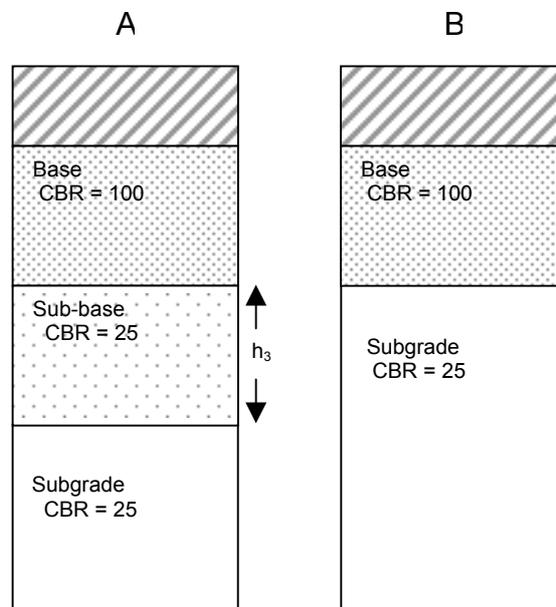

Figure 1 Identical pavements, different structural numbers

Since the two roads are identical, the pavements should have the same structural number but pavement *A* has a *SN* value which is greater than that of *B* by an amount equal to $a_3 \times h_3$. The problem arises because the contributions to the structural number are independent of depth. This cannot be correct. Logic dictates that a layer very deep within the subgrade, no matter how strong, can have little or no influence on the performance of the road. To eliminate the problem, a method of calculating the *SN* must be devised which gives the same answer irrespective of the choice of h_3. For this to occur, the contribution of each layer to the overall structural number must decrease with depth in such a way that:

$$\sum_i a_3 . \Delta Z_i . f\,(Z) + SNG . f\,(h) \tag{3}$$

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is independent of h. Here ΔZ_i is the thickness of layer i in the sub-base/subgrade, SNG is the subgrade contribution to SN, and $f(Z)$ is a function which ideally decreases monotonically as Z increases. The problem reduces to finding a suitable functional form for $f(Z)$.

Expression (3) can be written

$$a_3 . \int_0^h f(Z).dZ \; + \; SNG \;.f(h) \tag{4}$$

and its value must be independent of h and equal to the value obtained from carrying out the SN calculation using conventional thicknesses for sub-base and base.

3.   A SOLUTION

3.1   Adjusting the Contributions from Sub-base and Subgrade

If we have a pavement in which the sub-base and subgrade are of the same strength, we can define the sub-base/subgrade boundary anywhere. In calculating the structural number, we should obtain the same answer. Assuming the origin of coordinates is at the top of the sub-base we can choose the boundary at $Z = 0$, $Z = h_3$ and $Z = \infty$ and obtain:

$$a_3. \int_0^\infty f(Z).dZ = a_3. \int_0^{h3} f(Z).dZ + f(h3).SNG = f(0).SNG \tag{5}$$

An exponential function of the form:

$$f(Z) = \left(A_0 \; - \; A_1 \exp^{-\alpha z} \right) e^{-\beta Z} \tag{6}$$

has been shown to meet the requirements. By choosing a function of this form, which can have two points of inflection, it is possible to select a set of coefficients which

a)   minimises the difference between this new approach and the original modified structural number over the usual range of pavement types
b)   reduces the subgrade contribution for deep pavements (for the reasons discussed above).

3.2   Redefining the Subgrade Coefficient

Using this function in the limiting cases ($Z = 0$ and $Z = \infty$) produces :-

$$a_3 \left(\frac{A_0}{\beta} \; - \; \frac{A_1}{\alpha + \beta} \right) = \left(A_0 \; - \; A_1 \right) \; SNG \tag{7}$$

This implies that the ratio of a_3 to SNG should be constant, or at least as constant as possible subject to the requirement that the new method of calculating SNC should agree with the original method for normal structures where it is known to be reliable. As a first approximation, it is assumed that the ratio is indeed constant.

Comparison of a_3 with SNG indicates that the original values of a_3 do not always satisfy this relationship. However if we assume that values of SNG have been established from empirical evidence as embodied in design charts, it is reasonable to assume that the SNG term is the more correct of the two. Furthermore, since low CBR material is not normally used for sub-base, the relationship between CBR and a_3 for weak material is an extrapolation and is therefore likely to be less accurate. Thus it is justifiable to adjust the values of a_3 at low CBR to ensure that the above condition holds. The following Table illustrates the changes.

Table 1: Variation of sub-base and subgrade contributions with CBR

CBR	3	5	10	15	20	30	50
SNG	0.05	0.61	1.23	1.52	1.70	1.90	2.08
a_3	0.003	0.032	0.065	0.080	0.089	0.100	0.109

where the ratio

$$\frac{a_3}{SNG} = 0.0525 \tag{8}$$

and

$$a_3 = -0.075 + 0.184 \ (\log_{10} CBR) - 0.0444 \ (\log_{10} CBR)^2 \tag{9}$$

The difference between this equation and the original AASHO equation is quite small over the range where a_3 was originally derived. The relationship ensures that if the sub-base and subgrade are of similar strength, the resultant SN can be independent of the choice of sub-base/subgrade boundary. The actual magnitude of the structural number contribution from sub-base and subgrade will depend on the value of $f(0)$ or the coefficients $A_0 - A_1$.

Unfortunately most design charts do not help us to determine this for designs where the sub-base/subgrade strengths are similar. Design charts are often two-valued at subgrade strengths where the subgrade material is acceptable as a sub-base. For example, in several design charts no sub-base is required whenever the subgrade has a CBR of 30 per cent or more but if the subgrade has a CBR of slightly less than this, say 25 per cent, a minimum thickness of sub-base of 100 mm is recommended. Therefore it is necessary to calculate the SNC for a wide range of acceptable pavement structures to obtain a smooth relationship between SNC and traffic.

In addition, no design methods permit the sub-base to be too weak, hence a uniform strength

for sub-base and subgrade of, say, 15 per cent is unacceptable, although values such as this are commonly found.

3.3 Specification and Verification of the Approach

To determine the coefficients in the function $f(Z)$ it is better to examine well established pavement designs and to select coefficients in such a way that the proposed method of calculation and the original method give similar results. Since $f(Z)$ is exponential with constant exponents, it is not possible for this to hold true over **all** structures.

Data from various design charts in Overseas Road Note 31 (TRL, 1993) were entered into a spreadsheet and the original method was compared with the adjusted method for these designs. Using an iterative approach, and the fact that $(A_0 - A_1)$ should be close to unity, the following values were found to give the closest agreement :

$$A_0 = 1.6 , \quad A_1 = 0.6 , \quad \alpha = 0.008 , \quad \beta = 0.00207$$

Figures 2 and 3 illustrate the comparison for Chart 1 and Chart 8 of the Road Note. It can be seen that a reasonably close fit to the original method of calculation is obtained.

The 'theoretical' graph illustrates the comparison for each structure with a CBR assigned to each layer as given in the chart. However, in practice the CBR of a given layer will depend on the underlying layers. For example, it will probably not be possible to obtain an in-situ CBR of 15% immediately above a subgrade of CBR 2 % (Chart 8, S1T1), but the CBR of this overlying sub-base layer will increase as the depth below the surface decreases. Estimates of this effect have been made and these are the structures included in the 'practical' graphs.

The function $f(Z)$ is therefore :

$$f(Z) = (1.6 - 0.6.e^{-0.008.Z}) \; e^{-0.00207.Z} \tag{10}$$
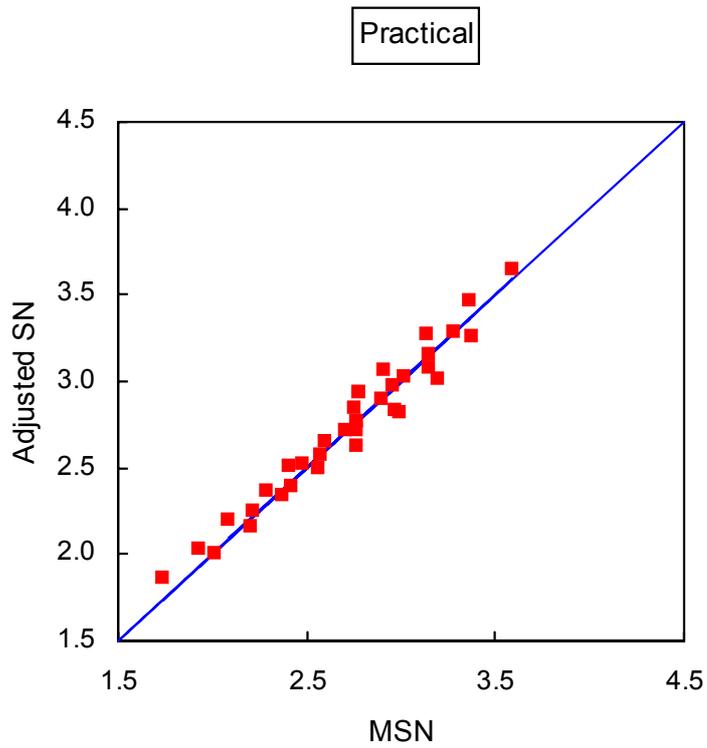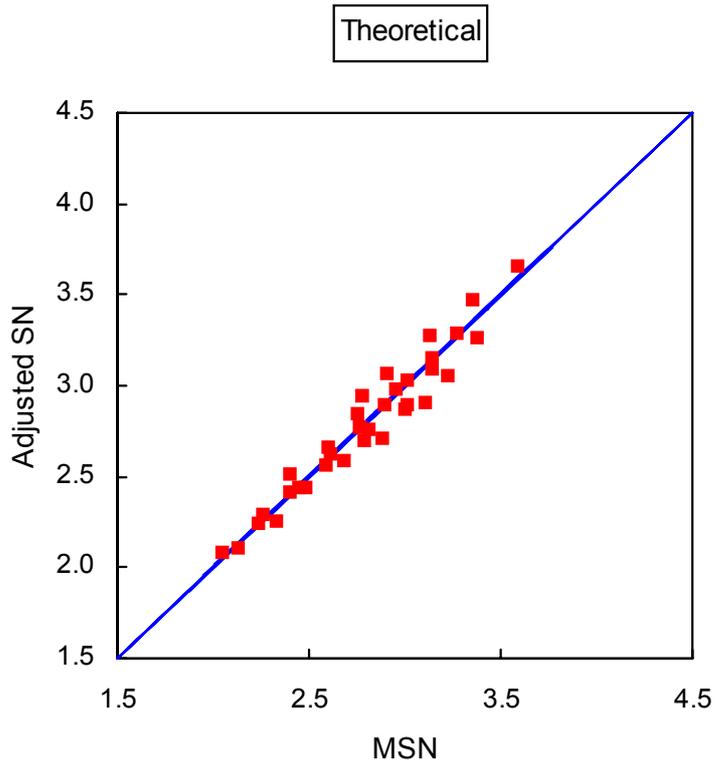
   where Z is defined in mm.

Figure 2. Comparison of the new adjusted method to the original method for typical pavement structures (Road Note 31 Chart 1)
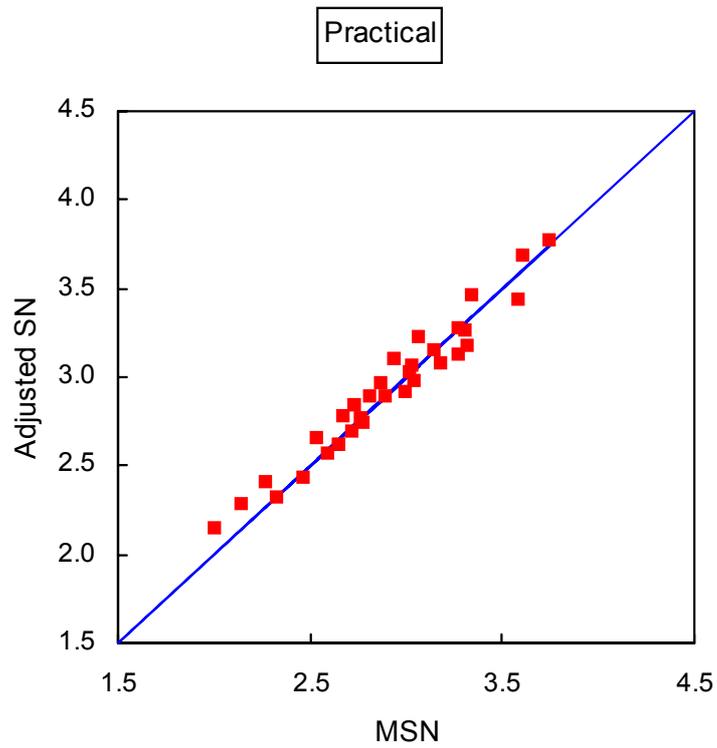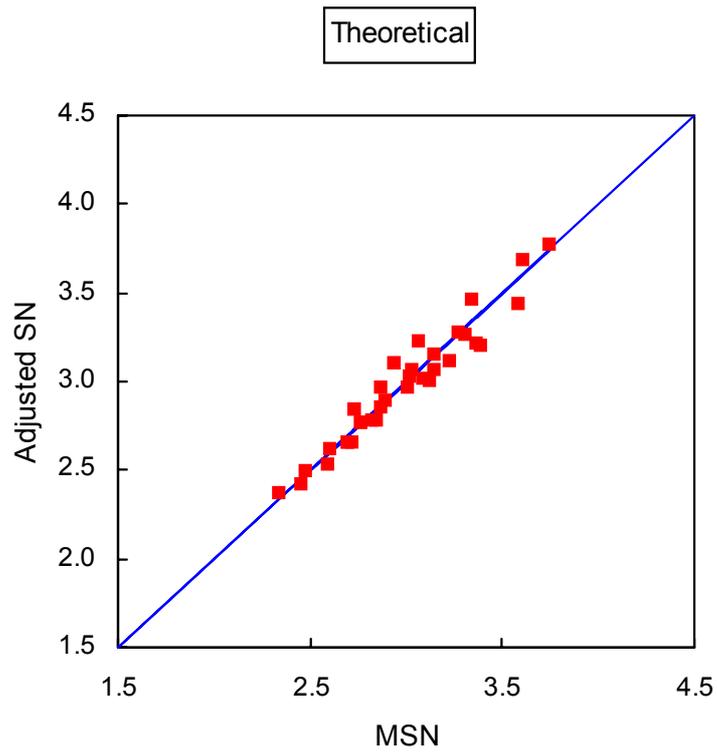
Figure 3. Comparison of the new adjusted method to the original method for typical pavement structures (Road Note 31 Chart 8)

Table 2 illustrates the effect of changing the sub-base/subgrade boundary for a typical example. To ensure that the correct SNC is obtained in all circumstances, the user must define a minimum thickness of sub-base of 200 mm. If the strength changes within this depth, the sub-base can be defined as several layers. For sub-base thicknesses greater than 200 mm, Table 2 shows that the result is independent of the choice of h_3.

Table 2 :Sub-base and subgrade contributions to adjusted structural number

Depth of top of subgrade	Thickness of sub-base (h_3)	$a_3 \int_0^h f(Z)\ dZ$	$f(h_3).SNG$ $= f(h_3)\ x1.23$	Total SN (excluding base)
200	0	0	1.23	1.23
300	100	0.27	1.33	1.60
400	200	0.54	1.20	1.74
500	300	0.77	1.02	1.79
600	400	0.96	0.85	1.80
700	500	1.12	0.69	1.81
800	600	1.25	0.57	1.81
900	700	1.35	0.46	1.81
1000	800	1.44	0.38	1.81

Sub-base and subgrade = 10% CBR
Base = 200 mm
The top of the subgrade is assumed to be at a depth ranging from 200 to 1000 mm from surface

Similar results are obtained for the practical range of sub-base and subgrade strengths

4.   SUMMARY AND CONCLUSIONS

Despite the slight inconsistencies described in Section 3, the new method of calculating structural number benefits considerably from the fact that it is not necessary to exercise subjective judgement in choosing where to define the boundary between sub-base and subgrade. The engineer can simply consider each change in strength as a sub-base layer until he has reached a layer of sufficient uniformity or depth to be considered as the subgrade. The method is therefore considerably more accurate and repeatable, especially where sub-bases and subgrades are of similar strength or where there are many layers eg. selected fill, capping layers.

For pavements with sub-bases of the typical pavement designs of Overseas Road Note 31, the new method will give a similar result to the original method. For structures with thicker sub-bases, the contribution of the sub-base and subgrade will decrease with depth so that the definition of sub-base/subgrade boundary will not be important.

Other models can undoubtedly be devised which give better results over a wider range of conditions. Such models would almost certainly require the parameters to be dependent on the interaction between layers and therefore iterative methods of solution would be required. In view of the approximate nature of the SN index itself, further refinement is considered unnecessary.

5. REFERENCES

American Association of State Highway Officials. 1972. *AASHO Interim Guide for Design of Pavement Structures*. AASHO, Washington DC, USA.

Hodges J W, J Rolt and T E Jones.1975. The Kenya road transport cost study : research on road deterioration. *TRL Research Report LR673*. Transport and Road Research Laboratory, Crowthorne, UK.

N D Lea International Ltd. 1995. *Modelling Road Deterioration and Maintenance Effects in HDM-4. Final report.* Asian Development Bank, Manila, Philippines.

Paterson W D O. 1987. *Road Deterioration and Maintenance Effects. Models for Planning and Management*. John Hopkins Press, Baltimore, USA.

Rolt J. 1984. An improved method of calculating pavement structural number for evaluation and design. *Overseas Unit Working Paper No 233*. Transport and Road Research Laboratory, Crowthorne, UK.

Transportation Department (1984). *Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization . - Brazil Road Deterioration Study : Listing of Paved Road Data Files.,* World Bank, Washington DC, USA.

Watanatada T, C G Harral, W D O Paterson, A M Dhareshwar, A Bhandari and K Tsunokawa (1987). *The Highway Design and Maintenance Standards Model. Volume 1. Description of the HDM-III Model*. John Hopkins Press, Baltimore, USA.