Risks in Using Materials Data Obtained from Repeated Load Triaxial Tests in Pavement Design and Analysis

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SUMMARY To design and to analyse granular pavements requires resilient moduli and Poisson's ratios for the granular base and sub-grade materials concerned. They can be obtained from repeated load triaxial tests. A typical basecourse crushed rock was tested using a range of cyclic and static confining pressures and the resilient moduli were derived and fitted to available models for resilient modulus. One of the models was found to produce comparable constants for both test types and, therefore, could provide a basis for correcting the moduli obtained from different methods of applying confining pressure. All the models were used in the program NONCIRL to predict the lives of some typical granular pavements selected from the NAASRA Pavement Design Guide. The predicted lives range from 0.3 to 3 times the NAASRA design lives and this indicates that the risks associated with the use of different characterisation models in pavement design and analysis are significant.

1. INTRODUCTION

The recent widespread acceptance of new mechanistic pavement design procedures, such as those adopted in the 1987 NAASRA Guide to the Structural Design of Road Pavements (1), has led to a need for improved material characterisation techniques for all classes of pavement materials, particularly for unbound granular materials and subgrades. At the National Workshop on Resilient Modulus Testing of Unbound Pavement Materials and Subgrades organised by the Australian Pavement Research Group (APRG) in South Australia, in February 1991 (2), there were consensus statements that the repeated load triaxial test with either cyclic or static confining pressure can be used to determine material resilient modulus and Poisson's ratio, which are used as input parameters in the mechanistic pavement design. However, as the two test types could produce different modulus values at the same stress levels (3, 4, 5, 6), their application to pavement design and analysis could produce different end results.

This paper describes the results for a typical base crushed rock determined by the two test types. Some available non-linear material models are used to fit the results and then incorporated in the program NONCIRL (7) to predict pavement lives for some typical granular pavements selected from the NAASRA Guide. Comparisons of pavement lives calculated with NONCIRL to those estimated by the NAASRA Guide enable identification of the risk associated with the use of different test types and different characterisation models in pavement design and analysis.

2. THE DESIGNED PAVEMENTS

Granular pavement structures as designed according to the NAASRA Guide consist of a thin sprayed chip seal surfacing of about 20 mm, a base layer and a sub-base layer laying on the subgrade. The predicted pavement life calculated using the NAASRA subgrade strain criterion is expressed as:

$$N = (8511/\varepsilon_v)^{7.14} \qquad (1)$$

where N is the number of Equivalent Standard Axles (ESA) and ε_v the critical subgrade strain at top of the subgrade calculated with the program CIRCLY (8), using the standard loading configuration as described in the NAASRA Guide.

Altogether 9 granular pavements were designed for three subgrade types (namely CBR3, CBR5 and CBR10, respectively) and for three design traffics (namely 10^5, 10^6 and 10^7 ESAs, respectively). The NAASRA Design Chart (Figure 8.4 in the 1987 NAASRA Guide) was used for selecting the total thicknesses of the base and sub-base layers of the 9 granular pavements, which are given in Table I.

TABLE I

PAVEMENTS ADOPTED IN THE STUDY.

Pav No	CBR	Total thickness (mm)	Design life (ESA)
1	3	400	10^5
2	3	520	10^6
3	3	640	10^7
4	5	300	10^5
5	5	390	10^6
6	5	480	10^7
7	10	200	10^5
8	10	260	10^6
9	10	320	10^7

3. CHARACTERISATION OF THE GRANULAR BASE/SUB-BASE MATERIAL

3.1. Testing methods

Four samples of a typical basecourse crushed rock, each with specified density of 2.20 t/m^3 and moisture content of 4 %, were prepared for repeated load triaxial testing to provide data for examining the difference in resilient modulus produced by the two test types.

For the samples 1 and 2, each sample was tested in accordance with the SHRP Protocol P46 testing procedure (9) using static confining pressures (σ_3) in the range of 20 kPa to 68 kPa and vertical/radial stress ratios (σ_1/σ_3) in the range of 2 to 10.

After the above test series, the same sample was subjected to another test series using cyclic confining pressures in the range of 20 kPa to 120 kPa and with stress ratios in the range of 2 to 10, as suggested in the ARRB testing procedures (10).

The samples 3 and 4 were tested in the reversing order, i.e. to start with the test series with cyclic confining pressures and then following with the test series with static confining pressures. The emphasis was to show the negligible effects of loading history on the resilient moduli derived in the tests.

The difference between the stress paths of the two test types can be seen in Figure 1, where they are plotted in the stress space of total stress, $\theta = (\sigma_1 + 2.\ \sigma_3)$, and deviator stress, $q = (\sigma_1 - \sigma_3)$. As shown in Figure 1, the two test types have the same peak total stress and deviator stress, but different initial static total stresses and different cyclic total stresses.

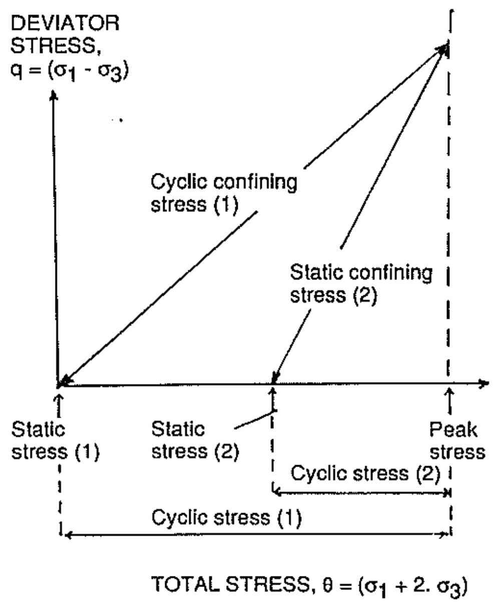

Figure 1 Stress paths for the two repeated load triaxial test types

3.2. Test results

The resilient modulus (E_r) and Poisson's ratio (v_r) were calculated using the generalised Hooke's law as:

$$E_r=(\sigma_{1r}-\sigma_{3r}).(\sigma_{1r}+2.\sigma_{3r})/(\varepsilon_{1r}.(\sigma_{1r}+\sigma_{3r})-2.\varepsilon_{3r1}.\sigma_{3r}) \quad (2)$$

$$v_r = (\varepsilon_{1r}.\sigma_{3r}-\varepsilon_{3r}.\sigma_{1r})/(\varepsilon_{1r}.(\sigma_{1r}+\sigma_{3r})-2.\varepsilon_{3r1}.\sigma_{3r}) \quad (3)$$

where σ_{1r} and σ_{3r} are the cyclic vertical and radial stresses, respectively; and ε_{3r} and ε_{1r} are the measured radial and vertical resilient strains, respectively.

In the test with static confining pressure, $\sigma_{3r} = 0$, the above equations are simplified as:

$$E_r = \sigma_{1r}/\varepsilon_{1r} = q_r/\varepsilon_{1r} \quad (4)$$

$$v_r = -\ \varepsilon_{3r}/\varepsilon_{1r} \quad (5)$$

The moduli obtained from a typical test are plotted against peak mean stresses, $\sigma_m = \theta/3$, as shown in Figure 2. The results indicate that, for the same peak mean stresses, the modulus obtained from the tests with static confining pressure are generally higher than those obtained from the tests with cyclic confining pressures.

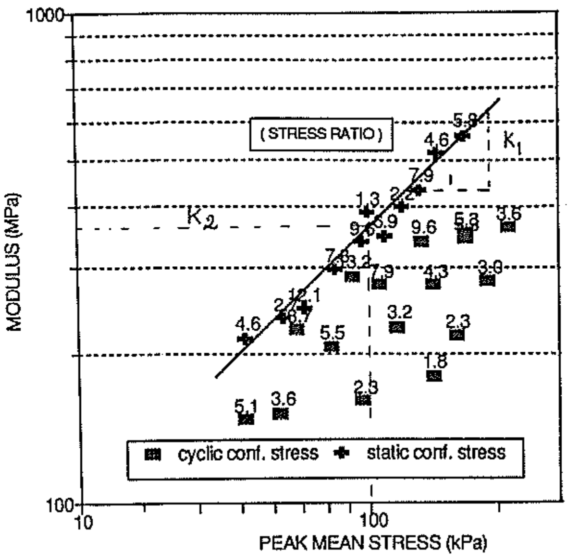

Figure 2 Typical results fitted with chacraterisation Model 1 (Sample No. 3)

3.3. Characterisation models for granular materials

There have been many characterisation models developed for different granular base materials using different testing techniques (3, 4, 5, 11). They are often expressed as functions of some common stress parameters, such as confining stress, σ_3, deviator stress, $q = \sigma_1 - \sigma_3$, total stress, $\theta = \sigma_1+2.\sigma_3$, mean stress, $\sigma_m = \theta/3$, and stress ratio $= \sigma_1/\sigma_3$. This study considers four models, which are closely fitted to the test results, as described below.

3.3.1 Model 1: Modulus Vs peak mean stress (for tests with constant confining pressure)

Past studies of resilient response of granular material using the method of applying static confining pressure (11) reported that the resilient modulus is a function of the peak total stress; i.e.

$$E_r = K_2.\theta^{K1} \quad (6)$$

To allow K_2 to have the same unit as E_r (MPa), Equation 6 can be expressed in another form of

$$E_r = K_2.(\sigma_m/\sigma_{Ref})^{K1} \quad (7)$$

where $\sigma_m = \theta/3$ is the peak mean stress and σ_{Ref} is the reference stress of 100 kPa.

Typical results fitted by Model 1 are shown in Figure 2. Average values of K_1 and K_2 obtained from all the tests are 0.70 and 340 MPa.

3.3.2. Model 2: Modulus Vs peak mean stress and stress ratio (for tests with cyclic confining pressure)

Studies for various granular materials carried out at ARRB using the test with cyclic confining pressure (12, 13, 14, 15) reported that the resilient modulus is a function of both peak mean stress and stress ratio (σ_1/σ_3).

In this case, the model has the same form as that of Equation 7 and with the same value for K_1, but K_2 is a function of peak stress ratio (σ_1/σ_3), i.e

$$K_2 = K_{2max} - S_1.\{\text{Log}7-\text{Log}(\sigma_1/\sigma_3)\} \quad \text{for } (\sigma_1/\sigma_3)<7 \tag{8}$$

$$K_2 = K_{2max} \quad \text{for } (\sigma_1/\sigma_3)\geq 7 \tag{9}$$

Typical results fitted by Model 2 are shown in Figure 3. Average values of K_{2max} and S_1 obtained from all the tests are 270 and 230 MPa, respectively.

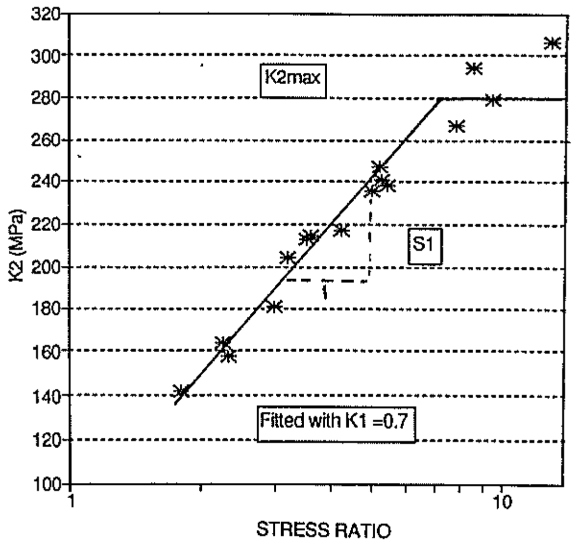

Figure 3 Typical results fitted with chacraterisation Model 2 (Sample No. 3)

3.3.3. Model 3: Modulus Vs average mean stress (for both test types)

Vuong (6) also reported that the two test types could produce comparable moduli at the same average mean stress, $\sigma_{mAve.} = (\sigma_{mstatic} + \sigma_{mpeak})/2$, providing that the peak stress ratios (σ_1/σ_3) in tests with cyclic confining pressures are greater than 5. The relationship is expressed as

$$E_r = K_3.(\sigma_{m\,Ave.}/\sigma_{Ref})^{K1} \tag{10}$$

Typical results fitted by Model 3 are shown in Figure 4. Average value for K_3 obtained from all the tests is 430 MPa.

3.3.4. Model 4: Modulus Vs average mean stress and stress ratio (for tests with cyclic confining pressure)

To fit more closely the results obtained from tests with cyclic confining pressure, the parameter K_3 in Equation 10 is expressed as a function of peak stress ratio (σ_1/σ_3), i.e.

$$K_3 = K_{3max} - S_2.\{\text{Log}7-\text{Log}(\sigma_1/\sigma_3)\} \quad \text{for } (\sigma_1/\sigma_3)<7 \tag{11}$$

$$K_3 = K_{3max} \quad \text{for } (\sigma_1/\sigma_3)\geq 7 \tag{12}$$

Typical results fitted by Model 4 are shown in Figure 5. Average value for K_{3max} is the same as the average value for K_3 in Model 3, whereas the average value for S_2 obtained from all the tests is 360 MPa

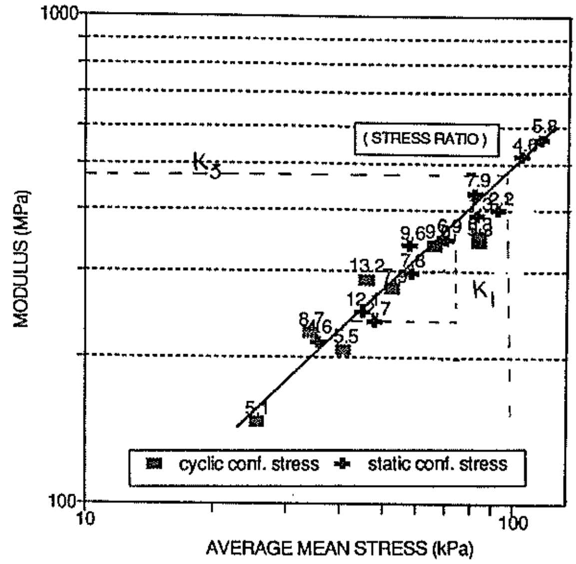

Figure 4 Typical results fitted with chacraterisation Model 3 (Sample No. 3)

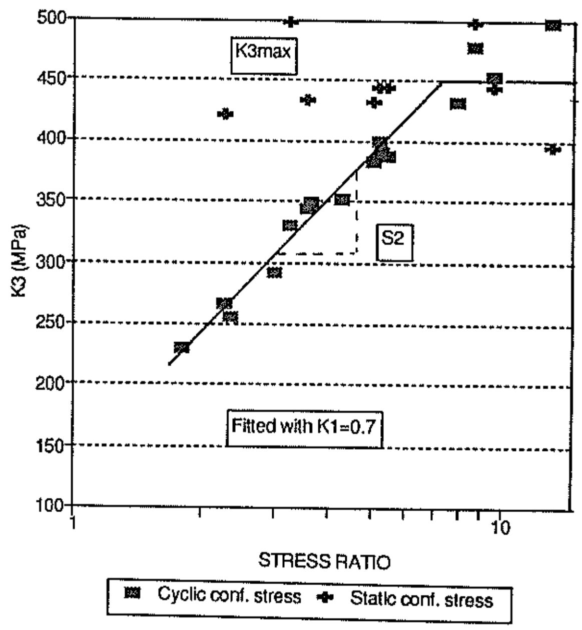

Figure 5 Typical results fitted with chacraterisation Model 4 (Sample No. 3)

4. CHARACTERISATION OF THE SUBGRADES

In the NAASRA Guide, the subgrade materials are assumed to be elastic and cross-anisotropic of Ev/Eh=2.0 and the vertical modulus of a subgrade can be determined from the empirical relationship of

$$E_v = 10.\text{CBR} \tag{13}$$

However, Vuong (13, 14, 15) reported that resilient modulus of subgrades can reduce with increasing deviator stresses. Generally the subgrade modulus can be expressed as:

$$E_r = E_{max}\,(1 - q/q_{ref})^p \tag{14}$$

and

$$E_r \geq E_{min} = E_{max}/4 \tag{15}$$

where q is the deviator stress; q_{ref} is the reference stress of 300 kPa; and E_{max} and p are the material constants

Vuong (16) also suggested typical E_{max} values of 32 MPa, 61 MPa and 145 MPa for the subgrades of CBR 3, 5 and 10, respectively, and a typical p value of 2. The Poisson's ratio for the subgrade is assumed to be a constant of 0.45.

5. STRUCTURAL ANALYSIS USING NONCIRL

5.1 Brief description of NONCIRL

In order to carry out structural analyses of flexible pavements, the NAASRA Guide uses a computer program CIRCLY (8), which is based on linear elastic layer theories. To make this program more versatile, it has been modified to NONCIRL (7) for analysing pavements having non-linear material characteristics. NONCIRL uses CIRCLY in the iterative process to calculate layer moduli of pavement layers and subgrades for given modulus-stress relationships. It also considers correction for tensile stresses in unbound layers and for overburden static pressures.

The computing process used in NONCIRL has the flow chart shown in Figure 6.

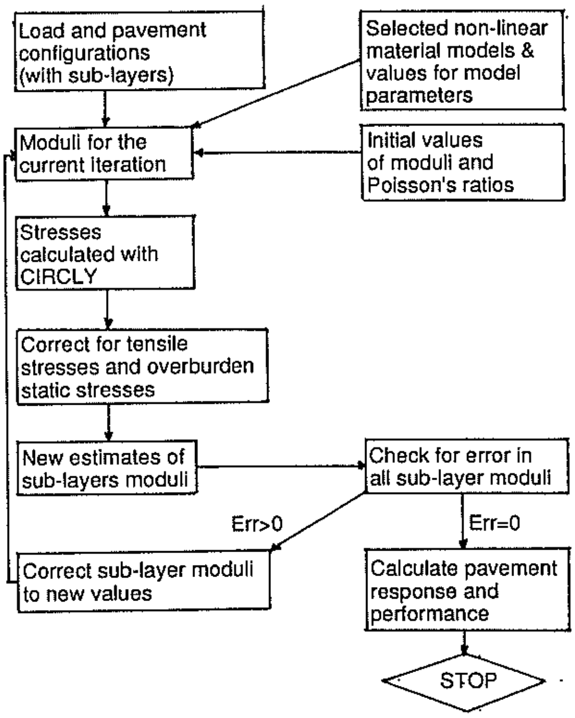

Figure 6 Flow chart for calculation of elastic moduli for materials having stress-dependent characteristics using NONCIRL.

To calculate the modulus values, E_r, for the final main analysis, a number of iterations were performed with E_r being calculated and updated based on the existing peak mean stress, average mean stress and peak stress ratio for each sub-layer. These are also checked for tensile stresses.

In the program, the cyclic stresses are the stresses induced from the applied wheel load; whereas the static stresses are the earth pressures at rest, which are assumed to be the same as the overburden pressure, i.e.

$$\sigma_{1s} = \sigma_{3s} = g.H \tag{16}$$

where g is the specific density of the material (in kN/m^3) and H is the depth to the point where the stresses are calculated.

Equilibrium is also checked after each iteration using the following measure of error in the calculated sub-layer moduli:

$$\text{Error} = \textbf{SUM}\,(E_i - E_{i-1})^2/E_i^2 \tag{17}$$

where E_i and E_{i-1} are the sub-layer moduli for the current and previous iterations, respectively.

5.2 Pavement and loading configurations

The granular layers and the subgrade were divided into sub-layers and the modulus of each sub-layer was then calculated accordingly to maximum stresses at the middle of the sub-layers, usually occurring at the vertical loading axis through the loading centre or through the centre of the loading group. The number of sub-layers of the base and sub-base layers for the nine pavements were 4, 5, 6, 4, 5, 6, 4, 4 and 4, respectively; whereas the subgrade was divided into 4 sub-layers having thicknesses of 250 mm, 350 mm, 500 mm and a semi-infinite bottom layer.

The Equivalent Standard Axle load (ESA) applies a uniform pressure of 550 kPa applied on two circular areas, each with an radius (R) of 0.107 m and a spacing between the loading centres of 0.330 m.

5.3. Analyses using different characterisation models for unbound materials

As shown in section 3, it is uncertain which testing method and material model for unbound material would be appropriate for use in NONCIRL. To show the risks associated with the use of different test types and different characterisation models in pavement design and analysis, five analyses were considered:

(a) Analysis 1 (AN1): This analysis uses Model 1 for the base material and assumes that the material is weightless and can have tensile stress. The purpose of this analysis is to demonstrate that the above assumptions can significantly affect the results.
(b) Analysis 2 (AN2): This analysis uses Model 1 for the base material and assumes that the material has a unit weight of 22.5 kN/m^3 and that tensile stress is unable to develop in the material. These assumptions are more realistic than those used in AN1.
(c) Analysis 3 (AN3): This analysis uses Model 2 for the base material and the same assumptions as made in AN2.
(d) Analysis 4 (AN4): This analysis uses Model 3 for the base material and the same assumptions as made in AN2.
(e) Analysis 5 (AN5): This analysis uses Model 4 for the base material and the same assumptions as made in AN2.

All analyses used Equation 14 for subgrade modulus and a typical value of E_v/E_h of 2 for all granular and subgrade sub-layers (E_v is vertical modulus and E_h is horizontal modulus) as suggested by the NAASRA Guide.

The input parameters for the material models for the base material and the subgrades are given in section 3 and section 4, respectively.

6. COMPARISON OF MODULI

The percentage differences between the calculated moduli for the five analyses and the NAASRA estimates, for a typical pavement, are plotted in Figure 7 for examination. In this case the NAASRA estimates for the 6 granular sub-layers, from top to bottom, are 500, 340, 232, 158, 108, 73 Mpa, respectively.

As shown in Figure 7, all analyses produced different moduli as compared to those estimated by the NAASRA Guide. The results also show that the analysis AN1 produced the lowest moduli, particularly at depths below 100 mm, because it assumed weightless materials and allowed tension in the base sub-layers and hence reduced significantly the peak stresses and sub-layer moduli.

By considering the overburden pressures and by correcting tensile stresses in the granular sub-layers, the analysis AN2 produced much higher moduli than those estimated by AN1.

The analysis AN3 (characterisation Model 2) produced very conservative estimates for all sub-layers as compared to those estimated by AN2 (characterisation Model 1). This reflects the difference in the constant K_2 in the models determined by the two test types.

The analyses AN4 and AN5 produced similar estimates of moduli for all sub-layers, except the top sub-layer. This indicates that the characterisation Model 3 and Model 4 could provide a basis for correcting the moduli obtained from different methods of applying confining pressure.

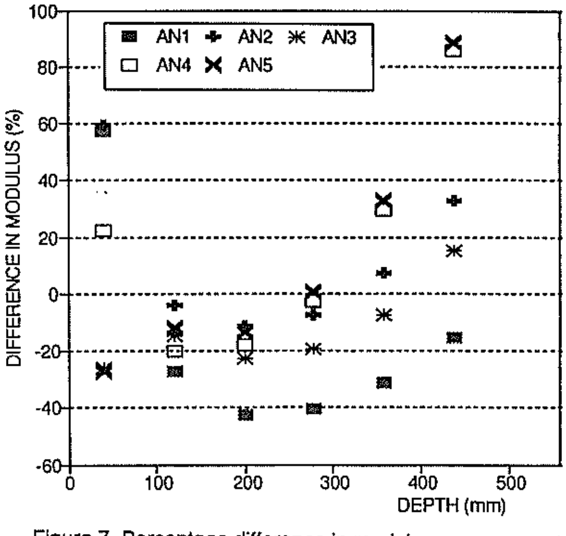

Figure 7 Percentage difference in modulus as compared with NAASRA estimates

7. COMPARISON OF CRITICAL STRAINS AND PAVEMENT LIVES

Pavement life is defined in the NAASRA Guide as a function of the maximum vertical compressive strain on the top of the subgrade (see Equation 1). The vertical strains at the top of the subgrade estimated by the five analyses are tabulated in Table II. It should be noted that the critical maximum vertical strains can occur at either the vertical axis through the load centre or the vertical axis through the centre of the loading group.

The pavement lives estimated by the five analyses are given in Table III. They vary from 0.3 to 3 times the NAASRA design lives and this indicates that the risks associated with the use of different characterisation models in pavement analysis and design are significant.

The results also indicate that the analyses AN1 and AN3 produced more conservative predictions than other analyses; whereas the analysis AN2 could involve higher risk by producing high design lives for all pavements. The analyses AN4 and AN5 produced less conservative predictions, except for pavements with very thick basecourse layers (Eg. pavements 2 and 3).

8. SUMMARY AND CONCLUSIONS

Resilient modulus testing of the typical base crushed rock were carried out using two repeated load triaxial test types (i.e. with constant and cyclic confining pressures) and the resilient moduli were derived and fitted to available models for resilient modulus. It was found that the constant (K_2) of the 'modulus-peak mean stress' model (Model 1) is different for different test types and this indicates that the effects of static confining pressure on the model constant may be significant.

It was also found that the 'modulus-average mean stress' model (Model 3) produced the same constants (K_3) for both test types (providing that stress ratios in the test with cyclic confining pressure are greater than 6). This model can provide a basis for correcting the moduli obtained from different methods of applying confining pressure.

K_2 and K_3 were also expressed as a function of stress ratio in order to fit more closely the test results obtained from tests with cyclic confining stress (Model 2 and Model 4).

The characterisation models were used in the program NONCIRL to calculate sub-layer moduli for their stress dependent characteristics and to predict pavement lives for nine typical granular pavements. It was found that besides the effect of different characterisation models determined from the repeated load triaxial tests, pavement analysis should take into account the effect of no-tension in the base layers and the effect of earth pressure at rest.

For the pavements studied, the pavement lives calculated with NONCIRL for different characterisation models ranged from 0.3 to 3 times the NAASRA design lives. This indicates that the risks associated with the use of different characterisation models in pavement analysis and design are significant.

Although the results in this study supported the use of the 'modulus-average mean stress' models (Model 3 and Model 4) for reducing the difference in the predicted pavement lives, the differences between the two models are still very high for some pavement configurations. In addition, this study does not take into account experimental errors involved in the tests. Therefore, there is scope for further studies to identify the risks involved in applying the repeated load triaxial tests into practice and to minimise them, particularly in the area of evaluating the performance of new and marginal materials.

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TABLE II

VERTICAL STRAINS ON TOP OF SUBGRADE CALCULATED WITH NONCIRL

VERTICAL STRAIN CALCULATED AT TOP OF THE SUBGRADE (microstrain)										
Pav. No.		1	2	3	4	5	6	7	8	9
Depth (mm)		400	520	640	300	390	480	200	260	320
AN1	R=0mm	2000	1351	964	1805	1284	946	1401	1100	873
	R=165m	1801	1221	887	1707	1155	846	1720	1136	820
AN2	R=0mm	1600	1123	818	1557	1147	861	1299	1041	836
	R=165m	1455	1020	755	1485	1037	773	1592	1080	788
AN3	R=0mm	1895	1304	940	1816	1316	979	1465	1164	931
	R=165m	1722	1179	863	1760	1192	875	1901	1246	891
AN4	R=0mm	1619	1088	764	1638	1182	870	1376	1100	878
	R=165m	1471	987	704	1570	1069	780	1728	1155	832
AN5	R=0mm	1751	1156	807	1763	1252	916	1461	1153	918
	R=165m	1595	1047	742	1711	1136	820	1901	1237	880

Grey shade = maximum critical strain

TABLE III

PAVEMENT LIVES CALCULATED WITH DIFFERENT CHARACTERISATION MODELS

RATIO OF 'CALCULATED LIFE/NAASRA ESTIMATED LIFE'									
Pav. No.	1	2	3	4	5	6	7	8	9
NAASRA estimate (10,000 ESAs)	10	100	1000	10	100	1000	10	100	1000
AN1	0.31	0.51	0.57	0.64	0.73	0.65	0.91	1.76	1.15
AN2	1.52	1.91	1.83	1.85	1.64	1.27	1.58	2.52	1.58
AN3	0.45	0.66	0.68	0.62	0.61	0.51	0.44	0.91	0.73
AN4	1.40	2.39	2.97	1.29	1.32	1.18	0.88	1.56	1.11
AN5	0.80	1.55	2.01	0.76	0.88	0.82	0.44	0.96	0.80

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