MECHANICAL BEHAVIOUR OF HYDRATED CEMENT TREATED CRUSHED ROCK BASE (HCTCRB) UNDER REPEATED CYCLIC LOADS

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ABSTRACT

This paper aims to report the mechanical behaviour of hydrated cement treated crushed rock base (HCTCRB) as granular road base material subjected to repeated cyclic loads from Repeated Loads Triaxial (RLT) tests with various stress paths in order to improve more understanding of such Western Australian roads based materials on mechanistic-empirical pavement design and analysis. As known, pavement surface rutting, longitudinal and alligator cracks are normally the main cause of damage in flexible pavements. Factors contributing to such damage are the excessive irreversible and reversible deformation of a base layer including the behaviour of a mechanical response of unbound granular materials (UGMs) under traffic load is not well understood. In this study, the shakedown concept was utilized to describe and determine limited use of HCTCRB subjected to different stress conditions. The concept is the theoretical approach of the UGMs used to describe the behaviour under RLT tests. The shakedown concept utilizes macro-mechanical observations of the UGM's response and the distribution of the vertical plastic strain in the tested material. While the shakedown limit of an UGM is known, whether the limitation of the accumulated plastic strain in an unbound granular layer causing rutting in pavements can be predictable. In this paper, compacted HCTCRB samples were subjected to the various stress condition defined by the stress ratio (the ratio of a vertical major stress, σ_1 and a horizontal minor stress, σ_3) in order to simulate the real condition of pavement. The study reports that HCTCRB was defined the working stress ratio of 11 in pavement structure and will be achieved stable state at the large number of load cycles. Moreover, the mechanical responses were investigated and the limit ranges of using HCTCRB in pavements were determined.

1 INTRODUCTION

The UGM layer with thin bituminous surfacing is widely used in the Australian road network. Normally, CRB and HCTCRB are used as base course material in Western Australia, can be determined as UGMs. The important function of the base course in pavements is to distribute and reduce amount of compressive stresses and strains because of vehicle wheel loads through the subbase and the subgrade without unacceptable strain. Consequently, an obvious understanding of shear strength, resilient and permanent strain, and shake down limit characteristics of materials relevant to pavement mechanistic design is very important to obtain the effective uses of such materials. However, Western Australia pavement design still relies on a traditional design procedure which is unreliable enough to explain a relationship between design parameter inputs and pavement performances. Roads need to be investigated to improve pavement analysis and design more precisely than in the past with respect of real behaviour and the amount of traffic during the service life. Consequently, a most economical of layer thickness and an appropriate material type for the pavement will be determined.

This paper focuses on applying the mechanical behaviour of base course materials and developing the typical models of HCTCRB for pavement analysis in Western Australia. The empirical design method is unacceptable because the test protocols to require the design parameter inputs from monotonic loading tests rather than cyclic loading tests which are more representative of real traffic loading conditions. A mechanistic design attempts to explain pavement characteristics under real pavement conditions such as load types, material properties of the structure and environments based on design parameters from sophisticated tests which can simulate real pavement conditions into the test protocol (Collins *et al.*, 1993). The main success of this analytical method is the experimental measurement and appropriate characterization of the mechanical responses from the RLT test which is the basic protocol of this study.

2 BACKGROUND

The empirical nature of traditional pavement design procedure is based on experience and the results of simple tests such as the California Bearing Ratio (CBR), particle size distribution (PSD), moisture sensitivity, Los

Angeles (LA) abrasion, shear strength and deflection. Such testing results are all static parameters and simple index parameters rather than any consideration of multidimensional geometry, realistic material performance and displacement distribution during cyclic loading, stresses and strain distribution in multilayered pavement design. Consequently, the use of empirical approaches becomes sub-standard. Traditional design procedure has been criticized by Wolff, who argued that it is too simplistic and does not take into account the non-linear behaviour of UGMs (Wolff and Visser, 1994).

The performance of a base course material depends upon its stiffness and deformation resulting from a traffic load. A large deformation causes rutting on the bituminous surface. Basically, the conventional pavement construction is designed to provide adequate thickness cover the sub layer in such a way that no shear failures and unacceptable permanent deformation takes place in each layer. For pavement design purposes, the stress level which is related with a reversible strain response must be determined and consequently not exceeded, once unacceptable permanent strains are prevented. This has improved the possibility of a critical boundary stress between stable and unstable conditions in a pavement.

The shakedown concept has been used to explain the behaviour of conventional engineering structures under repeated cyclic loading. Basically, it was originally developed to analyse the behaviour of pressure vessels subjected to cyclic thermal loading. Subsequently, it was improved to analyse the behaviour of metal surfaces under repeated rolling or sliding loads. For the theoretical approach of the UGMs' permanent deformation used to describe the behaviour of tested materials under RLT tests under macro-mechanical observations of the material response and in the distribution of the plastic strain in the tested material were investigated. They can predict progressive accumulations of plastic strains under repeated loading and whether the amount of the applied loads exceeds a certain limited-value called shakedown limit or limit load (SAMARIS, 2004).

Firstly, the possible employment of the shakedown concept in pavement design was introduced by Sharp and Booker (1984) and Sharp (1983). They explained the application of the shakedown concept based on the tested results of the AASHTO road tests (AASHTO, 1986) where in some cases, deterioration was reported due to stiffening or post-compaction after a number of load cycles (Kent, 1962). Moreover, studies have been produced to define upper-bound (Collins and Boulbibane, 1998) and lower-bound (Yu and Hossain, 1998) for the shakedown limit of UGMs in simple pavement structures. A low stress levels, the mechanism of permanent strain has an initial post compaction or re-arrange phase, while the permanent strain rate is relatively high but this is reduced with increasing numbers of load cycles. A stable state may be maintained for a period of time unless the states change. Maree reported the behaviour of gravel and crushed stone and that under constant confining stress, the specimens stabilized under a certain threshold of repeated deviator stress and developed a design procedure, based on a failure model (Maree *et al.*, 1982). Numerous investigations have been conducted regarding the behaviour of UGMs used in flexible pavements. Lekarp summarized the main findings regarding the effects of different material parameters on the permanent strain response of UGMs and the maximum applied stress in UGM layers is within the maximum repeated deviator stress limit (Lekarp *et al.*, 2000). The original shakedown concept maintains that there are three ranges of permanent strain response under repeated loading.

- Plastic shakedown range (Range A). The low loading levels apply and the material response indicates plastic in a few initial cycles, although the ultimate response is elastic after Post-compaction. The strain is completely reversible and does not lead any permanent strains when it reaches a state of stability.
- Plastic creep range (Range B). The applied loading level is low enough to avoid a quick collapse. The material achieves a long-term stable state response with any accumulation of plastic strain (Post-compaction). However the material will show failure with a large number of load cycles after a stable state.
- Incremental collapse range (Range C). The repeated loading is relatively large so that plastic strain accumulates rapidly with failure occurring in a small number of load cycles after stiffening.

A pavement is likely to show progressive accumulation of permanent strains (rutting) under repeated traffic loading if the magnitude of the applied loads exceeds the limiting value (Range C). If the applied traffic loads are lower than this limit, after any post-compaction stabilization, the permanent strains will level off and the pavement will achieve a stable state of "shakedown" (Ranges A and B) presenting only reversible strain under additional traffic loading (Sharp, 1985). This implies an adaptation by the pavement subjected to the working load. This could be due to a change in material response (compaction degree), due to a change in stress state or due to a combination of both effects. With this understanding of material behaviour, the shakedown concept typically then determine the load carrying capacity of the structure if it is not to reach excessive permanent strain. For performance prediction, it is of great importance to know whether a given pavement will experience progressive accumulation of permanent strain leading to state of incremental collapse or if the increase in permanent strain will cease, resulting in a stable response (shakedown state).

2.1 PERMANENT STRAIN UNDER A NUMBER OF LOAD CYCLES MODELS

In the considering, the long-term behaviour model of pavements, it is essential to take into account the accumulation of permanent strain with the number of load cycles and stress levels that play and important role. Hence the main research purpose focusing on long-term behaviour should be to establish a constitutive model which predicts the amount of permanent strain at any number of cycles at a given stress ratio. In the past, permanent strain of UGMs for pavement applications has been modelled in several ways. Some of these are logarithmic with respect to the number of loading cycles (Barksdale, 1972, Sweere, 1990) whilst others are hyperbolic, tending towards an asymptotic value of deformation with increasing numbers of load cycles (Wolff and Visser, 1994, Paute *et al.*, 1996). The first type is that due to this approach, the permanent axial strain is supposed to accumulate in linear relation to the logarithms (Barksdale, 1972) as in Equation **(1):**

$$\varepsilon^{p} = a + b\,log(N) \tag{1}$$

Where ε^{p} is permanent strain; a and b are regression constants; and N is the number of loading cycles. The long-term strain behaviour was also investigated by Sweere in a series of RLT tests and suggested that for a large number of load cycles the following approach should be employed:

$$\varepsilon^{p} = A \cdot N^{B} \tag{2}$$

where:

ε^{p}	[10^{-3}] % permanent strain
A, B	[-] regression parameters
N	[-] number of load cycles.

To implement the RLT measured permanent strain development in the computation of permanent strain development in a pavement structure, the permanent strain in the material under consideration has to be known as a function of both the number of load cycles and the stresses in the materials. Furthermore the shakedown approach should be considered. Lekarp and Dawson (Lekarp and Dawson, 1998) suggested that the shakedown approach might also be employed in explaining the permanent strain behaviour of UGM. In conclusion, they pointed out that more research is required to determine this shakedown limit. However, for finite element (FE) calculations of UGM layer as part of a FE based pavement design, the prerequisite is a stress and load cycles dependent model for the permanent strain behaviour of UGM. Some stress dependent models are available (Barksdale, 1972) but two the Theyse-Model and the Huurman-Model, use the shakedown approach, in particular modelling the stable and unstable permanent strain behaviour to model the permanent strain behaviour of UGM as a function of the number of load cycles. Similar to what Sweere (1990) found for his laboratory test results, the log-log approach was also used by Huurman (Huurman, 1997) to describe the permanent strain development in UGM layer in pavements under traffic by Equation **(3).** Huurman used a RLT apparatus to determine the permanent strain behaviour.

$$\varepsilon^{p} = A \cdot \left[\frac{N}{1000}\right]^{B} + C\left[e^{D\frac{N}{100}} - 1\right] \tag{3}$$

where:

ε^{p}	[%] permanent strain
e	[-] base of the natural logarithm (= 2.17828....)
N	[-] number of load cycles.

The first term of the model describes a linear increase of permanent strain with N on a log (ε^{p}) - log (N) scales. The parameter A gives the ε^{p} at 1,000 load cycles and B gives the subsequent slope of ε^{p} with the rising number of load cycles. In the case of stable behaviour, the model parameters C and D are equal to zero. It is clear that the unstable behaviour at high stress levels can not be described by the first term alone because an exponential rather than linear increase of ε^{p} with N on the same log (ε^{p}) - log (N) scales is observed. To implement the RLT, measured permanent strain development in the computation of permanent strain in a pavement structure, the permanent strains in the materials have to be determined as a function of the applied stresses and the number of load cycles. However, the determination of the parameters A, B, C and D for the model proposed by Huurman depends on σ_1 = major principal stress and $\sigma_{1,f}$ = major principal stress at failure. In this investigation it was found from tests that $\sigma_{1,f}$ could not be obtained for the crushed UGMs, as already explained. In this research the plastic Dresden-Model as developed at Dresden University of Technology has been used to determine the parameters A and B as a function of the principal stresses σ_1 and σ_3 for the Range A (Equations **(4** and **(5)** and for the Range B (Equations **(6** and **(7)**:

$$A = (a_1.e^{a_2.\sigma_3})\sigma_1^2 + (a_{3.}.\sigma_3^{a_4})\sigma_1 \tag{4}$$

$$B = (b_1.e^{b_2.\sigma_3})\sigma_1 + (b_{3.}.\sigma_3^{b_4}) \tag{5}$$

$$A = (a_1.\sigma_3^{a_2})\left(\frac{\sigma_1}{\sigma_3}\right)^2 + (a_{3.}.\sigma_3^{a_4})\frac{\sigma_1}{\sigma_3} \tag{6}$$

$$B = (b_1.\sigma_3^{b_2})\left(\frac{\sigma_1}{\sigma_3}\right) + (b_{3.}.\sigma_3^{b_4}) \tag{7}$$

where:

σ_3 [kPa] minor principal stress (absolute value)
σ_1 [kPa] major principal stress (absolute value)
a2, a3, b2, b3 model parameters
a1, a4, b1, b4 model parameters.

As already mentioned, the behaviour observed for the higher stress level cannot be described by means of Equations **(4)-(7).** A second term was therefore, added to Equation **(3).** The model parameters C and D are again stress dependent. By analysing the test results, the parameters C and D become available to recognize collapse. In this research, the parameters A, B, C and D were also determined. Finally, it was realized that it is possible to model the permanent deformation behaviour of UGMs in a stress dependent way. However, it is necessary to model each behaviour range separately.

3 MATERIALS

3.1 CRUSHED ROCK

The crushed rock samples used in this study were taken from a local stockpile of Gosnells Quarry and kept in sealed containers. RLT tests were performed on samples as part of the collaboration with Civil Engineering, Curtin University of Technology. The crushed rock samples were prepared (see Figure1 for the grading curve) at 100% of maximum dry density (MDD) of 2.27 ton/m^3 and optimum moisture content (OMC) of that 5.5%. Material properties achieve base course specifications (Main Roads Western Australia, 2003). Figure 1, shows the grading curve of the crushed rock in this study achieves the upper and lower bound of the base course specifications. Significant comparisons of basis properties with specifications were made as shown in Table 1.

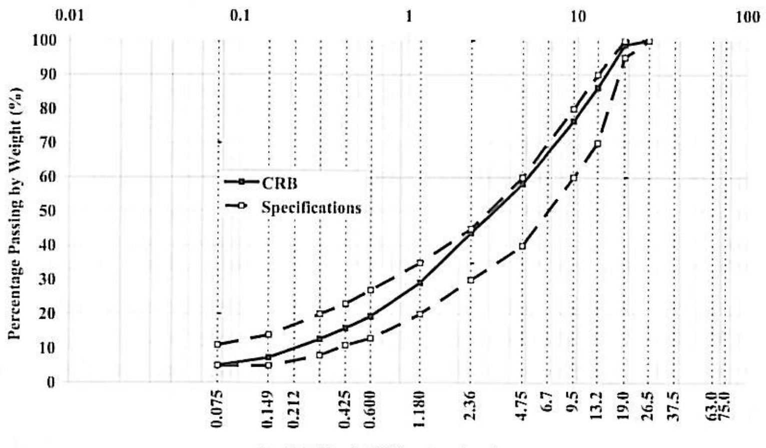

Figure 1 : CRB grading curves compared with WA Main Roads specifications (Jitsangiam and Nikraz, 2007).

Table 1: Characterization tests (Main Roads Western Australia, 2007b).

Tests*	Results	Tests*	Results
Liquid Limit (LL)	22.4%	Coefficient of uniformity (Cu)	22.4
Plastic Limit (PL)	17.6%	Coefficient of curvature (Cc)	1.4
Plastic Index (PI)	4.8%	% fines	5 %
Linear Shrinkage (LS)	1.5%	Cohesion of CRB (C**)	32 kPa
Flakiness Index (FI)	22.5%	Internal friction angle of CRB (ϕ**)	59°
Maximum dry density (MDD)	2.27 t/m^3	Max. Dry Compressive Strength (MDCS)	3,528 kPa
Optimum moisture content (OMC)	5.5%	California Bearing Ratio (CBR)	180

* Accordance with MRWA (Main Roads Western Australia, 2006).

** Drained triaxial compression tests at the100%OMC condition.

3.2 HYDRATED CEMENT TREATED CRUSHED ROCK BASE (HCTCRB)

Hydrated cement treated crushed rock base (HCTCRB) is manufactured by blending 2 % cement with a standard dry weight crushed rock base (Main Roads Western Australia, 2003). It is mixed and stockpiled in the range of -1.0% to +2.0% of the optimum moisture content of the untreated crushed rock base as obtained by MRWA Test Method WA 133.1 (Main Roads Western Australia, 2007a) during the initial hydration 7-day period. MDD and OMC of HCTCRB change to 2.12 t/m3 and 8% and this figure indicates that after cement hydration occurs, the impact of cement on soil compaction was increased optimum moisture content (Zhongjie and Mingjiang, 2008).

3.3 CEMENT

The cement used in this study was the bagged cement product of Cockburn Cement (Cockburn Cement, 2006) of General Purpose Portland Cement -type GP following the standard of AS 3972-1977.

4 LABORATORY PROGRAM AND TESTING

4.1 SPECIMEN PREPARATION

Sample preparation was carried out using a standard cylinder mould 100 mm in diameter and 200 mm in height by the modified compaction method (Main Roads Western Australia, 2007a). Compaction was accomplished on 8 layers with 25 blows of a 4.9 kg rammer at a 450 mm drop height each layer. Each layer was scarified to a depth of 6 mm before the next layer was compacted to ensure full bonding between the layers. After compaction the basic properties of each specimen were determined after which it was carefully carried to the base platen set of the chamber triaxial cell. A crosshead and stone disc were placed on the specimen and it was wrapped in two platens by a rubber membrane and finally sealed with o-rings at both ends.

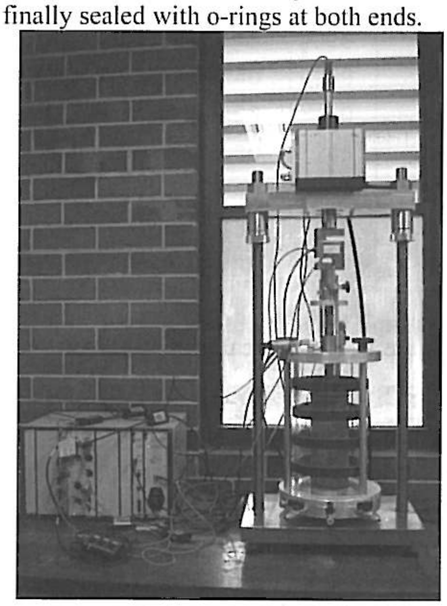

Figure 2: The repeated loads triaxial (RLT) apparatus.

4.2

4.3 REPEATED CYCLIC LOAD TRIAXIAL (RLT) TESTS

The tests were carried out with a cyclic triaxial apparatus consisting of main set containing the load actuator and a removable chamber cell. The specimens were placed in the triaxial cell between the base platen and crosshead of the testing machine (Figure 2). Controls were used to manage the chamber and the air pressure. The analogical signals detected by the transducers and load cell are received by a module where they are transformed to digital signals. A computer converts modules of the digital signals sent from the system. The system is located in the main set and facilitates the transmission of the orders to the actuator controller. User and the triaxial apparatus communication is controlled by a computer which uses convenient and precise software. This makes it possible to select the type of test to be performed as well as all the parameters, stress levels and data to be stored. The load cell, the confining pressure and the externally linear variable differential transducer (LVDT) on the top of the triaxial cell are used to measure deformations over the entire length of the specimens. The control and data acquisition system (CDAS) provided the control signals, signal conditioning, data acquisition. The CDAS was networked with the computer which provided the interfacing with the testing software and stored the raw test data. These enabled the resultant stress and strain in the sample to be determined.

This apparatus is limited to laboratory samples with a maximum diameter of 100 mm and a height of 200 mm based on the standard method of Austroads APRG 00/33-2000 (Voung and Brimble, 2000). Moreover, the apparatus allows the laboratory sample to be subject to cyclic axial deviator stresses but it is not feasible to vary the confining radial stresses at the same time. Confining pressure was generated using air to simulate the lateral pressure acting on the surrounding materials as occurs in a pavement layer. The pressure was applied and stresses were found at different points in the granular material. The results were expressed in terms of deviator stress q= σ_1 -σ_3, mean normal stress p= (σ_1 -2σ_3)/3 and the confining pressure was simulated from the thickness of pavement base course layer that common use in Western Australia. For this reason, it was decided to subject the laboratory samples to 11 different stress levels and the selected confining pressure was 40 kpa. After the confining pressure had been applied additional dynamic vertical stress was applied. Triaxial tests were carried out with axial stress pulses reaching stress ratios of σ_1/σ_3 = 5-26. The dynamic axial stress came from a high pressure air actuator capable of accurately applying a stress pulse following the stress level. In this test, there was haversine waveform frequency of 1 Hz over a period of 1.0 sec and a load pulse of 0.1 sec duration, as illustrated in Figure 3.

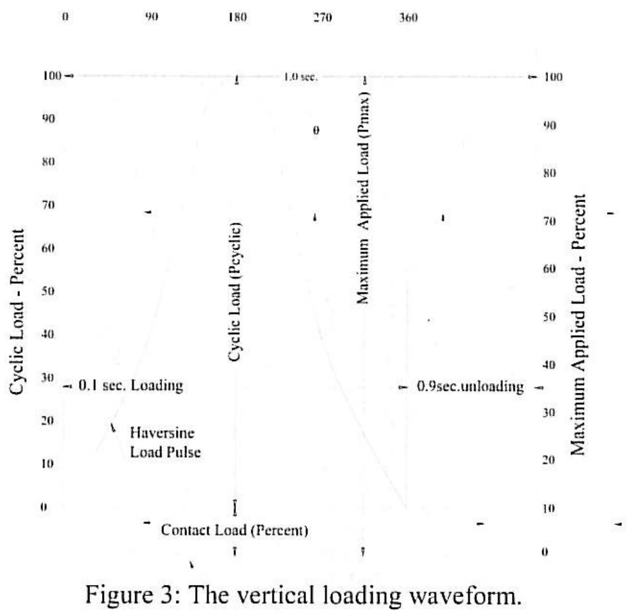

Figure 3: The vertical loading waveform.

4.4 RESILIENT MODULUS TESTS AND PERMANENT DEFORMATION TESTS

The standard method of Austroads APRG 00/33-2000 (Voung and Brimble, 2000) for RLT Test Method was followed for the resilient modulus tests and the permanent deformation tests. The UTM-14P digital servo control testing machine in the Geomechanics Laboratory, Department of Civil Engineering, Curtin University of Technology was used.

New specimens were prepared as described in the previous section. Permanent deformation testing was performed during which the specimens were loaded with three stress stages at the ratios of the dynamic deviator stress (σ_d) with frequency of 0.33 Hz to the static confining stress (σ_3) based on Austroads APRG 00/33-2000 (Voung and Brimble, 2000), each involving 10,000 cycles for each particular stress condition as shown in Table

2. After permanent deformation tests, in accordance with this Standard (Voung and Brimble, 2000), the same specimens were applied sequentially by the difference of the 65 stress stages straightaway to conduct the resilient modulus test to check the elastic condition of each specimen throughout the multiple loading stress stages. This process simulates complicated traffic loading acting on pavement. Two hundred loading cycles of each stress stage were applied to the specimens. Table 3 shows the stress levels for the resilient modulus.

Table 2: The permanent deformation Austroad-APRG 00/33 (Austroads, 2004).

Permanent Deformation Stress Levels		
Stress	Base	
Stage Number	Confining pressure σ_3 (kPa)	Dynamic deviator stresses σ_d (kPa)
1	50	350
2	50	450
3	50	550

Table 3: The resilient modulus Austroad-APRG 00/33 (Austroads, 2004).

Resilient Modulus Stress Levels								
Stress Stage Number	σ_3 (kPa)	σ_d (kPa)	Stress Stage Number	σ_3 (kPa)	σ_d (kPa)	Stress Stage Number	σ_3 (kPa)	σ_d (kPa)
0	50	100	22	30	150	44	20	185
1	75	150	23	40	200	45	30	275
2	100	200	24	50	250	46	40	370
3	125	250	25	75	375	47	50	450
4	150	300	26	100	500	48	30	275
5	100	200	27	50	250	49	20	225
6	50	150	28	30	180	50	30	335
7	75	225	29	50	300	51	40	450
8	100	300	30	75	450	52	50	550
9	125	375	31	50	300	53	20	250
10	150	450	32	30	180	54	30	375
11	75	225	33	40	250	55	40	500
12	40	125	34	30	210	56	20	300
13	30	100	35	40	280	57	30	450
14	40	150	36	50	350	58	40	600
15	50	200	37	75	525	59	30	500
16	75	300	38	40	280	60	20	350
17	100	400	39	20	150	61	30	550
18	125	500	40	30	245	62	20	375
19	75	300	41	40	325	63	30	575
20	30	125	42	50	400	64	20	400
21	20	100	43	30	245	65	20	500

. Table 4 : Coefficients for Range A and B of model Equation (4), (5), (6) and (7).

Type	HCTCRB	
parameter	Range A	Range B
a_1 [-]	0.00185	0.00011
a_2 [kpa^{-1}]	-0.11000	1.45000
a_3 [kpa^{-1}]	0.0000013	-0.00040
a_4 [-]	1.8500	1.4450
b_1 [-]	0.00030	0.00350
b_2 [kpa^{-1}]	-0.01200	-0.08550
b_3 [kpa^{-1}]	0.00200	0.00010
b_4 [-]	0.12000	0.55000

5 RESULTS AND DISCUSSION

5.1 RESILIENT MODULUS TESTS AND PERMANENT DEFORMATION TESTS

The resilient modulus determined from the RLT test is defined as the ratio of the repeated deviator stress to the recoverable or resilient axial strain:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{8}$$

Where M_r is the resilient modulus, σ_d is the repeated deviator stress (cyclic stress in excess of confining pressure), and ε_r is the recoverable strain in a vertical direction. Based on the specification, the results of HCTCRB in the condition of 100% MDD at 100% OMC are represented to show its characteristics and to determine suitable mathematical models of resilient modulus and permanent deformation of HCTCRB.

Figure 4 shows the results of the resilient modulus tests which are plotted versus the bulk stress ($\sigma_1+\sigma_2+\sigma_3$). Generally, they are non-linear with respect to the magnitude of applied stresses. Figure 4 also shows that the resilient modulus of HCTCRB can be modelled reasonably by using The K-Theta (K-θ) model (Hick and Monosmith, 1971). The representative K-θ model of HCTCRB is exhibited in equation (**9**).

$$M_r = k_1 \cdot \theta^{k_2} = 8.91020^{0.6817} \tag{9}$$

Where M_r is resilient modulus in MPa; θ is bulk stress ($\sigma_1+\sigma_2+\sigma_3$) where ($\sigma_2=\sigma_3$); σ_1 is major principal stress (vertical axial stress); σ_3 is minor principal stress (confining stress); k_1 and k_2 are regression coefficients as shown in Figure 4.

Figure 5 show the typical results of the permanent deformation tests in terms of the relationship between permanent deformation and loading cycles for HCTCRB, to exhibit the comparison of the measured and permanent deformation values and the predicted values for proposed permanent deformation models. Figure 5 also indicates that the permanent deformation can be modelled quite reasonably by using the model suggested by Sweere, G.T.H from SAMARIS (SAMARIS, 2004). Sweere suggested that for the long-term deformation behaviour of unbound granular materials (UGMs) under a large number of load cycles an approach as shown in equation (**10**) should be employed as the proposed permanent deformation model of HCTCRB.

$$\varepsilon^p = A \cdot N^B = 0.0231 \cdot N^{0.1841} \tag{10}$$

Where ε^p is permanent deformation in Millimeters; A and B are regression constants and N is the number of loading cycles.

5.2 SHAKEDOWN BEHAVIOUR

The permanent deformation accumulations were observed as shown in Figure 6. As the test results present HCTCRB response always produce permanent deformation during cyclic loading no purely elastic behaviour under repeated cyclic loads in basecourse materials was identified (Werkmeister *et al.*, 2001) and the multi-layer linear elastic theory is not enough to analyse the UGM layer. Permanent deformation is on the basis of internal friction between grains, particle shape, compaction, consolidation, distortion, etc and test results can be separated into three ranges (Ranges A, B and C) based on the shakedown concept.

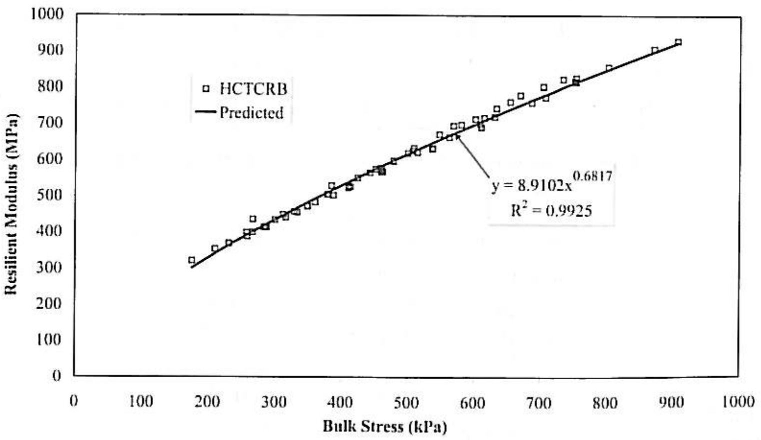

Figure 4: The resilient modulus predictions.

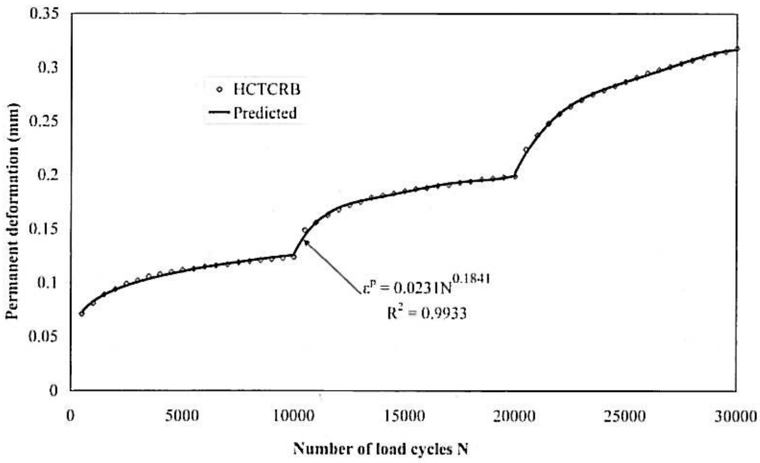

Figure 5: The HCTCRB permanent deformation predictions.

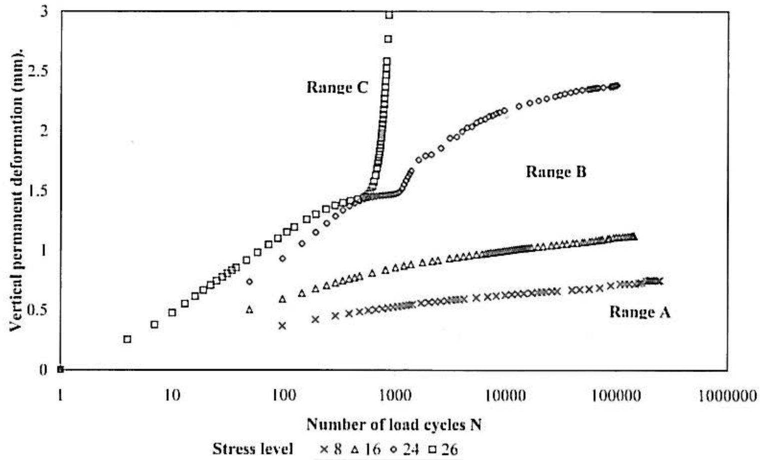

Figure 6: HCTCRB permanent deformation versus number of load cycles (N).

5.2.1 Range A – Plastic Shakedown Range

The lower lines (Stress levels 8-16) indicate the response of Range A. The behaviour is entirely plastic for a number of cyclic load cycles although when it reaches a stable state after the post-compaction period, the response becomes completely resilient and no further vertical permanent displacement occurs. Figure 6 shows Range A of HCTCRB at stress levels 8-16 and the HCTCRB working state of stress level 11 achieved in Range A. Figure 8 indicates that the vertical permanent strain rate decreases rapidly until it reaches a state of equilibrium. Figure 7 compares the measured strain with the strain model and model coefficients as shown in Table 4. For this range of material the response amount of vertical displacement accumulation depends upon the stress level. Observation of each stress level shows the number of cycles required before a stable state is achieved. UGMs behaviour at these stress levels would become stable after post-compaction under service load. If the Range A of the shakedown behaviour is allowed in the pavement an adequately small accumulated displacement, as acceptable permanent deformation, would be seen in a basecourse layer and this would terminate after a set number of load cycles. The material does not reach failure.

5.2.2 Range B - Plastic Creep

Figure 8 shows an intermediate response of Range B (Stress levels 16-24 for HCTCRB). At the beginning of the load cycles the level of permanent strain rate decreases rapidly but is less than Range A at the same time at a lower rate. The number of load cycles may define the end of post-compaction. A slow increase of the permanent strain rate occurred after 80,000 load cycles. Test results showed that, although the deformation is not completely resilient, permanent deformation is acceptable for the first period of the cycles. In Figure 7, the vertical permanent strains are compared with the strain model and model coefficients as shown in Table 4. A great number of failures could occur if the condition does not change and if it is maintained long enough, it will deteriorate at the end as in Range C.

5.2.3 Range C – Incremental Collapse

Figure 8 shows the Range C behaviour (stress level 26 for HCTCRB). The permanent strain rate decreases during the first period of load cycles and then becomes lower, nearly constant. Failure occurs with a relatively small number of load cycles when the cumulative permanent strain rate increases very rapidly after which the strain rate does not decrease again. UGMs do not reach a stable state. Range C behaviour in UGMs would result in the failure of the pavement by shear deformation in the base layer shown as rutting at the road pavement surface. This range should not develop in a pavement designed to the Standard.

In Figure 8 the behaviour of Ranges A, B and C is completely different. The different vertical strain responses under the number of cycles with no cessation of the strain accumulation responses in Range C has to be separated from Ranges A and B. These can also be distinguished on the basis of plastic strain rate behaviour. With Range A the permanent strain rate decreases rapidly and does not reach the constant level throughout the duration of testing. In Figure 9 the vertical permanent strain of range C is compared with the strain model.

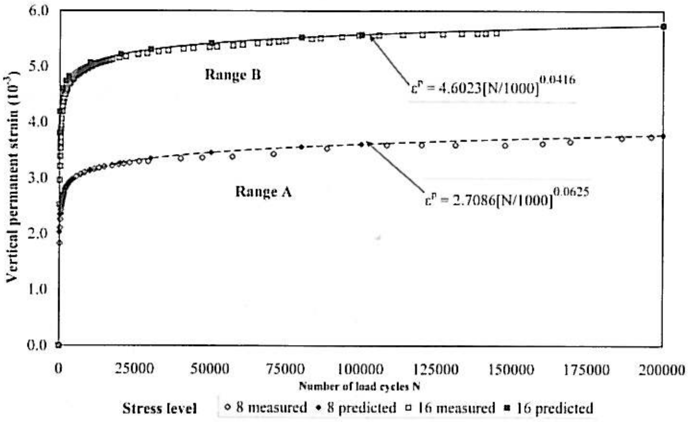

Figure 7: HCTCRB Ranges A and B vertical permanent strain compared with the strain model.

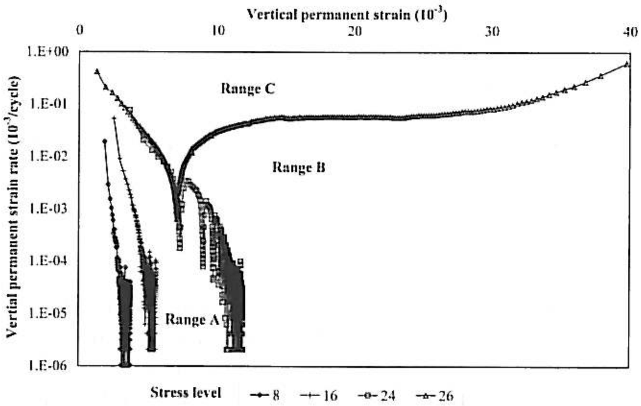

Figure 8: HCTCRB permanent strain rate versus vertical permanent strain.

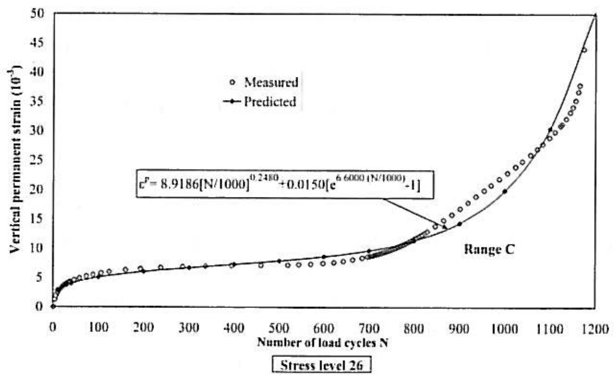

Figure 9: HCTCRB Range C vertical permanent strain compared with the strain model.

6 CONCLUSIONS AND DISCUSSION

The mechanical behaviour of HCTCRB, normally used as a basecourse material in Western Australia, was investigated by RLT tests. The tests were carried out in terms of the resilient modulus test and the permanent deformation test to obtain an understanding of the resilient and permanent deformation characteristics of this material under simulated conditions of real traffic loading. The resilient modulus characteristics could be modelled using the K-θ model (Hick and Monosmith, 1971). The long-term permanent deformation characteristics could be modelled by using Sweere's model (Sweere, 1990).

It has been shown that the use of the shakedown concept application to UGMs in the pavement analysis is possible. The limit ranges defined in this study, Ranges A, B, and C, occur in HCTCRB. HCTCRB under fixed stress level conditions shows a relationship between the permanent strain and stress level. When a cyclic loading is applied the sample responds by changing its permanent strain. In a continuous and gradual increase of the loading amplitude $\Delta\sigma$, the material will start by trying to change its mechanical behaviour. The possibility of a purely elastic approach in pavement analysis has been discarded as no purely elastic response is found in HCTCRB during repeated cyclic loading.

For low stress ratios HCTCRB reacts corresponding to Range A under stress level 16. After a few cycles the particles will reach a stable state because some energy will have been dissipated due to viscosity. At this range of repeated loadings the dissipated energy is independent of the loading and does not change from one cycle to another. The pavement will reach a shakedown limit after post-compaction deformation with no development of further permanent deformation. The vertical strain rate rapidly increases and the material subsequently responds elastically. Hence Range A of HCTCRB is accepted in pavement construction if the accumulated strain is sufficiently small before the development of fully resilient behaviour.

The next step is to examine the application of material in the pavement that responds to Range B. For higher loadings at stress levels 16-24 of HCTCRB, the energy input is first quickly dissipated by a rearrangement of the sliding internal contacts of material, the so-called post-compaction. The dissipated energy per cycle relaxes then to a stationary value so that the vertical strain rate decreases to a constant rate depending on the loading, but also on the characteristics of the grains such as the friction or the stiffness of the contacts. A closer investigation of the size dependence of this phenomenon would help to identify if the material is evolving on a much longer time scale to a final shakedown state in which all the energy supplied to the system is dissipated. This process may take a longer time in the simulation than in the real experiment where more dissipative mechanisms exist.

It seems that the material in Range B does not shakedown, rather it will fail at a very high number of load repetitions. It is important to know the acceptable maximum number of load cycles that will prevent distress in the pavement from occurring. Further tests with load applications up to 2,000,000 load cycles may be necessary to find the point of failure. For many low-traffic road pavements where the total number of vehicles carried will be small and maintenance ultimately required to correct inadequacies other than traffic-induced rutting, Range B behaviour will probably be acceptable.

Range C behaviour at stress level 26 for HCTCRB, should not be allowed to occur in the pavement. If the stress levels imposed are high enough, there is no possibility for material to re-arrange to the new state and post-compaction leads to an incremental collapse. Material is not able to dissipate enough energy without changing its configuration so it needs to modify its shape.

The Range A limit (plastic shakedown limit) can be used to predict whether or not a stable state occurs in the UGM layer of the road structure. The plastic shakedown limit of HCTCRB should be used in the Western Australian pavement design guidelines, It can be shown that the maximum stresses occurring in the pavement UGM are within Range A. Based on pavement design guidelines the approximate working stress of Western Australian road was level 11 at the base layer indicating that HCTCRB achieves Range A behaviour and will reach a stable state with this amount of traffic without rutting failure. The new approach has been partially validated by the data from which those guidelines should be derived. It has been shown that the permanent strain characteristics of HCTCRB could be modelled using the Dresden-model, with each behaviour range tested separately.

The paper demonstrates that having defined the ranges from the laboratory results it is possible to determine whether HCTCRB is adequate or whether other thicknesses of surfacing layer are required to implement satisfactory pavement performance. However, suitable experience is currently unavailable to confirm the reliability of this proposed relationship between the ranges defined by the RLT tests and real performance. The pavement mechanical response is affected by several parameters. Further investigations on this topic are required to verify results made by the concept introduced in this investigation by relying on alternative means, accelerated pavement tests and falling weight deflectometer (FWD) tests. Furthermore, research should be focused on the influence of parameters such as the determination of the range boundary factors as a function of the values of grading, aggregate type, density, moisture content, cycles etc.

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