Technical Paper

ISSN 1996-6814 Int. J. Pavement Res. Technol. 4(1):56-66

Mechanical Behavior of Unbound Granular Road Base Materials under Repeated Cyclic Loads

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Abstract: This paper aims to report the mechanical behavior of crushed rock base (CRB) and hydrated cement treated crushed rock base (HCTCRB) as granular road base materials 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 is well known, pavement surface rutting, longitudinal and alligator cracks are normally the main cause of damage in flexible pavements. The factors contributing to such damage are the excessive irreversible deformation of base layers and the behavior of a mechanical response of unbound granular materials (UGMs) under traffic loads which at the moment are not well understood. In this study, the shakedown concept was utilized to describe and determine limited use of CRB and HCTCRB subjected to different stress conditions. This concept is a theoretical approach used to describe the behavior under RLT tests. It utilizes macro-mechanical observations of the UGM's response and the distribution of the vertical plastic strain in the tested material. When the shakedown limit of an UGM is known, the limitations of the accumulated plastic strain in an unbound granular layer which causes rutting can be predictable. In this paper, compacted CRB and 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 revealed that road base defined the working stress ratio of a pavement structure and that after a large number of traffic loads, deterioration will occur. Moreover, the mechanical responses were investigated and the limit ranges of using UGMs in pavements were determined.

Key words: Crushed rock base; Hydrated cement treated crushed rock base; Repeated load triaxial tests; Unbound granular materials.

Introduction

The UGM layer with thin bituminous surfacing is widely used in the Australian road network. Generally, CRB and HCTCRB are used as unbound granular base course material in Western Australia. 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 not reliable enough to explain the 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 behavior and the amount of traffic during the service life. Consequently, the most economical and appropriate pavement material will be employed.

This paper focuses on applying the mechanical behavior of base course materials and developing the typical models of UGMs for pavement analysis in Western Australia. The empirical design method is unacceptable because the protocol required design parameter inputs from monotonic loading tests rather than cyclic loading tests which are more representative of real traffic 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 [1]. 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.

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 behavior of UGMs [2].

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,

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Note: Submitted October 14, 2009; Revised August 2, 2010; Accepted August 5, 2010.

conventional pavement construction is designed to provide adequate thickness over the sub layer in such a way that pavement structure has no shear failures and unacceptable permanent deformation taking place in each layer. For pavement design purposes, the stress level related to a reversible strain response must be determined and consequently not exceeded, once unacceptable permanent strains are prevented. This increases the possibility of a critical boundary stress between stable and unstable conditions.

The shakedown concept has been used to explain the behavior of conventional engineering structures under repeated cyclic loading. Basically, it was originally developed to analyze the behavior of pressure vessels subjected to cyclic thermal loading. Subsequently, it was improved to analyze the behavior of metal surfaces under repeated rolling or sliding loads. The theoretical approach of the UGMs' permanent deformation is used to describe the behavior of tested materials under RLT tests through macro-mechanical observations. These can predict progressive accumulations of plastic strains under repeated loading and whether the amount of the applied loads exceeds a certain limited-value called the shakedown limit or limit load [3].

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 [4] where in some cases, deterioration was reported due to stiffening or post-compaction after a number of load cycles [5]. Other studies have defined upper-bound [6] and lower-bound [7] for the shakedown limit of UGMs in simple pavement structures. At low stress levels, the mechanism of permanent strain is initially in a post compaction or re-arranged phase, when the strain rate is relatively high, however, it becomes level with the number of load cycles. A stable state may be maintained for a period of time unless the states change. Maree reported on the behavior 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 [8]. Numerous investigations have been conducted regarding the behavior of UGMs used in flexible pavements. Lekarp summarized the main findings regarding the effects of different material parameters and applied stresses on the permanent strain response of UGMs [9]. In the original shakedown concept, 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 plasticity in a few initial cycles, although the ultimate response is elastic after post-compaction. The strain is completely reversible and does not lead to 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 an accumulation of plastic strain (post-compaction). However, the material will show failure with a large number of load cycles.
- Incremental collapse range (Range C). The repeated loading is relatively high 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 [10]. Such responses are likely to take place when the pavement is subjected to the working load and could be due to a change in material response (compaction degree), the stress state, or a combination of both. With this understanding of material behavior, the shakedown concept typically then determines 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).

Permanent Strain under a Number of Load Cycles Models

In considering, the long-term behavior model of pavements, it is essential to take into account features that play an important role, the accumulation of permanent strain with the number of load cycles and stress levels. Hence the main research purpose focusing on long-term behavior 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, the permanent strain of UGMs for pavement application has been modeled in several ways. Some are logarithmic with respect to the number of loading cycles [11, 12] whilst others are hyperbolic, tending towards an asymptotic value of deformation with increasing numbers of load cycles [2, 13]. The first type, the permanent axial strain is supposed to accumulate in linear relation to the logarithms [11] as Eq. (1):

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

where ε^{p} is the permanent strain; a and b are regression constants; and N is the number of loading cycles. Long-term strain behavior was investigated by Sweere in a series of RLT tests and he 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 and the shakedown approach should be considered. Lekarp and Dawson [14] suggested this approach might also be employed in explaining the permanent strain behavior of

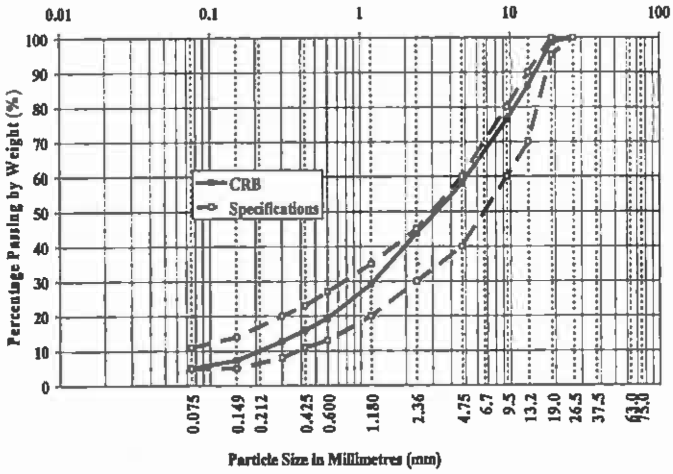

Fig. 1. CRB Grading Curves Compared with WA Main Roads Specifications [27].

UGM. In conclusion, they pointed out that more research is required to determine this shakedown limit. However, for finite element (FE) calculations of an UGM layer as part of a FE based pavement design, the prerequisite is a stress and load cycles dependent model for its permanent strain behavior. The Theyse and the Huurman-model use the shakedown approach, in particular modeling the stable and unstable permanent strain behavior to model the permanent strain behavior of UGM as a function of the number of load cycles [11]. Similar to what Sweere (1990) had found in his laboratory test results, the log-log approach was also used by Huurman [15] to describe the permanent strain development in an UGM layer in pavements under traffic using Eq. (3) and RLT apparatus to determine the permanent strain behavior.

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

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 the linear increase of permanent strain with N on a log (ε^p) - log (N) scale. 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 behavior, the model parameters C and D are equal to zero. It is clear that the unstable behavior at high stress levels cannot 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) scale is observed. To implement the RLT for measuring permanent strain development in a pavement structure, the permanent strains of the materials have to be determined in terms of the applied stresses and 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, the tests revealed that $\sigma_{1,f}$ could not be obtained for the crushed UGMs, as already explained. This research used the plastic Dresden-Model developed at Dresden University of Technology to determine parameters A and B as a function of the principal stresses σ_1 and σ_3 for the Range A (Eqs. (4) and (5)) and for the Range B (Eqs. (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)

a_2, a_3, b_2, b_3 model parameters

a_1, a_4, b_1, b_4 model parameters.

As already mentioned, the behavior observed for the higher stress level cannot be described by means of Eqs. (4)-(7). A second term was therefore, added to Eq. (3). The model parameters C and D are stress dependent and the test results revealed collapse. Parameters A and B were also determined. Finally, it was realized that it is possible to model the permanent deformation behavior of UGMs in a stress dependent way by modeling each behavior range separately.

Materials

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

Table 1. Characterization Tests [24].

Tests*	Results	Tests*	Results
Liquid Limit (LL)	22.4%	Coefficient of uniformity (C_u)	22.4
Plastic Limit (PL)	17.6%	Coefficient of curvature (C_c)	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 [25].

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

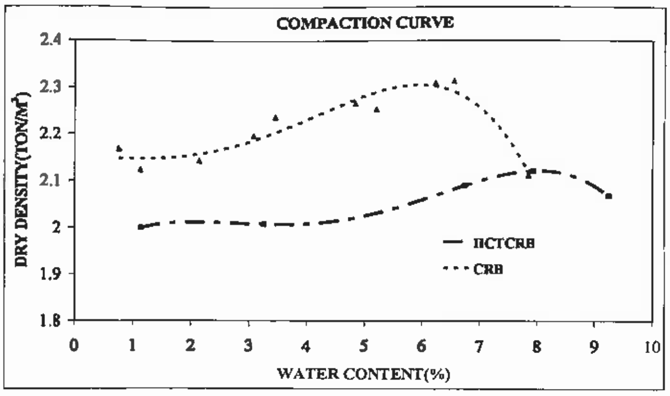

Fig. 2. Compaction Curves of CRB and HCTCRB.

Civil Engineering, Curtin University of Technology. They were prepared (see Fig. 1 for the grading curve) at 100% of maximum dry density (MDD) of 2.27 ton/m^3 and optimum moisture content (OMC) of 5.5%. Material properties achieve base course specifications [16]. Fig. 1 shows that the grading curve of the crushed rock in this study achieves the upper and lower bound of the base course specifications. Significant comparisons of basic properties with specifications were made as shown in Table 1.

Hydrated Cement Treated Crushed Rock Base (HCTCRB)

HCTCRB is manufactured by blending 2 % cement with a standard dry weight crushed rock base [16]. 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 [17] during the initial hydration 7-day period. Fig. 2 shows the comparison of the compaction curves between CRB and HCTCRB. In Fig. 2, MDD and OMC of HCTCRB change to 2.12 t/m^3 and 8% respectively from 2.27 and 5.5% of CRB. These of OMC values indicate that after cement hydration occurs, the impact of cement on soil compaction will increase optimum moisture content [18].

Cement

The cement used in this study was the bagged product of Cockburn Cement [19], General Purpose Portland Cement -type GP following the standard of AS 3972-1977 [20] as shown in Table 2.

Laboratory Program and Testing

Specimen Preparation

Sample preparations were carried out using a standard cylinder mould 100 mm in diameter and 200 mm in height by the modified compaction method [17]. 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 one was compacted. After compaction, the basic properties of each specimen were determined and 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.

Table 2. General Specifications of the Cement Used in This Study [19].

Parameter	Method	Units	Typical	Range	AS3972-1977 Limits
Chemical Analysis					
SiO_2	XRF	%	20.7	19.5-21.6	-
Al_2O_3	XRF	%	4.8	4.5-5.3	-
Fe_2O_3	XRF	%	2.7	2.3-3.1	-
CaO	XRF	%	63.8	62.2-65.5	-
MgO	XRF	%	2.1	1.5-2.8	-
SO_3	XRF	%	2.5	2.0-3.2	3.5% Max
LOI	AS2350.2	%	1.8	0.5-2.7	-
Chloride	ASTM C114	%	0.01	0.01-0.02	-
Na_2O equiv.	ASTM C114	%	0.50	0.45-0.65	-
Fineness Index	AS2350.8	m^2/kg	400	350-450	-
Normal Consistency	AS2350.3	%	29.5	28.0-30.0	-
Setting Times	AS2350.4				
Initial		mins	120	90-150	45 mins Min
Final		mins	190	135-210	10 hrs Max
Soundness	AS2350.5	mm	1	0-2	5 mm Max
Compressive Strength	AS2350.11				
3 days					
7 days		MPa	38	33-40	-
28 days		MPa	48	41-52	25 MPa Min
		MPa	60	53-68	40 MPa Min

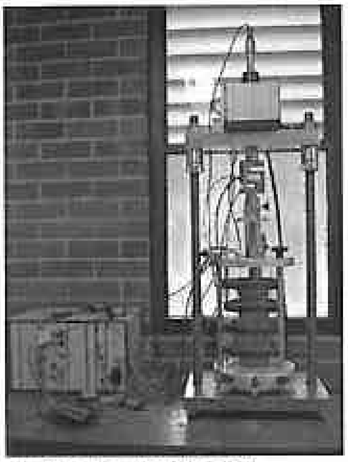

Fig. 3. The Repeated Loads Triaxial (RLT) Apparatus.

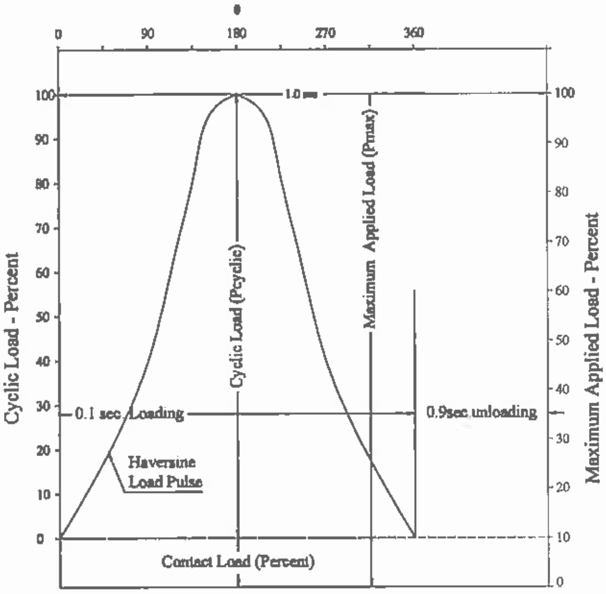

Fig. 4. The Vertical Loading Waveform.

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 as Fig. 3 shows. Controllers were used to manage the chamber, as well as 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 external linear variable differential transducer (LVDT) on the top of the triaxial cell, used to measure deformations over the entire length of the specimens were measured by the control and data acquisition system (CDAS) which provided the control signals, signal conditioning, and data acquisition. The CDAS was networked with the computer which provided the interfacing with the testing software and stored the raw test data. This enabled the resultant stress and strain in the sample to be determined.

This apparatus, however, 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 [21]. Moreover, although the apparatus allows a laboratory sample to be subject to cyclic axial deviator stresses, it is not feasible to vary the confining radial stresses at the same time. Confining pressure was generated to simulate lateral pressure acting on the surrounding samples as occurs in a pavement layer. After pressure was applied, stresses were found at different points in the granular material. The results were expressed in terms of deviator stress $q = \sigma_1 - \sigma_3$, mean normal stress $p = (\sigma_1 - 2\sigma_3)/3$, and the confining pressure was simulated from the pavement base course layer that is in common use in Western Australia. For this reason, it was decided to subject the laboratory samples to 11 different stress levels with a confining pressure of 40 kPa. After pressure had been applied, additional dynamic vertical stress was applied and 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 a 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 Fig. 4.

Resilient Modulus Tests and Permanent Deformation Tests

The standard method of Austroads APRG 00/33-2000 [21] 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 a frequency of 0.33 Hz to the static confining stress (σ_3) based on Austroads APRG 00/33-2000 [21], each involving 10,000 cycles for each particular stress condition as shown in Table 3. After the permanent deformation tests, in accordance with this standard [21], the specimens were applied sequentially by the different 65 stress stages (see Table 4) straightaway to conduct the resilient modulus tests to check the elastic condition of each specimen throughout the multiple loading stress stages. This process simulates complicated traffic loading

Table 3. The Permanent Deformation Austroad-APRG 00/33 [26].

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

acting on a pavement. Two hundred loading cycles of each stress stage were applied to the specimens. Table 4 shows the stress levels for the resilient modulus.

Results and Discussion

Resilient Modulus 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 specifications of CRB and HCTCRB, the results in the condition of 100% MDD at 100% OMC are represented to show their characteristics and to determine suitable mathematical models of resilient modulus and the permanent deformation of CRB and HCTCRB.

Fig. 5 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. Fig. 5 also shows that the results of resilient modulus of CRB and HCTCRB can be modeled reasonably well by using the K-Theta (K-θ) model [22] exhibited in Eqs. (9) and (10) respectively:

$$\text{CRB}: M_r = k_1 \cdot \theta^{k_2} = 1.86046\theta^{0.7606} \tag{9}$$

$$\text{HCTCRB}: M_r = k_1 \cdot \theta^{k_2} = 8.91026\theta^{0.6817} \tag{10}$$

where M_r is the resilient modulus in MPa; θ is bulk stress ($\sigma_1+\sigma_2+\sigma_3$) where ($\sigma_2 = \sigma_3$); σ_1 is the major principal stress (vertical axial stress); σ_3 is the minor principal stress (confining stress); k_1 and k_2 are regression coefficients as shown in Fig. 5.

Figs. 6 and 7 contain the typical results of the permanent deformation tests in terms of the relationship between permanent deformation and loading cycles for CRB and HCTCRB respectively to show comparisons of the measured and permanent deformation values and the predicted values for proposed permanent deformation models. They also indicate that the permanent deformation can be modeled quite reasonably by using the model suggested by Sweere, from SAMARIS [3].

He suggested for the long-term deformation behavior of unbound granular materials (UGMs) under a large number of load cycles, this approach should be employed as the proposed permanent deformation model of CRB and HCTCRB as shown in Eqs. (11) and (12), respectively.

Table 4. The Resilient Modulus Stress Levels Austroad-APRG 00/33 [26].

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

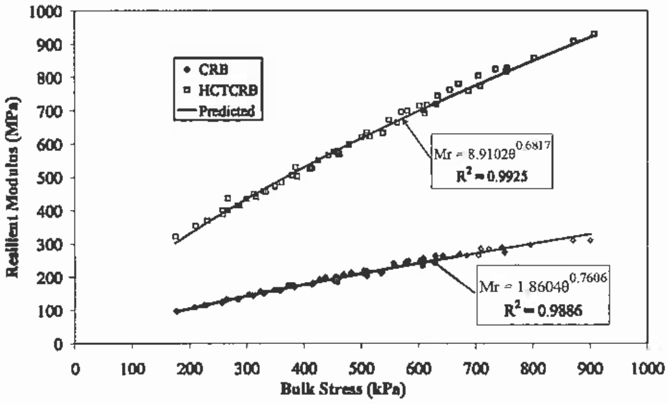

Fig. 5. The Resilient Modulus Predictions.

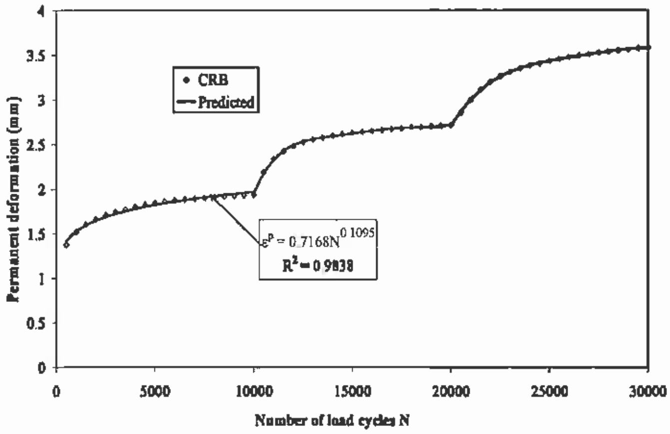

Fig. 6. The CRB Permanent Deformation Predictions.

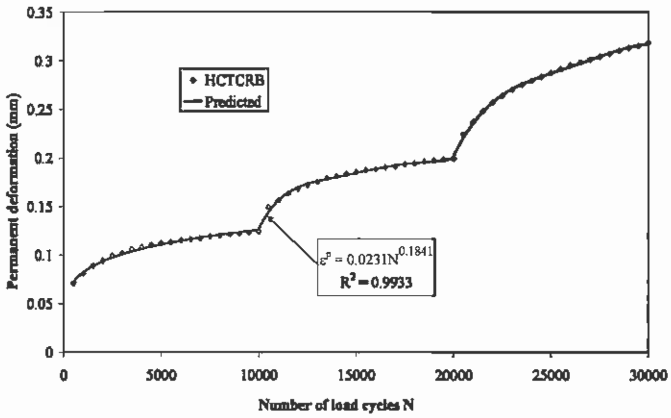

Fig. 7. The HCTCRB Permanent Deformation Predictions.

$$\text{CRB}: \varepsilon^{p} = A \cdot N^{B} = 0.7168 \cdot N^{0.1095} \tag{11}$$

$$\text{HCTCRB}: \varepsilon^{p} = A \cdot N^{B} = 0.0231 \cdot N^{0.1841} \tag{12}$$

where ε^{p} is permanent deformation in millimeters; A and B are regression constants; and N is the number of loading cycles.

Shakedown Behavior

Permanent deformation accumulations were observed as shown in

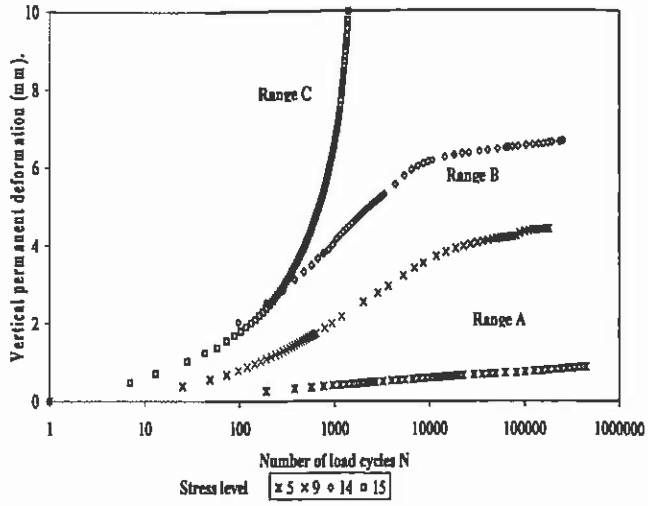

Fig. 8. CRB Permanent Deformation versus Number of Load Cycles (N).

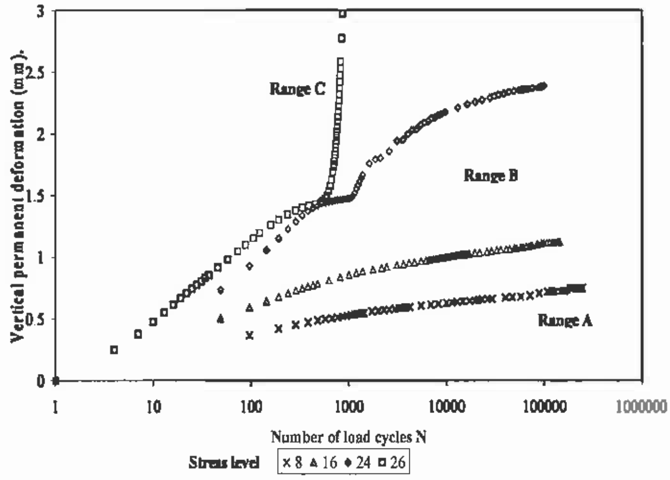

Fig. 9. HCTCRB Permanent Deformation versus Number of Load Cycles (N).

Figs. 8 and 9. As the test results reveal, CRB and HCTCRB always produce permanent deformation during cyclic loading; hence they cannot describe purely elastic behavior under repeated cyclic loads in course base materials [23]. The multi-layer linear elastic theory therefore is not satisfactory in analyzing the UGM layer. Permanent deformation behavior is described on the basis of internal friction between grains, particle shape, compaction, consolidation, distortion, etc and test results can be separated into the three ranges (A, B, and C) based on the shakedown concept.

Range A - Plastic Shakedown Range

The lower lines (Stress levels 5-9) in Fig. 8 for CRB indicate the response of Range A. The behavior 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 occur as seen in Fig. 8 and Fig. 12. Fig. 9 shows Range A of HCTCRB at stress levels 8-16 more than twice that of CRB and HCTCRB at a stress level 11

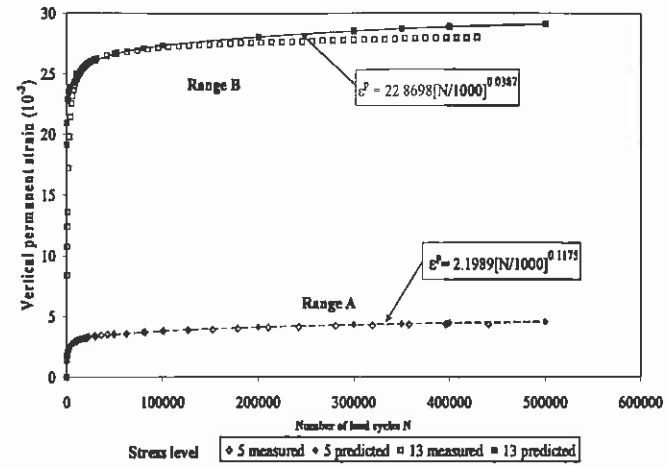

Fig. 10. CRB Ranges A and B Vertical Permanent Strain Compared with the Strain Model.

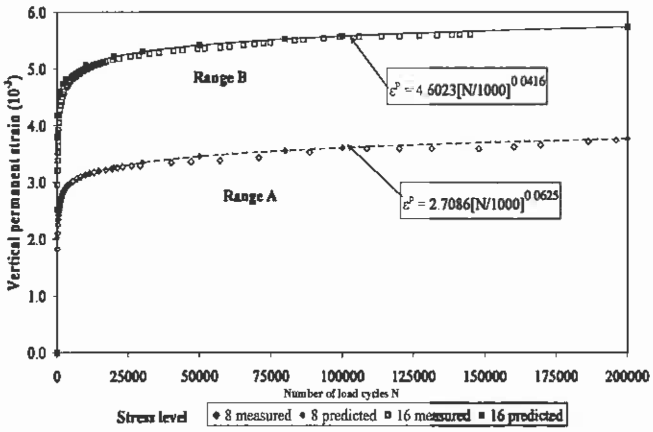

Fig. 11. HCTCRB Ranges A and B Vertical Permanent Strain Compared with the Strain Model.

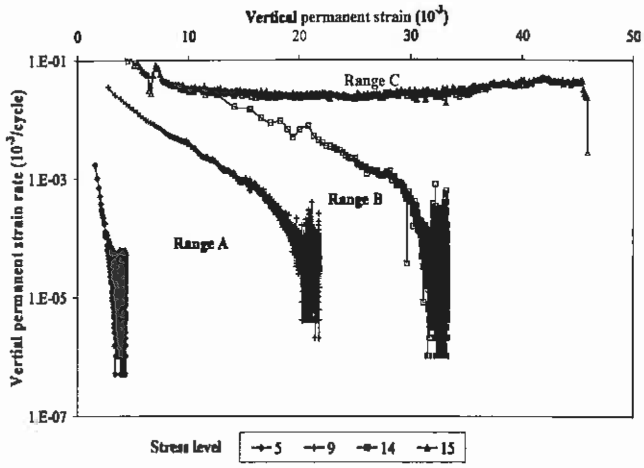

Fig. 12. CRB Permanent Strain Rate Versus Vertical Permanent Strain.

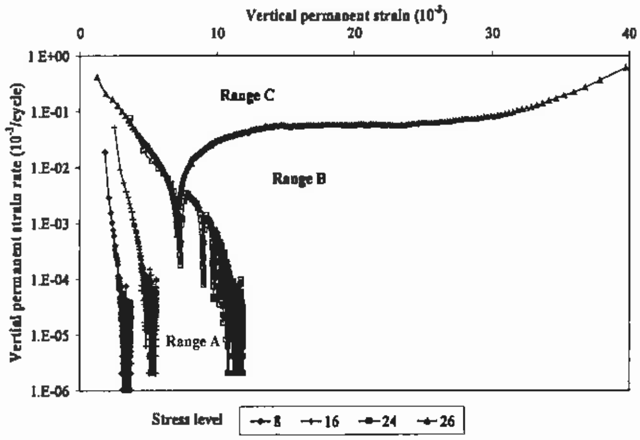

Fig. 13. HCTCRB Permanent Strain Rate versus Vertical Permanent Strain.

Table 5. Coefficients for Range A and B of model Eqs. (4) to (7).

Type parameter	CRB		HCTCRB	
	Range A	Range B	Range A	Range B
a_1 [-]	0.000115	0.00050	0.00185	0.00011
a_2 [kPa^{-1}]	-0.02000	1.44000	-0.11000	1.45000
a_3 [kPa^{-1}]	0.000001	-0.00150	0.0000013	-0.00040
a_4 [-]	1.7600	1.3550	1.8500	1.4450
b_1 [-]	0.00050	0.00735	0.00030	0.00350
b_2 [kPa^{-1}]	-0.00800	0.00010	-0.01200	-0.08550
b_3 [kPa^{-1}]	0.00590	0.00010	0.00200	0.00010
b_4 [-]	0.55000	0.000001	0.12000	0.55000

achieved Range A. Figs. 12 and 13 indicate that the vertical permanent strain rate decreases rapidly until it reaches a state of equilibrium.

Figs. 10 and 11 compared measured strain with the strain model and model coefficients as shown in Table 5. 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 behavior in these stress levels would become stable after post-compaction under a service load. In Range A of the shakedown behavior, a small accumulated displacement is allowed in the pavement as acceptable permanent deformation in the course base layer and which would terminate after a set number of load cycles. The material does not reach failure.

Range B- Plastic creep

Figs. 12 and 13 show an intermediate response of Range B (Stress levels 10-14 for CRB and 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 revealed that although the deformation is not completely resilient, permanent deformation is acceptable for the first period of the cycles. In Figs. 10 and 11, the vertical permanent strains are compared with the strain model and model coefficients as shown in Table 5. CRB reacts corresponding to Range B and a great number of failures could occur if conditions do not change. If it is maintained long enough, it deteriorates in the end as Range C.

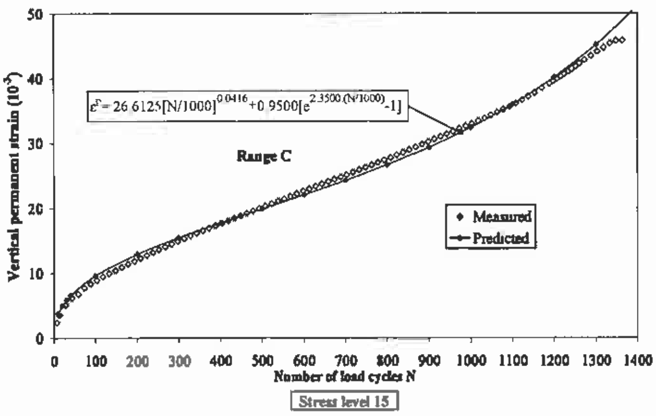

Fig. 14. CRB Range C Vertical Permanent Strain Compared with the Strain Model.

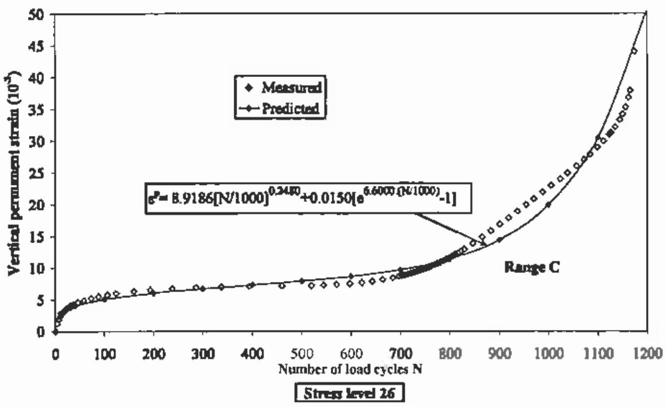

Fig. 15. HCTCRB Range C Vertical Permanent Strain Compared with the Strain Model.

Range C – Incremental Collapse

Figs. 12 and 13 (Stress level 15 for CRB and 26 for HCTCRB) indicate Range C behavior and the permanent strain rate decreases during the first period of load cycles after which it becomes lower, nearly constant. Failure occurs with a relatively small number of load cycles as the cumulative permanent strain rate increases very rapidly and does not decrease again. UGMs do not reach a stable state. Range C behavior in UGMs would result in the failure of the pavement, shear deformation in the base layer experienced as rutting at the road pavement surface. This range should not develop in a designed pavement.

In Figs. 12 and 13, there are distinctions between the behavior of Ranges A, B and C. However, only Range C presents vertical strain with no cessation of strain accumulation under a number of cycles. These can also be distinguished on the basis of plastic strain rate behavior. With Range A, the permanent strain rate decreases rapidly and does not reach a constant level throughout the duration of testing. In Figs. 14 and 15, the vertical permanent strain of Range C is compared with the strain model.

Conclusions and Discussion

The mechanical behaviors of CRB and HCTCRB, normally used as a base course material in Western Australia, were investigated by RLT tests. These were the resilient modulus and the permanent deformation test to obtain an understanding of the resilient and permanent deformation characteristics of this material under real conditions of traffic loading simulated in the tests. The resilient modulus characteristics could be modeled using the K-θ model [22] and the long-term permanent deformation characteristics could be modeled by using Sweere's model [12].

It has been shown that the use of the shakedown concept application to UGMs in pavement analysis is possible. This study defined the limit ranges (Ranges A, B, and C) of CRB and HCTCRB. UGMs under fixed stress level conditions show relationships between permanent strain and stress level. When a cyclic loading is applied, a sample responds by changing its permanent strain. With a continuous and gradual increase of the loading amplitude $\Delta\sigma$, the material will begin to change its mechanical behavior. The possibility of a purely elastic approach in pavement analysis is discarded as no such response was found in CRB and HCTCRB during repeated cyclic loading.

For low stress ratios, the CRB reacts corresponding to Range A under stress levels 9 and HCTCRB under stress levels 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 further permanent deformation. The vertical strain rate rapidly increases and the material subsequently responds elastically. Hence Range A of CRB and HCTCRB is acceptable in pavement construction if the accumulated strains before the development of fully resilient behavior is sufficiently small. The next step is to examine the application of material in the pavement that responds to Range B. For higher loadings at stress levels 10-14 of CRB and 16-24 of HCTCRB, the energy input is first quickly dissipated by a re-arrangement of the sliding internal contacts of material, the so-called post-compaction. The dissipated energy per cycle relaxes to a stationary value so that the vertical strain decreases to a constant rate depending on the loading and the characteristics of the grains such as the friction or the stiffness of the contacts. A thorough investigation of the size dependence of the 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 simulation than in the full scale 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 is ultimately required to correct inadequacies other than traffic-induced rutting, Range B behavior will probably be acceptable. Range C behavior at stress level 15 and 26 for CRB and HCTCRB respectively, should not be allowed to occur in the pavement. If the stress levels imposed are high, there is no possibility of the material to re-arranging itself to the new state and post-compaction will lead 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 (plastic shakedown limit) can be used to predict whether or not stable state occurs in the UGM layer of the road structure. The plastic shakedown limit of CRB and HCTCRB should be used in 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 roads were level 11 at the base layer indicating that CRB reacts corresponding to Range B behavior with possible deterioration at a number of load repetitions. HCTCRB achieves Range A behavior and will be stable at a certain amount of traffic without rutting failure. This new approach has been partially validated by the data from which these guidelines have been derived. It has been shown that the permanent strain characteristics of CRB and HCTCRB could be modeled, each behavior range separately using the Dresden-model.

This paper maintains that having defined the ranges from laboratory results, it is possible to determine whether CRB and HCTCRB are adequate as base courses or whether further thicknesses of surfacing layer are required to implement satisfactory pavement performance. However, suitable experience is currently unable to confirm the reliability of this proposed relationship between the ranges defined by the RLT tests and real performance. Pavement mechanical response is affected by several parameters. Further investigation of this topic will be necessary to verify the results of the concept introduced by employing 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. There is plenty of scope for further studies.

References

1. Collins, I.F., Wang, A.P., and Saunders, L.R., (1993). Shakedown theory and the design of unbound pavements, *Road Transportation Research*, 2(4), pp. 28-39.
2. Wolff, H. and Visser, A.T., (1994). Incorporating elasto-plasticity in granular layer pavement design, *Transportation Engineering*, Vol. 105, pp. 259-272.
3. SAMARIS, (2004). Selection and evaluation of models for prediction of permanent deformations of unbound granular materials in road pavement, *SAM-05-DE10*, Sustainable and Advanced Materials for Road Infrastructure.
4. AASHTO, (1986). *Guide for the design of pavement structures*, Association of State Highways and Transportation Officials, Washington, D.C., USA.
5. Kent, M.F., (1962). Road test vehicle operating costs related to gross weight, *Highway Research Board Special Report 73*, pp. 149-165, Washington, D.C., USA.
6. Collins, I.F. and Boulbibane, M., (1998). The application of shakedown theory to pavement design, *Metals and Materials International*, 4(4), pp. 832-837.
7. Yu, H.-S. and Hossain, M.Z., (1998). Lower bound shakedown analysis of layered pavements using discontinuous stress fields, *Computer Methods in Applied Mechanics and Engineering*, Vol. 167, pp. 209-222.
8. Maree, J.H., Freeme, C.R., van Zyl, N.J., and Savage, P.F., (1982). The permanent deformation of pavements with untreated crushed stone bases and measured in Heavy Vehicle Simulator tests, *10th Australian Research Board Conference*, Part 2.
9. Lekarp, F., Isacsson, U., and Dawson, A.R., (2000). State of the Art. II: Permanent strain response of unbound aggregates, *Transportation Engineering*, 126(1), pp. 76-84.
10. Sharp, R.W., (1985). Pavement Design Based on Shakedown Analysis, *Transportation Research Record*, No. 1022, pp. 99-107.
11. Barksdale, R.D., (1972). Laboratory Evaluation of Rutting in Base Course Materials, *3rd International Conference on the Structural Design of Asphalt Pavements*, London, UK.
12. Sweere, G.T.H., (1990). Unbound granular bases for roads, *PhD Dissertation*, Delft University of Technology, the Netherlands.
13. Paute, J.L., Hornych, P., and Benaben, J.P., (1996). Repeated load triaxial testing of granular materials in the French Network of Laboratories des Ponts et Chaussées, *European Symposium on Flexible Pavements*, Balkema, Rotterdam, the Netherlands.
14. Lekarp, F. and Dawson, A.R., (1996). Influences on Permanent Deformation Behavior of Unbound Granular Materials, *Transportation Research Record*, No. 1547, pp. 68-75.
15. Huurman, M., (1997). Permanent deformation in concrete block pavements. *PhD Dissertation*, pp. 119-125, Delft University of Technology, the Netherlands.
16. Main Roads Western Australia, (2003). A guide to the selection and use of naturally occurring materials as base and subbase in roads in Western Australia, *Standards and Technical*, Main Roads Western Australia. Online http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}, Last Accessed September, 2008.
17. Main Roads Western Australia, (2007). Dry density/moisture content relationship: modified compaction fine and medium grained soils, *Standards and Technical*, Main Roads Western Australia. Online http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}, Last Accessed September, 2008.
18. Zhang, Z. and Tao, M., (2008). Laboratory Testing of Cement-Treated Wet Subgrade and Design Recommendations, *International Journal of Pavement Research and Technology*, 1(4), pp. 135-142.
19. Cockburn Cement, (2006). Cockburn General Purpose Portland Cement Type GP, *General Specification*. Online http://www.cockburncement.com.au/productinfo/range/specifications/Cockburn%20GP.pdf, Last Accessed November, 2006.
20. Australian Standard, (1997). *Portland and Blended Cements*, pp. 1-10, Australian Standard, Australia.
21. Voung, B.T. and Brimble, R., (2000). Austroads Repeated Load Triaxial Test Method-Determination of Permanent Deformation and Resilient Modulus Characteristics of

Unbound Granular Materials under Drained Conditions, *APRG DOCUMENT APRG 00/33(MA)*, Austroads, Australia.

22. Hick, R.G. and Monosmith, C.L., (1971). Factors influencing the resilient response of granular materials, *Highway Research Record*, No. 345, pp. 15-31.
23. Werkmeister, S., Dawson, A.R., and Wellner, F., (2001). Permanent deformation behaviour of unbound granular materials and the shakedown theory, *Transportation Research Record*, No. 1757, pp. 75-81.
24. Main Roads Western Australia, (2007). Test Method (Aggregate), *Standards and Technical*, Main Roads Western Australia. Online http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}, Last Accessed September, 2008.
25. Main Roads Western Australia, (2006). Test Method (Aggregate), *Standards and Technical*, Main Roads Western Australia. Online http://www.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={1532D87F-C1AC-4386-9968-5E5F4FD002E5}, Last Accessed September, 2006.
26. Austroads, (2004). *Pavement Design - A Guide to the Structural Design of Road Pavements*, Austroad Inc., Australia.
27. Jitsangiam, P. and Nikraz, H., (2007). Mechanical Behaviour of Hydrated Cement Treated Crushed Rock Base (HCTCRB), *Australian Geomechanics*, 42(3), pp. 133-143.