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DEFORMATION BEHAVIOUR OF UNSATURATED SOIL AS CRUSHED ROCK BASE (CRB) LAYER

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Abstract

This paper reports the deformation behaviour of crushed rock with unsaturated condition under repeated cyclic loading triaxial (RLT) tests performed at different stress levels, in order to investigate the effect of moisture content to crushed rock performances and permanent deformation behaviour. Road surface rutting is the core basis of damage in flexible pavements which the mainly significant is crushed rock still unclearly thoughtful regarding deformation behaviour under real situation. RLT test is used to describe the various deformation responses of unbound granular materials (UGMs) by macro-mechanical observations. The main study focused on the influence of moisture content. Moreover permanent deformation behaviour of crushed rock will be investigated and cyclic loading limit ranges of crushed rock should be considered in pavement analysis and design.

Keywords: CRB, permanent and resilient deformation, RLT, unsaturated soil

1. Introduction

UGMs are widely used in the road system and crushed rock is used as a UGM base course material. A road base structure in service is subjected to the accumulated number of various vehicles together with climatic influence cause deterioration over time. The main purpose of base course is to distribute and reduce stress and strain quantity of vehicle wheel loads into sub layers with no unacceptable strain at the top of subgrade. Current pavement design is absence of the surrounding environmental effects (such as moisture, temperature). The result of improper design reduces service life and increases maintenance and user expenses which rutting, cracking and shoving in the longitudinal and transversal direction represents usual deteriorations. In order to provide roads with a long service life, the potential of deterioration have to be terminated. Especially crushed rock base in the frequent situation, performs in unsaturated condition that is inevitable to deal with moisture effects. An obvious knowledge of complete characteristics of materials relevant to pavement mechanistic design is very great important to gain the efficiency of such materials. In reality finance limitations are restricting investments or design for long life pavements in particular on low-volume traffic roads. It is a need for utilising the material strength in each layer properly, and to take the costs into consideration when doing the pavement design. UGMs need to be investigated to improve analysis and design more precisely than in the past with respect of deformation during the service life. Consequently, a most economical construction and an appropriate material type for the pavement will be determined.

This paper focuses on the permanent deformation of crushed rock under unsaturated condition and developing the specific requirement of crushed rock for pavement construction. The empirical design method is unacceptable because the procedure ignores the environmental effect and real traffic loading conditions. From these issues, several roads reached premature deterioration before design service life. 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 achievement of this approach is the experimental measurement and appropriate characterisation of the mechanical responses from the RLT test which is the basic protocol of this study.

2. Background

UGMs are very significant pavement construction materials that are used as base, subbase and subgrade. Understanding on the characteristics and behaviour of UGM with the response of the materials when subjected to applied load, is therefore necessary for practical work. Basically, UGMs are rather complicated materials to deal with and show varied behaviour as a consequence of physical history that has an influence on the mineralogical composition, the particle form and particle size distribution. Moreover the actual degree of compaction and moisture content are of enormous importance. Normally, an UGM consists of gravels or crushed rock aggregates which have particular grading that formulates mechanical strength, practicable and are able to be compacted. Their performances are largely governed by their shear strength, stiffness and resistance of material breakdown under construction and traffic loading. The most common modes of UGM deterioration are rutting due to insufficient resistance of deformation.

In road construction, water is used to contribute and lubricate individual particle during compaction to achieve high levels of compaction. In this case the water acts as a lubricant between the grains, allowing slipping into the most effective packing. With out water, UGMs are unable to achieve the maximum density but moisture also presents a major contribution to the deterioration processes. Water plays a major role interface with the material properties which water in a pavement has its source from many sources such as groundwater, stormwater, even if construction process of the road. In many low volume roads, the drainage system is not designed for large amount of surface water, so during a long rainy period the water level in the drainages may increase and penetrate into the road structure. Water is a polar substance, which means the particles have a definite positive and negative direction. This makes that the molecules are able to dissolve with the minerals in the aggregate surface. Water also tends to migrate into the pore system by capillary attraction if the pores are small enough, which is related to the grain size distribution of the material and the amount of fines. Too much water in the road structure combined with the repeated loading from traffic may cause reduced effective stress and due to excessive pore pressure in the pavement material. This is what happens in moisture susceptible materials during the wet climatic period. The consequences reduced strength capacity and increased deformation in the base and subbase leading to cracking and rutting of the pavement. Traffic load in the pavement structure is transferred by grain-to-grain contacts. Eventually these may lead to deterioration by crushing and abrasion in the particle contact surfaces. The particles may then rearrange to form denser structures. The deformation behaviour of UGMs under repeated loading may be separated into two modes; resilient mode (recoverable) and permanent deformation mode. These types of deformation mechanisms are usually treated independent in the systematic approach. The UGM deformation behaviour is known to be subjective by many factors and moisture content.

Many researchers have studied the effect of water on the resilient and permanent deformation. Hicks and Monismith (1971) reported an apparent increase in deformation with increasing water content [2]. Barksdale and Itani (1989) observed a significant decrease in resilient modulus of soaking material that all samples were run under drained conditions [3]. For the well-graded materials, moisture seemed to be positive until optimum moisture content was achieved after that the resilient modulus decreased towards complete saturation. The resistance of deformation is influenced by the changes in moisture content rather than the resilient modulus for UGMs. An UGM is being susceptible to water and has the capability to rise up water into the pore structure. Water is unusual to concern the deformation of coarse single-sized particle materials significantly. Because of the water is not kept in the pore structure but is more or less on the surface of the particles and the simply effect of the water here would be as a lubricant. However, over time a uniformly graded material may be subjected to interparticle crushing. The accumulation of fines may change the grading of the material and make it more sensitive to water. By using strong materials the risk of this is reduced.

The nature of empirical pavement design procedure is based on experience and the results of simple tests. Consequently, the use of empirical approaches becomes sub-standard. Traditional procedure has been criticized by Wolff [4], who argued that it is too simplistic and does not take into account the environmental effects of UGMs. 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 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. The RLT test has been used to simulate the behaviour of conventional engineering structures under repeated cyclic loading.

3. Laboratory program and testing

3.1 Crushed rock

Crushed rock is composed of rock fragments produced by the crushing and screening of igneous, metamorphic or sedimentary source rock. The crushed rock samples used in this study were taken from a local stockpile and kept in sealed containers. The crushed rock samples were prepared at 100% of maximum dry density (MDD) of 2.27 ton/m^3 and optimum moisture content (OMC) of that 5.5% and were tested at various ratios of OMC (100%, 80% and 60%) to observe the effect of moisture content in term of deformation response. Material properties achieved the base course specifications [9].

3.2 Specimen preparation

Sample preparations were carried out by using a standard cylinder mould 100 mm in diameter and 200 mm in height by the modified compaction method [10]. Compaction was accomplished in 8 layers with 25 blows of a 4.9 kg rammer at a 450 mm drop height each layer. Full bonding between the layers of each layer was achieved by scarifying to a depth of 6 mm before for the next layer was compacted. Each compacted crushed rock sample was left open at the top of the mould to simulate as close as possible to the dry back process in pavement construction (as shown in Figure 1). Dry back targets of 80% OMC and 60% OMC were desired. After the dry back 1 day period, the basic properties of each specimen were determined after which the specimen was carefully carried to the base platen of the triaxial cell. A crosshead and stone disc were placed on the specimen and it wrapped by a rubber membrane and finally sealed with o-rings at the top and bottom platens.

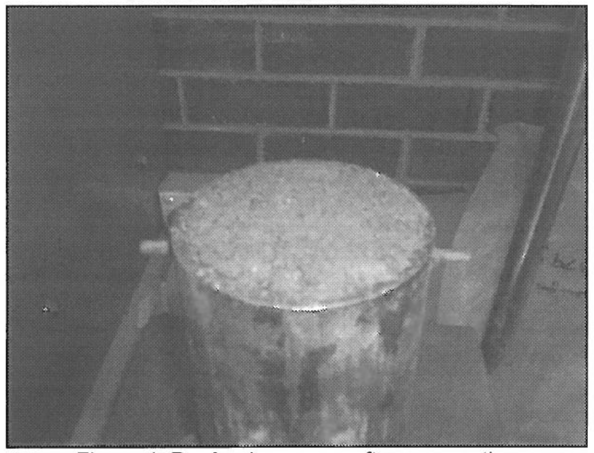

Figure 1: Dry back process after compaction

3.3 Repeated cyclic load triaxial tests

The tests were carried out with a cyclic triaxial apparatus consisting of a load actuator and a removable chamber. The specimens were placed in the triaxial cell between the base platen and crosshead of the testing machine as shown in Figure 2. Controllers were used to manage the vertical load, as well as the confining air pressure. The analog signals detected by the transducers and load cell were received by a module which transformed the signals into digital signals. Communications between the user and the triaxial apparatus were made by computer. The computer software makes it possible to select the type of test to be performed as well as all the parameters, stress levels, and the data to be stored. The load cell, the confining pressure and the externally linear variable differential transducer (LVDT) on 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, 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 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 [11].

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 by air under pressure 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 is commonly used in Western Australia. For this reason, it was decided to subject the laboratory samples to 11 different stress levels (also referred to as stress ratios) and the constant confining pressure of 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 (σ_1/σ_3) of 5 to 15. 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 waveform frequency of 1 Hz over a period of 1.0 sec and a load pulse of 0.1 sec duration.

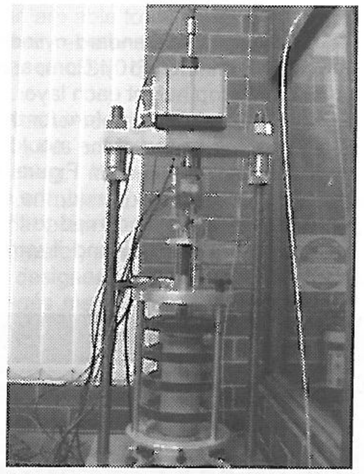

Figure 2: The repeated loads triaxial apparatus.

3.4 Permanent and resilient modulus tests

The standard method of AustRoads APRG 00/33-2000 [12] for Repeated Load Triaxial Test Method was followed for the permanent deformation tests. New specimens were prepared as stated in the previous section. The testing was performed during which, the specimens were loaded with various stress stages at the ratios of the dynamic deviator stress (σ_d) with frequency of 0.33 Hz (see the vertical force waveform in Figure 3) to the static confining stress (σ_3) as shown in Table 1 and Table 2.

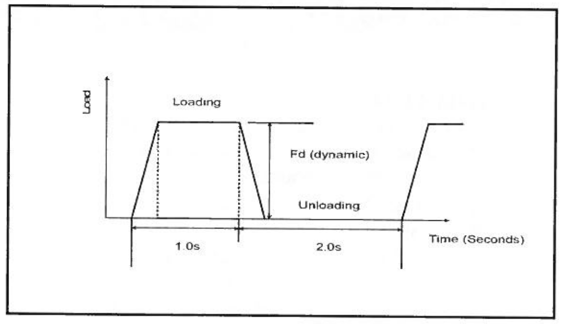

Figure 3: The vertical loading waveform.

Table 1: Stress levels for permanent deformation following AustRoads-APRG 00/33 standard.

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

Table 2: Stress levels for the resilient modulus following AustRoads-APRG 00/33 standard.

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

4. Results and Conclusions

The permanent deformation behaviours of crushed rock normally used as base course material were investigated by RLT tests. The tests were carried out to determine the resilient modulus and the permanent deformation and the effect of moisture content for the soil in an unsaturated condition under traffic loading. The resilient deformations in this study were indicated by resilient modulus. The resilient modulus from the RLT tests is defined as the ratio of the axial deviator stress to the resilient axial strain:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \qquad (1)$$

where

M_r = [MPa] resilient modulus
σ_d = [kPa] the repeated deviator stress
ε_r = [-] the resilient strain

Based on the specification of crushed rock, the results of 100%, 80% and 60% OMC dry back were reviewed for moisture effect. Figure 4 shows the results of the resilient modulus tests which were plotted against the bulk stress ($\sigma_1+\sigma_2+\sigma_3$). Generally, the plots are non-linear with respect to the magnitude of applied stresses. Figure 5 shows the typical results of the permanent deformation tests in terms of the relationship between permanent deformation and loading cycles for crushed rock at different %OMC. From the laboratory results, it was found that:

- All crushed rock samples exhibit stress-dependency behaviour.

- There is a large improvement of resilient modulus and permanent deformation between 100%OMC and 80%OMC after dry back process. However the results indicate little difference of resilient modulus and permanent deformation characteristics between 80%OMC and 60%OMC. So partial saturation greatly affects the performance of crushed rock. During tests on the 100%OMC samples, the pore pressure inside the soil skeleton reduced the crushed rock stiffness and the cohesive bonding was degraded during the repeated loading.

- Based on 65 stress levels, the resilient modulus values of all crushed rock samples were between 150 MPa and 550 MPa at 60%OMC.

- Crushed rock should achieve at least 80% OMC in the dry back process to improve its performance before bituminous surfacing.

- Three ranges of permanent deformation accumulation were observed as shown in Figures 6 and 7. Permanent deformation behaviour is described 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) [1].

Range A - The lower lines (Stress levels 5-9) in Figures 6 and 7 indicate the response of Range A. The behaviour is entirely plastic for a number of 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. In Figure 7, the vertical permanent strain rate decreases rapidly until it reaches a state of equilibrium. For this range of material, the 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. Crushed rock base (CRB) behaviour under these stress levels would become stable after post-compaction under service loads. Also, Range A is allowed in the pavement; an acceptably small accumulated displacement would be seen in a course base layer and this would terminate after a set number of load cycles. The CRB would not reach failure.

Range B- Figures 6 and 7 show an intermediate response of Range B (Stress levels 10-14). 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 (Figure 6). Test results were observed that although the deformation is not completely resilient, permanent deformation is acceptable for the first cyclic period. Failures could occur if the condition does not change and if cyclic loading is maintained long enough, it may deteriorate to Range C response.

Range C – Stress level 15 indicates Range C behaviour and the permanent strain rate decreases during the first period of load cycles then becomes lower, nearly constant. Failure occurs within 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. The CRB cannot maintain a stable state. Range C behaviour in CRB would result in the failure of the pavement by shear deformation in the base layer and would be experienced as rutting of the road pavement surface. This range should not develop in a standard designed pavement. 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 a constant level throughout the duration of testing.

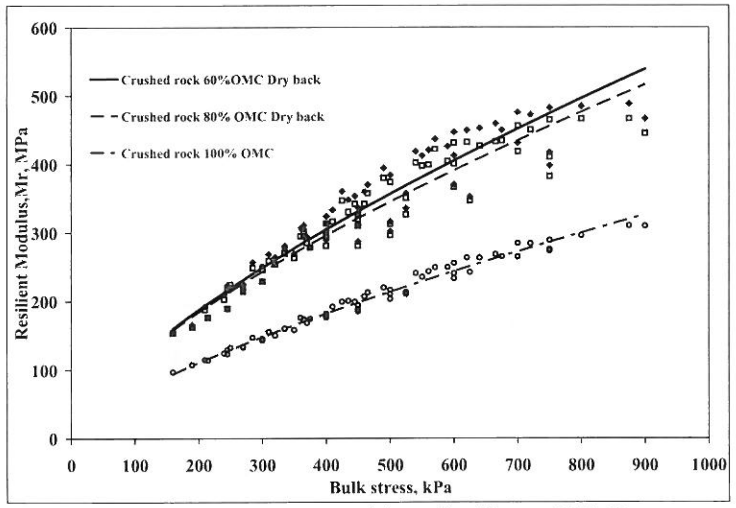

Figure 4: The resilient modulus with different %OMC.

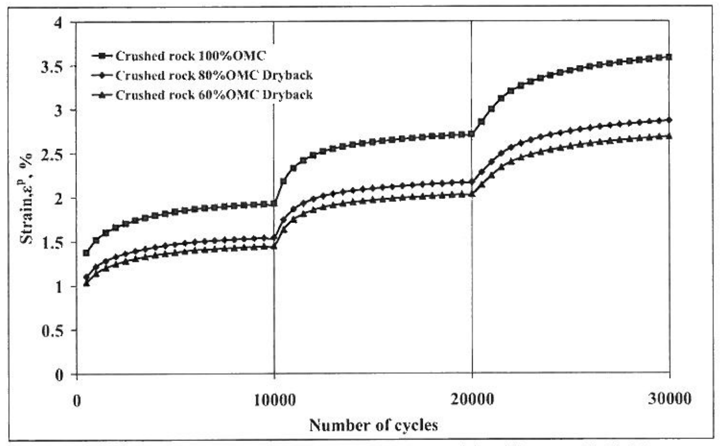

Figure 5: The permanent deformation with different %OMC.

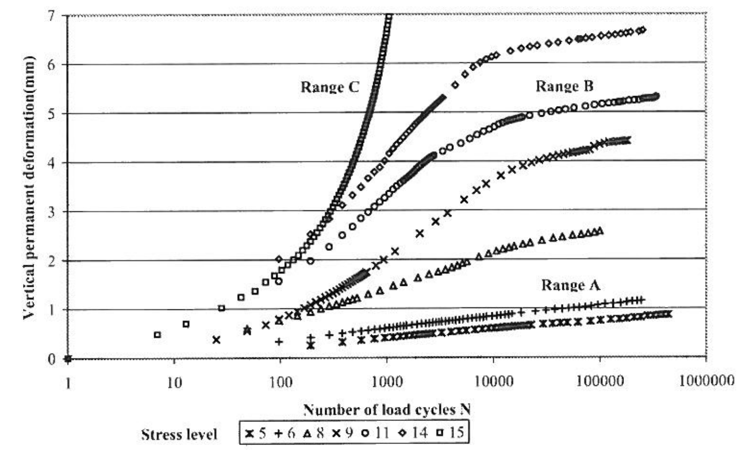

Figure 6: Vertical permanent deformation versus number of load cycles.

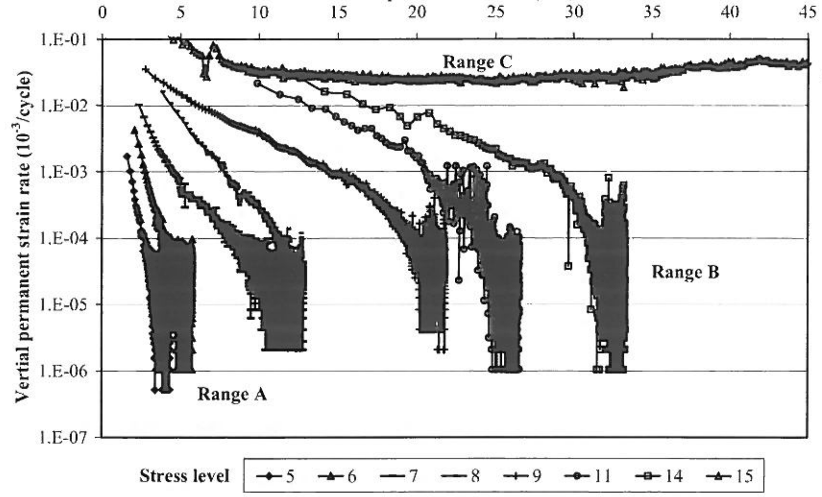

Figure 7: Vertical permanent strain rate versus vertical permanent strain.

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