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Theoretical Challenges of Cement Modified Crushed Rock Base Material In Pavements

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Abstract

The use of Crushed Rock Base (CRB) course materials in Western Australia (WA) has had the challenge of moisture susceptibility. This causes the reduction of resilient modulus and consequently undesirable deformations in wearing course of pavements. As such, the drying back method along with two methods of cement modification in the form of low-cement CRB material and Hydrated Cement Treated Crushed Rock Base (HCTCRB) have been tried and utilised in practice by stakeholders. Thus, the results of field experiences require to be reviewed in conformity with theoretical assumptions to provide better understandings for further improvements. Thus, based on laboratory test results, the reliability of the above methods is discussed in this paper after reviewing the background of each method.

1 Introduction

CRB material is the common base course material in WA, which includes the granite and dolerite aggregates. In 1994, the performance of CRB material assessed using deflection surveys in different metropolitan areas and repeated load triaxial (RLT) tests in the laboratory. The results showed that this material has poor performance when its field moisture content is more than 60% of Optimum Moisture Content (OMC) as Relative Moisture Content (RMC) and should not be used (Butkus, 1994). It has moisture susceptibility and loses about 20 to 25% of its Modulus resilient (Mr) with nearly 1% increase in moisture content. Thus, the drying back of the material after compaction, cement modification and combination of these methods have been studied as different options for treating this material. In case of cement modification, the concern of fatigue cracks with as little as 1% cement addition exist (Butkus & Lee Goh, 1997), which has led to the idea of HCTRCB material. Production of HCTCRB material, usually includes the addition of 2 % GP cement to the CRB, mixing by pug mill and stockpiling for at least 7 days for hydration. Then this material is disturbed, loaded onto the trucks and carted to work place.

Further investigations continued by nine trial sections with total 860m length in Reid highway in 1996 (Butkus, 2004). Details of these sections are shown in Table1.

Table 1 Reid Highway trial section base course material and thickness (Butkus, 2004)

Section	Modified Base Course Material	Thickness (mm)
1	2% HCTCRB	100
2	2% Bitumen Stabilised Limestone	100
3	Crushed Rock Base	100
4	Crushed Rock Base	200
5	1% HCTCRB	200
6	2% HCTCRB	200
7	0.75% Cement CRB	200
8	2% Cement Stabilised Limestone	200
9	LIMUD	200

Note:
HCTCRB = Hydrated Cement Treated Crushed Rock Base
LIMUD = Lime stabilised base course

The design traffic was 3.5E+7 equivalent standard axles (ESAs) for 40 years. In addition, using empirical design procedure, 135 mm crushed limestone sub base considered over the sand subgrade with California Bearing Ratio (CBR) of 12%. Constructions of base course layers commenced from March and completed in May 1996. In sections 1, 5 and 6, base course layers primed after one, nine and ten days after compaction, respectively. This was to adjust the drying back moisture content of section 1 for 95% RMC and the other two sections for 85% RMC. Primer utilised at the rate of 0.9 L/m2 and blinded with sand to prevent further drying back of base course before sealing. Finally, test sections were covered with a nominal 30 mm of 10 mm dense graded granite asphalt and were opened to traffic in Dec 1996. The actual asphalt thicknesses vary between 44 mm and 65 mm.

Later performance evaluation of these sections showed that the 2% HCTCRB in section 6 was in the first rank over 0.75% cement modification in section 7 in the second rank. However, section 6 had significant transverse cracks at the centre line and on both shoulders. It was thought that moisture changes were the reason due to highly active clay underlying the embankment of this section since the cracks were not in the wheel paths and were not believed to be due to traffic loads. Section 1 was in the low in the ranking of all sections, which emphasize the likely effects of high moisture content of material beyond RMC assumptions in designs. The other reason of these cracks could be cement disappearance or carbonation as observed in core samples as reported by Harris and Lockwood (2009). This subject has been discussed in details by authors (Rezagholilou & Nikraz, 2012).

In other experience in Orrong Rd in east of Perth city , insitu stabilisation of 150mm CRB with 0.75% cement was used between McDowell St and just west of Felspar Road in 2001 and opened to traffic in 2003. The HCTCRB material application continued up to Roe Highway to compare these various cement content materials. Recent observations in 2014 show that while 0.75% cement section looks stressed, the HCTCRB section was patched significantly.

The above experiences of stabilisation have resulted more stringent considerations in Engineering Road Note 9 (2012) by MRWA which inhibit the incorporation of ordinary cemented materials in flexible pavements unless as a space layer under a full depth asphalt pavement or under a HCTCRB base course layer. The 7-days Unconfined Compressive Strength (UCS) of HCTCRB or cement modified layers (with 2% LH cement) are limited to less than 1.0 MPa to make restrictions in tensile strength and fatigue cracks. This criterion obtained by Cray et al. (1997) upon field observations in WA.

This inhibition for low-cement modified CRB material has been the main motivation here to compare it with HCTCRB material in design parameters and also evaluate the dry back concept. The binder content of modified soils is considered less than 2%, according to the related experiences in Australia (Jameson and Shackleton 2009). As such, since the low amount of cement itself might be sufficient for modification of CRB material, thus fly ash was also considered in binder composition to serve as filler and to lower the porosity of mixtures. Results of RLT, UCS, shrinkage, permeability and Particle Size Distribution (PSD) tests are presented first to initiate the discussions about aforementioned aims.

2 Experimental Program

This program included the study of the CRB as parent material and six different batches of modified CRB material with 2% binder content. The binders of the mixes had cement from 0.5% to 1.5% and fly ash from 1.5 % to 0.5% in a way that the sum of cement and fly contents was 2% in total. In other word, fly ash was included in the binder composition to keep total binder content equal to 2% for each trial mixture.

2.1 Particle Size Distribution & Plasticity Indexes

The Particle Size Distribution (PSD) and Atterberg limits of CRB material required to be accordance with MRWA Specification 501 as shown in Figure 1 and Table2. The test methods were undertaken according to AS 1289.3.6.1-2009 for PSD and AS 1289.3.3.1 for plasticity index of a soil.

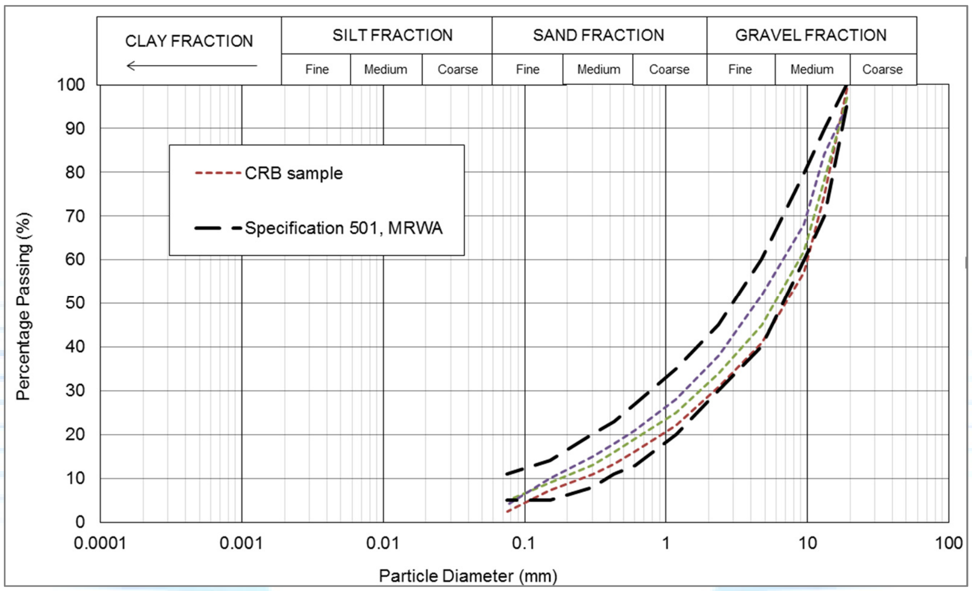

Figure 1 Particle Size Distribution of CRB material

According to Figure 1, it seems that CRB material has less than 5 % passing of 0.075 mm sieve and meets the specification for base course material.

Table 2 Atterberg limits of CRB material

Test type	Results	Specification Limits	Test method
Liquid Limit	20.2%	25 %	AS 1289.3.1.2
Plastic Limit	18.3%	--	AS 1289.3.2.1
Plasticity Index	1.9%	---	AS 1289.3.3.1
Linear Shrinkage	1.7%	0.4% -2 %	AS 1289.3.4.1

2.2 Compaction

The compaction test of CRB material was to obtain the Optimum Moisture Content (OMC) and maximum dry density (MDD) for further tests. The test was according to standard AS 1289.5.1.1, Maximum Dry Density, for the standard Proctor compaction procedure as suggested by Austroad, Part 4D (Andrews, 2006). The tests results are described as Figure 2

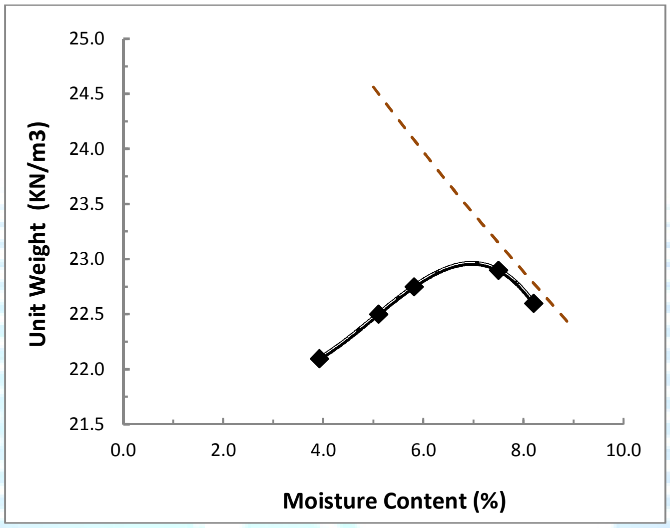

Figure 2 Compaction test results of CRB material

Similar tests on modified CRB mixtures showed that the Maximum Dry unit weight is about 23.4 KN/m^3 and the OMC is 6%.

2.3 Permeability

The permeability tests were undertaken to evaluate the drainability potential of CRB materials in compacted condition. The standard test method, AS 1289.6.7.1, was utilised using the constant head method. The final results are presented in Table 3

Table 3 Permeability tests results of CRB material

Test No.	K (m/s)	Hydraulic Gradient
1	1.55E-07	1.2
2	1.48E-07	1.5
3	1.24E-07	1.8
Avg	1.36E-07	-

Where K is permeability coefficient. According to Table 3, the mean value of results is about 1.00E-07 (m/s) which indicates low permeable characterisitic of CRB material. These permeability coefficients are in line with Austroad philosophy about increasing the permeability of materials with increasing depth of pavement layers as noted by Jameson (2008). However above permeability coefficients are for poor drainage materials according to Terzaghi et al. (1996).

This implies that CRB material cannot drain excess water from unexpected saturations, which provide a undesired condition for the assumed design parameter of material.

2.4 Unconfined Compression Strength (UCS)

The standard test method of AS 1141.51—1996 was used here for modified CRB material. It started with mixing of CRB material and binder in dry condition. Each test specimen was compacted using standard proctor effort in three layers of approximately equal thickness as suggested by Jameson and Shackleton (2009) in Austroad designs. After 28 days curing, the samples were soaked in water for about 5 hours before UCS testing with loading at a rate of 1 mm/ min. All of the tests included duplicate cylindrical samples; the averages of results are reported as Figure 3.

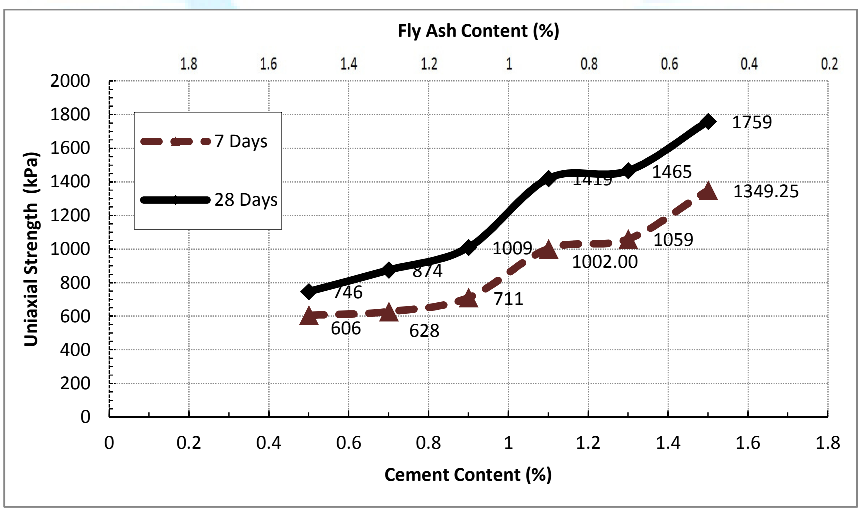

Figure 3 UCS results of different batches of CRB mixtures

In each test, the peak loads of bell like strength curves were considered as UCS corresponding to failure compressive strain. The evaluations of failure curves are summarised in Table 4.

Table 4 Summary of UCS test results

Batch	Cement Content (%)	Fly ash Content (%)	7days			28days		
			Elasticity Modulus (MPa)	Failure Strain (%)	UCS (MPa)	Elasticity Modulus (MPa)	Failure Strain (%)	UCS (MPa)
S1	0.5	1.5	174	3.2	606	220	3.2	746
S2	0.7	1.3	191	3.6	628	230	3.6	874
S3	0.9	1.1	200	3.8	711	240	4.6	1009
S4	1.1	0.9	266	4.5	1002	350	4.0	1419
S5	1.3	0.7	316	3.2	1059	400	4.0	1465
S6	1.5	0.5	406	3.6	1349	404	4.2	1759

In Australia, the design of stabilised material can be determined according to the Austroad Guide, part 4D (Andrews, 2006), based on UCS test results. In Figure 4, all of the considered binder compositions except batch S6, can fall into the UCS limits suggested by Austroad for modified materials.

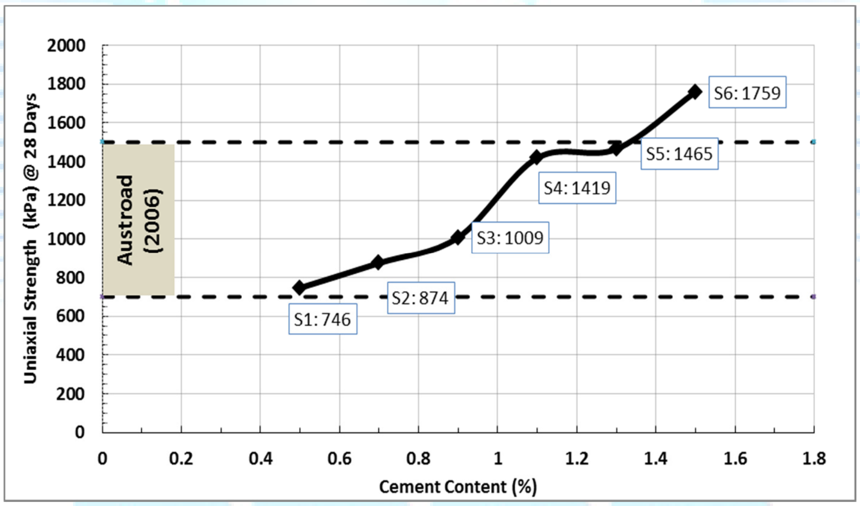

Figure 4 UCS results in 28 days within Austroad limits

In a similar manner, It can be seen in Figure 5 that the other batches like S1, S4 and S5 are located in the boundaries or outside of the limits suggested by MRWA in design guide No.9 (MRWA 2012).

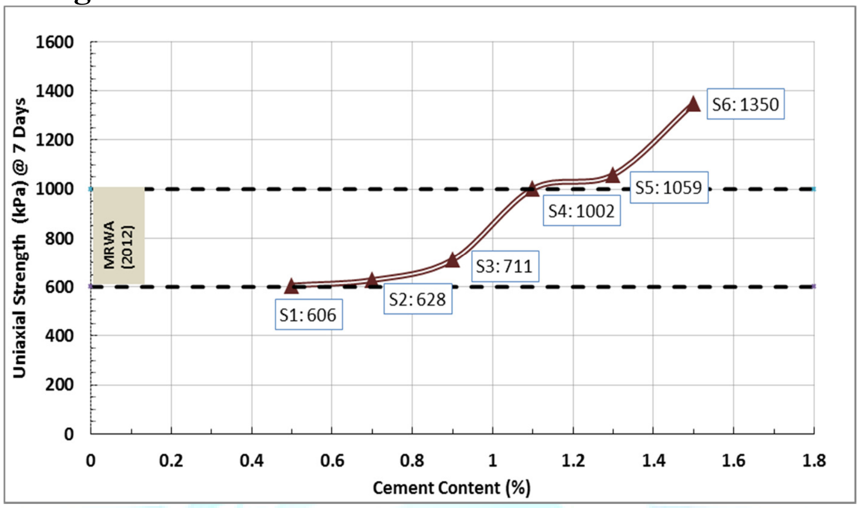

Figure 5 UCS results in 7 days within MRWA limits

Therefore, only batch S3 was considered here as suitable modified CRB mixture to be compared with HCTCRB. It should be noted that HCTCRB is rehydrated after compaction leading to bound behaviour with development of micro cracks under traffic.

2.5 Modulus of Resilient (Mr)

This parameter of modified material is important in mechanistic-empirical design procedure. It was initially introduced to characterise the elastic response of the subgrade soil material against repeated loads and to calculate the deformations of pavements (Seed, Chan et al. 1962).

This parameter takes into account the permanent deformations during repeated loads. Figure 6 shows the straining of specimens during this type of loading. In repeated loadings, plastic strains decrease by progress of load repeats. The strains tend to be recoverable after 100 to 200 repetitions.

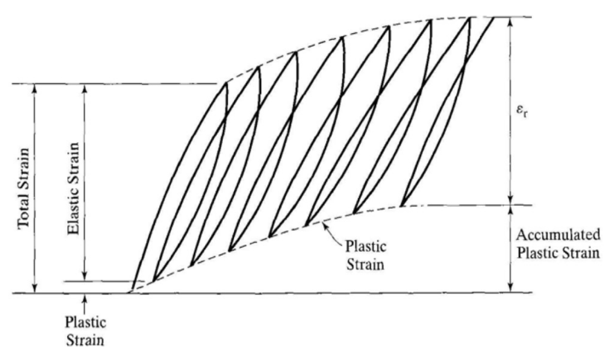

Figure 6 Strains under repeated loads (Huang, 2004)

Resilient modulus is defined based on the recoverable strain as

$$Mr = \frac{\sigma_d}{\varepsilon_r} \tag{1}$$

In which σ_d is the deviator stress in a triaxial compression test. This stress is small in such an extent that the test can be considered as non-destructive (Huang, 2004). However, one of the common models for describing the behaviour of resilient modulus in different stress levels, is called K-θ model as presented in equation (2) by Rada and Witczak (1981)

$$Mr = K_1.P_a.\left(\frac{\theta}{P_a}\right)^{K_2} \tag{2}$$

Where θ is bulk stress (kPa), Pa is atmospheric pressure (%), K_1 and K_2 are constant coefficients. The samples were prepared at the 100% standard Maximum Dry Density (MDD) with 100% OMC of each batch at standard cylinder moulds with 100 mm in diameter and 200 mm in height according to AG: PT/053 - Austroad(2007). The standard compaction method was utilised by dividing into 5 layers, using 2.54 kg rammer with 305 mm height of drop and 25 blows for each layer. After compaction, two specimens of each mix were kept in the mould for 1 day at room temperature before demoulding and were cured in the plastic bags for 28 days.

Two specimens of each batch were tested in dry and saturated conditions to compare the effect of moisture content changes on resilient modulus. Drying was done at 70^{O}C to keep moderate thermal changes on specimens. The specimens were tested in drained condition by Universal Testing Machine UTM-14P (14 kN Pneumatic) equipment with a digital data unit. Each specimen was subjected to 66 stress stages; the first stress stage consists 1000 cycles of pre-conditioning; the next stages, each consist of 200 repeated loading/unloading cycles of 66 stress conditions The repeated vertical loadings had a rectangular block wave waveform as Figure 7

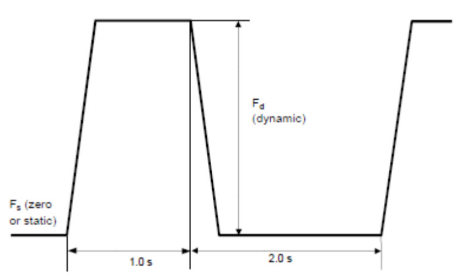

Figure 7 Illustration of loading waveform in AG:PT/053 - Austroad(2007)

Cyclic deformations were measured by dual linear variable differential transformers (LVDTs). Figure 8 to Figure 11 as typical results of this test for batch S1 on dry and saturated condition. The power regression based on K- Θ model is also shown in these figures.

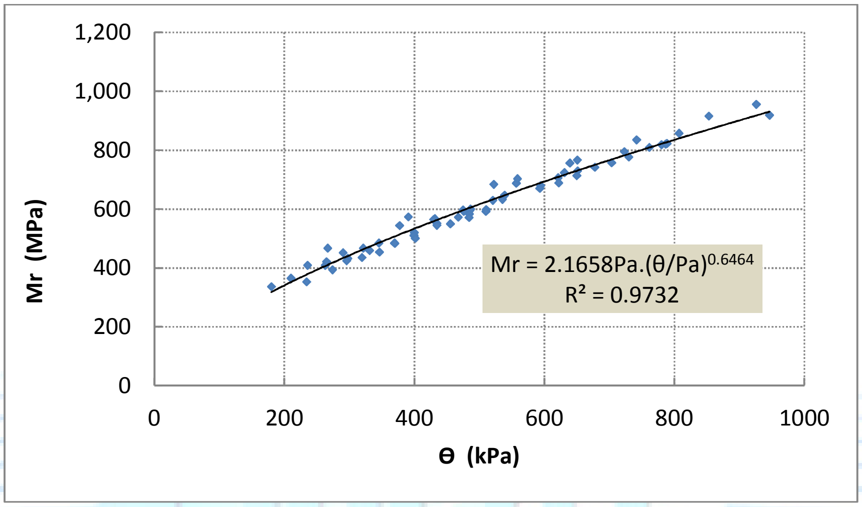

Figure 8 Result of RLT test, Batch S3 (Dry)

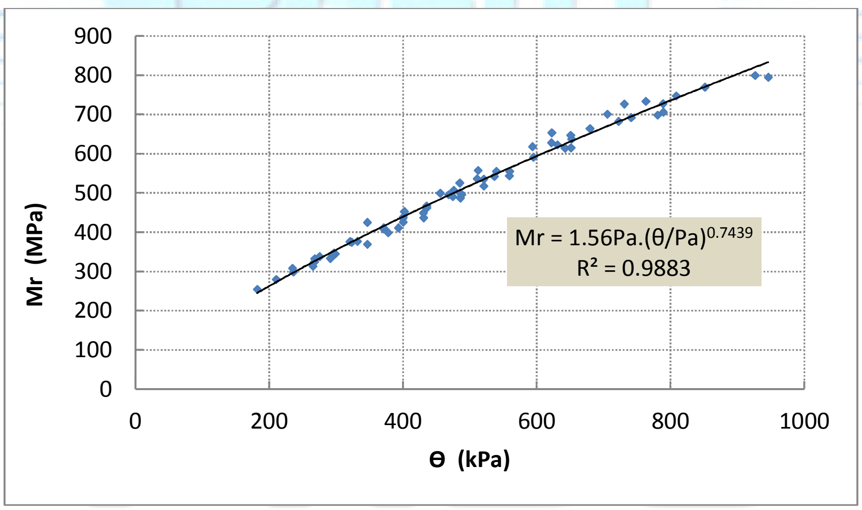

Figure 9 Result of RLT test, Batch S3 (Saturated)

Like other parameters in soil mechanics, the resilient modulus depends highly to moisture content. Figure 11 clearly shows the effect of saturation on the deformation capacity of the material. The axial deformations nearly become doubled while saturation which can cause undesired cracks for the performance of pavements.

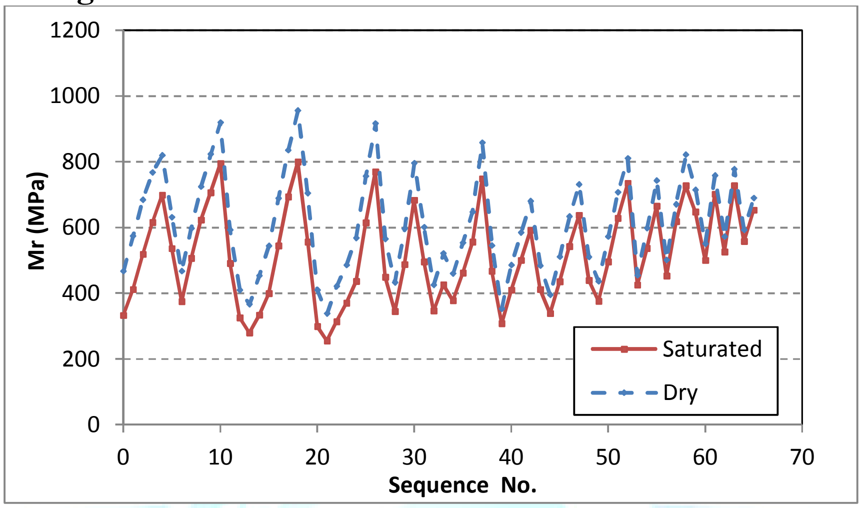

Figure 10 Resilient modulus variations, Batch S3

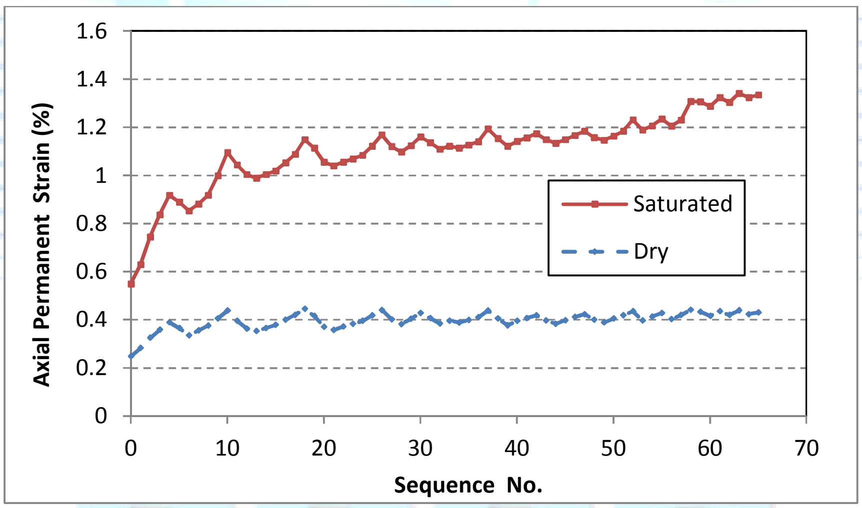

Figure 11 Permanent Axial Strain variations, Batch S3

The bulk stress in pavement layers depends on thickness of layers and load stresses. There are different recommendations as Table 5 for reporting purposes of resilient modulus of base course material according to popular pavement configuration by each reference.

Table 5 Suggested stress condition for base course material

References	Pavement Layer	Confining Stress σ_3 (kPa)	Deviatoric Stress σ_d (kPa)	Bulk Stress θ (kPa)
Rada and Witczak (1977)	Granular sub base	-	-	68.9
Rada and Witczak (1977)	Granular base	-	-	137.9-275.8
NCHRP 1-28A	Granular base/ sub base	34.5	103.4	

Note: Bulk stress $\theta = \sigma_1 + \sigma_2 + \sigma_3 = 3\sigma_3 + \sigma_d$

Similar recommendations, like above tables, upon the finite element analysis of granular pavements has provided by Austroad for mechanistic method as below (Vuong et al., 2007)

- Mean stress of 300 kPa, shear stress of 212 kPa for base course layer.
- Mean stress of 150 kPa, shear stress of 141 kPa, for sub base layer.

In WA, the vertical stress of 300 kPa and horizontal stress of 50 kPa is more popular and has been by Butkus (1994) , Cray (1992) and Butkus and Lee Goh (1997) in the related technical reports. Hence, two sets of stress levels were considered here, according to NCHRP 1-28A and MRWA reports.

This make is it easy to process and compare resilient modules of different batches as shown in Figure 12 and Figure 13. At both levels of bulk stress, the resilient modulus (Mr) has an increasing trend with cement content in both saturated and dry conditions of CRB modified material. The proportion of Mr in dry over saturated condition has a rising norm, which signifies the saturation issues in mixtures with higher cement contents.

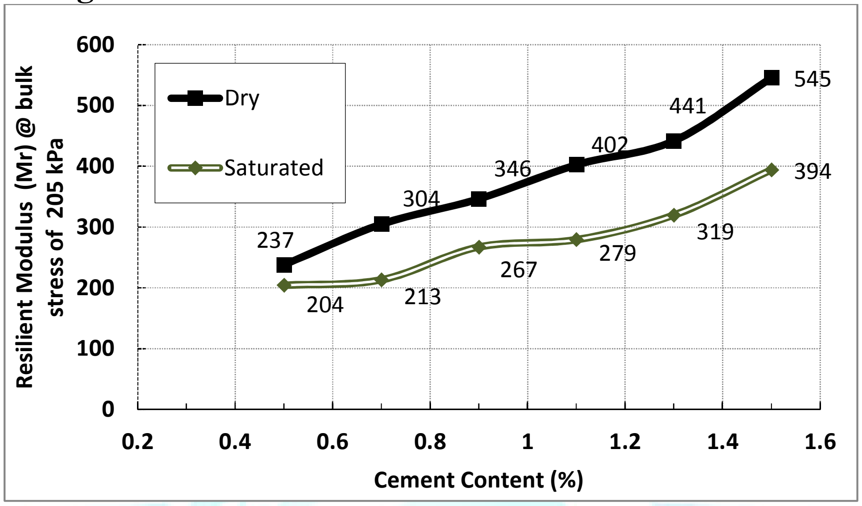

Figure 12 Resilient Modulus versus cement content of batches, θ=205 kPa

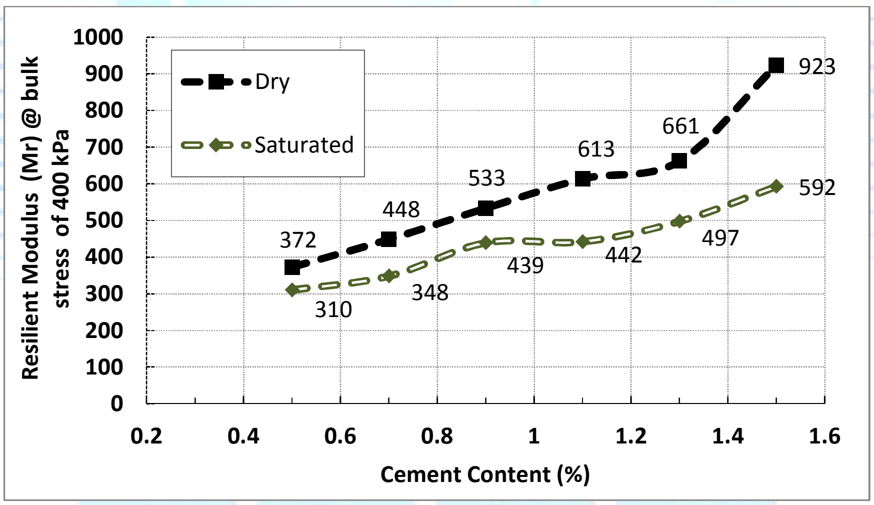

Figure 13 Resilient Modulus versus cement content of batches, θ=400 kPa

2.6 Shrinkage (Length Changes)

The drying of cement-stabilised materials is always associated with cracking due to shrinkage. Despite some methods for shrinkage testing of soils or concrete, there is no similar standard test method for stabilised soils (Chakrabarti & Kodikara. Jayantha, 2004). However, the standard test method of AS 1012.13-1992 for concrete is used for cement treated CRB material. In this test, beam specimen with 75x75x280 mm in size is used. The maximum size of particles required to be adjusted to consider the limit of specimen/maximum aggregate ratio of less than 5. Thus, the aggregates over 9.7 mm sieve size were separated, removed and replaced by fine aggregates. The reading of beam specimen was performed by a horizontal comparator to measure the changes in horizontal plane parallel to the compaction direction as Figure 14

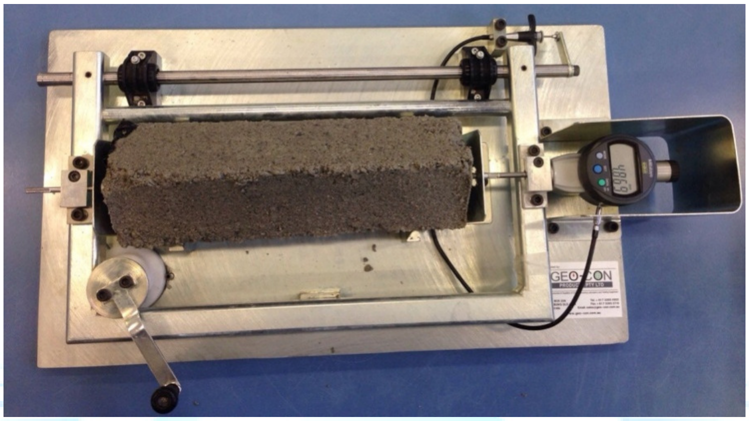

Figure 14 Horizontal Comparator (Cat. No. 55-C0115/9.Con)

The results of length measurements for specimens of batch S3 and batch S2 are presented in Figure 15.

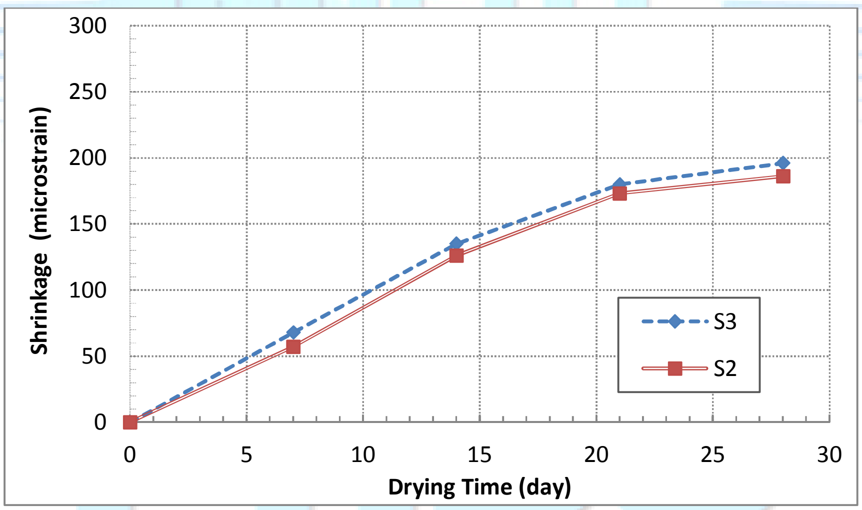

Figure 15 Shrinkage test results

In Figure 15, the shrinkages are less than 310 micro strain for coarse material as suggested by George (2002) upon experiences by Portland Cement Association (PCA). The aim of this limit is to have numerous minor cracks rather that few wide cracks. Thus, cracks cannot provide pathways for water infiltration or destruction of cement treated layers.

3 Discussion

3.1 Dry-back concept

During construction of granular materials, some delays after compaction can provide the dry-back condition or adjusted the moisture content of the base course layer before the sealing stage. This assumed in-service moisture condition is to limit the permanent deformation and the stiffness of the granular material. But it should be considered that the wind or sun drying methods affect more the surficial zones of compacted layer and also there is not a significant certainty for moisture content endurance during the service life of pavement.

Kodikara and Chakrabarti (2005) found that moisture diffusion of cement treated CRB material may take weeks from saturation to semi wet condition. Similarly, Strohm et al. (1967) has noted that base course material with less than 5% fines content shows zero drainage even after 24 hours.

This subject can be evaluated by empirical drainability potential, also. Drainability potential in terms of drainable porosity, nd, is defined as a reduction of the volumetric water content of soil from its saturated state, to the lowest content achievable by gravity forces.

In practice, the drainable porosity can be estimated empirically using the graph that is given by Lyle (1980) in a report entitled Highway Sub drainage Design. The algebraic form of that graph is:

$$n_d = 0.0355k^{0.235} \quad (3)$$

Where k is the saturated permeability of soil (m/day) and n_d is drainable porosity (%). This equation is based on the measured values of n_d and k for a variety of soils with different densities. Alternatively, the outcome of the empirical method can be presented by the degree of saturation (S) which is more sensible in practice. Based on the volume and void relationships in soil mechanic, it can be shown that the minimum degree of saturation in a gravity drained case, as S_{min} is related to n_d in equation (4):

$$S_{min} = 1 - \frac{n_d}{n} \quad (4)$$

Where S_{min} is the minimum degree of saturation after draining

n = Total porosity (%)

Thus, considering the permeability coefficient (K) of about 1.00E-07 (m/s) and total porosity of 25% for CRB material, the minimum degree of saturation in gravity drainage condition would be about

95%. This range of saturation or moisture content is far beyond the RMC of 60% or 85% as suggested by Main Roads of Western Australia (MRWA) (2012) for this material.

Therefore, it can be inferred that assumption of the semi-dry moisture condition of base course material is not a reliable method for design of pavements.

3.2 Modulus of resilient

Khobklang et al. (2013) utilised other form of K-θ model for HCTCRB material and suggested below equation.

$$M_r = k_1\theta^{k_2} \qquad (5)$$

Where the regression constants k_1 and k_2 were around 10.15 and 0.6637 respectively. Thus, the resilient modulus of HCTCRB material can be considered about 350MPa and 540 MPa corresponding to bulk stresses assumed by NCHRP and MRWA. Comparison of these values with the resilient modulus result of batch S3 shows that HCTCRB is stiffer than even dry condition of modified CRB material.

3.3 Shrinkage

Chummuneerat et al. (2012) studied the shrinkage of treated CRB material with cement contents from 2% to 6%. They found that the shrinkage of these materials varies from about 350 microstrain to 400 microstrain which is nearly twice of results for batch S3 and batch S2. In the absence of published records for shrinkage of HCTCRB, it can guess that it should have more shrinkage than low-cement modified CRB since it is stiffer and stronger.

4 Conclusion

This research reviewed the treatment methods of CRB material related to its moisture susceptibility. For this reason, the reliability of dry-back method was discussed and it was shown that it is not a conservative approach based on drainabilty potential studies.

In addition, the results of laboratory tests on low-cement CRB materials show that it can be considered as a substitute for HCTCRB material apart from construction difficulties of HCTCRB material. These findings confirm the better performance of this material rather than HCTCRB in practice.

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