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Characterisation of some crushed rock base materials from Queensland

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Summary: This paper summarises recent investigations into characterisation of crushed rock materials carried out by the Geotechnical Group at Department of Main Roads, Queensland, on road projects across the state. Some of these investigations have been prompted by premature base failures. Empirical specification tests, for the coarse and fine aggregates; monotonic static triaxial tests, Texas triaxial tests, repeated load triaxial tests, direct shear tests (300mm and 60mm) and wheel tracker tests have been used to varying degrees in the these investigations. Resilient moduli, plastic deformations and moisture sensitivity with varying degrees of saturation levels have been the main focus in these investigations and only the results from repeated load triaxial and wheel tracker are discussed in this paper.

INTRODUCTION

In recent years the Department of Main Roads, Queensland (QDMR) has experienced a number of premature failures of crushed rock pavements (with sprayed seal surfacing). Such failures have usually manifested through inadequacies in the base course leading to rapid failures, occurring from several hours to several days after opening to traffic or even during construction. Most of these have been attributed to excess construction moisture. It is pertinent to note that the road base materials involved in such failures have conformed to Departmental material specifications on most occasions and in a few situations may even have complied with construction moisture levels stipulated.

The material properties currently specified by Road Authorities (e.g. QDMR MRS 11.05) include criteria such as aggregate strength, Atterberg Limits etc. However, none of these are a direct measure of resistance to rutting caused by repeated loading. These simple specification tests although would contribute towards material quality assessment, but fail to clearly highlight performance risks. Such limitations in knowledge can be partly mitigated by local knowledge of materials and construction practices.

Performance based tests such as the Repeated Load Triaxial (RLT) and the Wheel Tracker (WT) offer additional insights into material behaviour. QDMR have been using these new technologies in source assessment/ forensic/ material development roles. This paper summarises crushed rock characterisation methodology as undertaken by QDMR in general, and discusses some of the more recent results in particular, with a view to establishing the effectiveness of these new technologies in QDMR practice.

SOME BACKGROUND ON DRAINAGE CONSIDERATIONS IN CRUSHED ROCK BASES

Most of the studies reported in the literature on the shear behaviour of crushed rock materials had been carried out under drained conditions (Brown (1974), Barksdale (1972)). Although Brown (1974) shows the results of an undrained test on a well graded crushed rock where the undrained plastic strain is nearly six times that of a corresponding drained test, it was concluded that the drained condition was more realistic. On the other hand, Barksdale (1972) states that densely-graded aggregate bases, (Queensland sources would be similar), by virtue of their lower permeability, would be more susceptible to pore pressure build-up in the field leading to undrained conditions. In the present study, in view of the rapid premature pavement failures reported and in view of the high rainfall experienced along the eastern coast of Queensland, emphasis is placed on the **undrained** plastic response of the crushed rocks.

QDMR CRUSHED ROCK DATABASE

The source of the data used in this paper is based on the test results stored in the database which contains data spanning more than a decade. It comprises investigation results on crushed rock and gravels pertaining to the various product types defined in MRS11.05 with data on type 2.1 product being the most common. The specification requirements as per MRS11.05 can be divided into three groups: defining the requirements for coarse component (37.5mm – 2.36mm), fine component (0.425mm – 0.075mm) and the whole assemblage. The coarse component standards are separated into source material groups based on the geological origin as igneous (acid/intermediate/basic as separate groups), metamorphic, and finally sedimentary and duricrust grouped together. The fines (minus 2.36mm fraction) are sourced either from outside e.g. a loam, or are the by-product of the parent rocks. All test procedures reported are generally in accordance with QDMR Materials Testing Manual or Australian Standards and the majority of the testing has been carried out at the Pavements, Materials and Geotechnical Laboratories at Herston.

Test Equipment: A feedback controlled universal testing system supplied by Industrial Process Controls is used by QDMR to carry out repeated loading triaxial testing (RLT) of unbound granular base materials. The elements of the system broadly conform to AS1289.6.81.

Currently a rectangular waveform is used to apply the total vertical stress (750kPa: the current design tyre pressure) to specimens. The frequency of the load pulse is 0.33Hz. The confining stress is applied to the specimen using water as a confining medium and held constant during testing to simplify the procedure. The deviator stress is determined from the load measured by an external load cell of 6kN capacity with ±1% accuracy. During unloading the deviator stress is reduced to a residual value of 10kPa. Pore water pressure is monitored with a Druck pressure transducer mounted at the base. Pappin et al (1992) have shown with saturated undrained tests on limestone that pore pressures measured on the base are not seriously in error for loading rates up to 1 Hz. Axial deformation of the sample is measured with three linear variable displacement transducers (LVDTs; 2 Nos. 5mm and 1 No. 20mm) mounted externally on top of the cell by monitoring movement of the loading plunger. Lateral deformation of the sample is not measured.

The WT, originally designed for asphalt, has been recently adapted by QDMR to suit the testing of granular materials. In its current form, a rolling wheel with a 50mm by 18.5mm footprint undertakes channelised bi-directional travel, as opposed to the unidirectional loading that occurs under channelised vehicular traffic. It applies the design tyre pressure of 750kPa at 0.69 Hz frequency and is housed in a temperature controlled cabinet. Rut depth and the cycle count is measured electronically under software control. It has been shown (Brown & Chan, 1996) that the bi-directional rut test can be significantly more damaging than the unidirectional test that reproduces the field conditions.

The RLT is able to test specimens compacted at different moisture/ densities under varying axial and radial stresses and thus can simulate different elements in the road base in terms of stress level. It enables the measurement of pore water pressure under undrained conditions. However, the rotation of principal planes that occur under a moving load cannot be simulated and plastic strain is sensitive to rotation of principal planes. Therefore, it can be expected that the RLT gives a lower bound solution whilst the WT returns an upper bound. Hence the RLT and WT complement each other.

Sample Preparation: Materials passing the 37.5mm sieve are used for road base materials. However the fraction passing the 19mm sieve is used for compacted specimens. Samples received are fractionated into four size ranges (37.5mm – 19mm; 19mm – 9.5mm; 9.5mm – 4.75mm; and finer than 4.75mm) and recombined from these constituent size ranges to achieve a required specimen size for a test. Most compacted specimens conform to the 100% Standard Proctor Maximum Dry Unit Weight using dynamic compaction unless otherwise indicated, but at different degree of saturation levels (DoS). RLT samples have a nominal height of 200mm and a diameter of 100 mm. The WT samples are 300mm by 300mm with 100mm thickness, also generally compacted to Standard Proctor unless otherwise indicated, using a slab compactor.

Test Details: Permanent strain testing in the RLT is now carried out under a cell pressure of 125kPa and a pulsating deviator stress ($\sigma_1 - \sigma_3$) of 625kPa under undrained conditions. To minimise membrane penetration effects double membranes (each 0.3mm thick) are used for the RLT specimens with a bottom coarse porous stone (80kVA) to enable pore water pressure measurement. The triaxial cell has a bronze bushing without an O ring. Previous RLT results in the database correspond to a cell pressure of 125kPa and a deviator stress of 425kPa. and are not included in this paper. RLT testing is carried out in accordance with Australian Standard AS1289.6.8.1. The WT is performed in accordance with an "in-house" procedure.

DISCUSSION OF RESULTS

As mentioned, this paper will examine a representative cross-section of the more recent results [($\sigma_1 - \sigma_3$) of 625kPa] for discussion. However, compaction characteristics from the database are also presented.

The degree of saturation (DoS) has been primarily used to correlate performance, as the shear behaviour of an unsaturated material is heavily dependent on DoS (for example Toll, 1990). The liquefaction resistance of unsaturated sand has also been shown to increase significantly with decrease in DoS (Yoshimi, et al, 1989).

QDMR Database: Compaction Characteristics: The discussion begins with the laboratory compaction characteristics of the crushed rocks in the QDMR database. Figure 1 shows the Standard Proctor Maximum Dry Density and OMC of the samples presented in volumetric co-ordinates i.e. porosity versus volumetric moisture content (volume of water/total volume). In this framework the degree of saturation lines are linear and independent of particle specific gravity (Prashanth et al, 1998). This enables the comparison of widely different crushed rock materials. These materials shown in Fig. 1 generally conform to the MRS11.05 'C' grading (see Fig. 2). As seen in Fig. 1, the bulk of the crushed rocks have their Standard Proctor OMC's greater than about 80% DoS. The permeabilities of some of the materials tested range between 10^{-5} m/s to 10^{-8} m/s using the constant head permeability test. These materials predominantly classify as GW/GC/GM in the Unified Soil Classification.

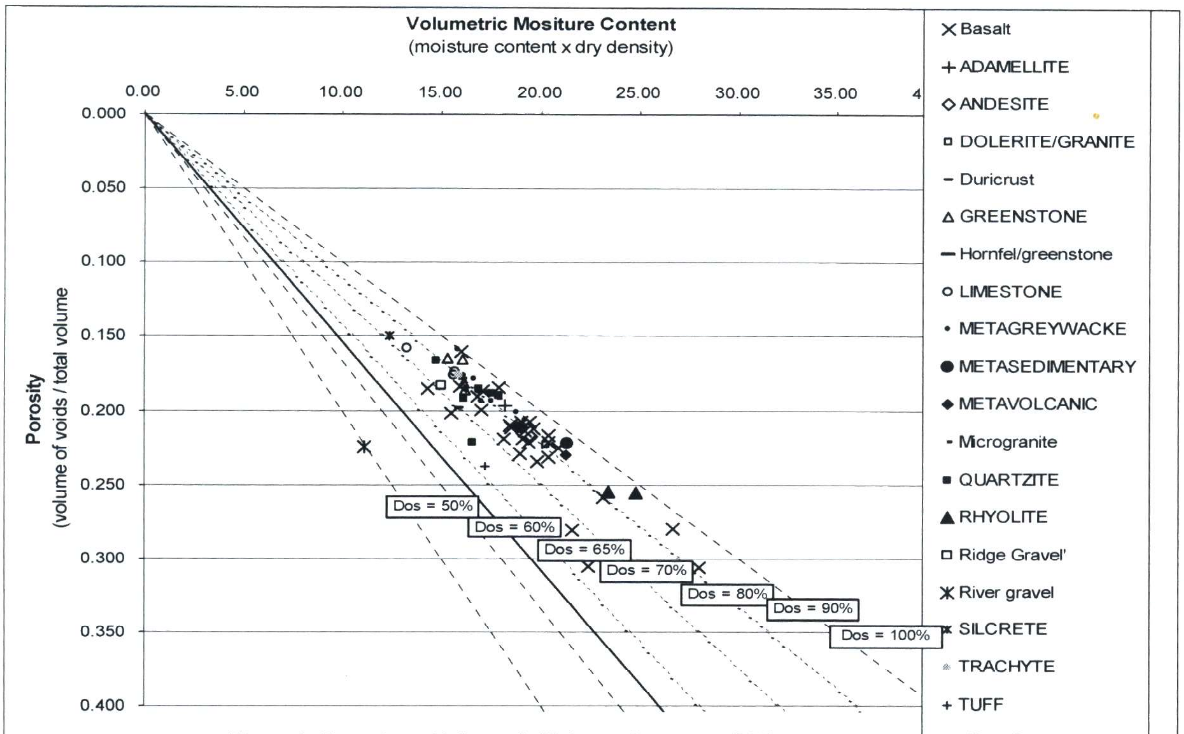

Figure 1. Porosity vs Volumetric Moisture Content at OMC and MDD

Selected Materials Study: Specification Properties: The second part of the discussion deals with some specific materials of more recent investigation ($\sigma_1 = 750$ kPa), with Table 1 showing their basic material properties as per MRS 11.05. They are broadly representative of the major lithologies used as road base materials in the QDMR network. The specification tests fall into three categories: tests to assess the suitability of the coarse component (but different tests use different size ranges); tests to assess the fines

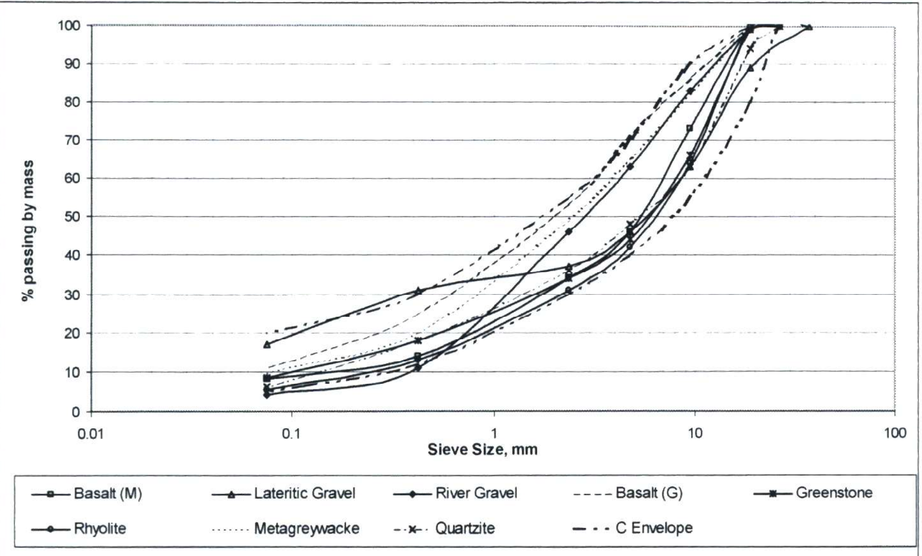

Figure 2: 'C' Grading Envelope

component; and the CBR test, which is the only test that examines the entire grading curve (at least the minus 19mm material) in the compacted state. The CBR test, however, has not found wide acceptance for granular materials as an indicator of moisture sensitivity. In the current QDMR specifications the size fraction between 2.36mm and 0.425mm does not get subjected to any assessment except in the CBR test.

Table 1. Specification Testing Results.

Source Rock Group	Basic Igneous	Basic Igneous	Acid Igneous	Acid Igneous	Metamorphic	Metamorphic	Metamorphic	Sedimentary
Rock Type	Basalt (M)	Basalt (G)	River Gravel [1]	Rhyolite	Greenstone	Metagreywacke	Quartzite	Ridge Gravel [1]
Component Property								
Coarse								
Q205B 10% fines (wet) kN (min)	341	200	195	182	214	191	167	
Q205A/C Wet/Dry Variation (%) (max)	10	29	8	12	30	22	9	N/A
Q208B Degradation Factor (min)	70	53	75	29	54	72	44	
Q201A Flakiness (%) (max)	14	16	14	22	24	20	23	
Q109B APD	2.895	2.963	2.665	2.619	2.975	2.725	2.66	2.683
Q214B Water Absorption coarse	1.45	N/A	0.93	4.06	0.79	0.35 - 0.54	0.80	N/A
Fines								
Q104A/D LL (%) (max)	19.2	21.8	20.6	23.0	20.4	18.4	17.8	21.2
Q105 PI (%) (max)	0	2.6	1	0.6	2.2	0.4	0.2	9.4 [2]
Q106 LS (%) (max)	1.2	3.4	2	1.6	2	2.2	1.4	6.4 [2]
% sand (2.36-0.075mm)	25.8	42	41.8	25.5	25.4	29	18	20
% passing 0.425mm	14	25	11 [2]	13	18	20	18	31 [2]
Ratio 0.075/0.425	0.57 [2]	0.45	0.36	0.42	0.48	0.5	0.36	0.55
PI x %< .425mm (max)	0	65	11	8	40	8	4	292 [2]
LS x %< .425mm (max)	17.2	85	22	21	36	44	25	199 [2]
Q109A APD	2.896	2.878	2.653	2.646	2.888	2.725	2.671	2.586
Grading								
Coefficient of Uniformity Cu = D_{60}/D_{10}	48	N/A	13.6	39	82	53	61	N/A
Fuller's 'n' value (19mm max particle size)	0.46	N/A	0.6	0.52	0.43	0.43	0.47	0.27
Unified Soil Classification	GW-GM	GP-GM	GW	GW-GM	GW-GM	GW-GM	GW-GM	GC
Compaction Conditions								
Q110A OMC (%)	8.6	7.4	5.4	12.7	6.7	7.2	8.3	6.6
Q110A MDD (t/m^3)	2.253	2.317	2.062	1.955	2.41	2.245	2.158	2.16
Q113A CBR (soaked)	135	96	105	84	78	120	90	
Q113A OMC (CBR)	8.4	6.8	N/A	11.8	N/A	6.2	7.6	N/A
Q113A MDD (CBR)	2.28	2.32		1.96		2.25	2.17	
APD Total Sample (t/m^3)	2.895	2.923		2.627	2.945	2.725	2.664	2.641
B Value @ approx 65% DoS	0.15	0.10	0.23	0.18	0.13	0.20	0.26	0.06

Notes:

(1) = Non conforming with QDMR 'C' grading; (2) = Non compliant with QDMR Standard Specification, MRS11.05; N/A = Not Available

The gradings of these materials are shown in Figure 2. As seen the ridge gravel (gap graded) and the river gravel do not conform to the MRS11.05 gradings. Except these two materials, the rest have Fuller n values generally in the optimum range for gradings that are attributed to produce maximum compacted dry density. (0.35 – 0.5; Brown & Chan (1996)) and these gradings have GW/GM/GC classifications

Resilient Stiffness: The behaviour of unbound granular materials is complex, because of their particulate nature. This complexity results from a combination of mechanisms including elastic deformation of particles; slip between particles, crushing of particles producing irrecoverable (plastic) strains and generation of pore pressure affecting inter particle behaviour.

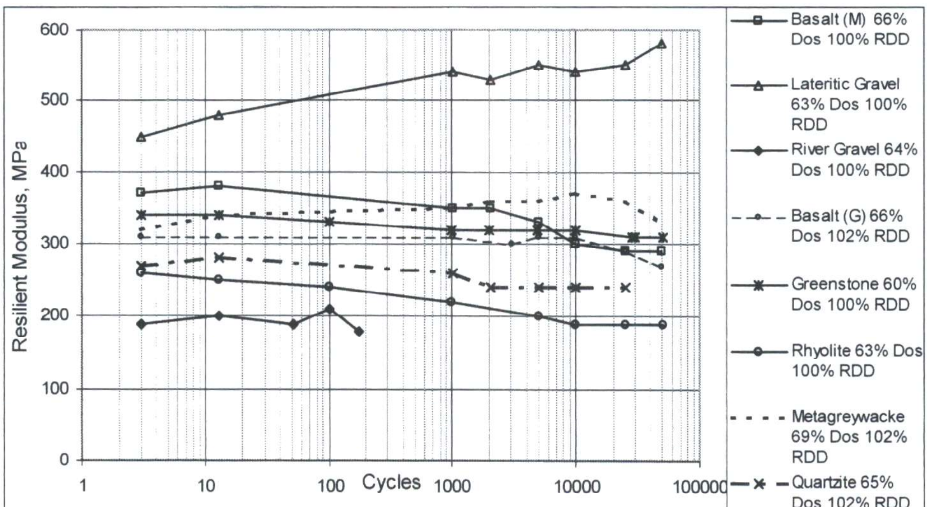

Figure 3. Resilient Modulus

Under cyclic loadings, these materials undergo resilient strains, which are recovered after each cycle, and permanent strains, which accumulate with the number of cycles.

Figure 3 shows the resilient modulus , M_r (deviator stress divided by resilient strain), with cycle count around the 65% DoS level in the RLT. The two 'non-standard' materials bound the moduli for the crushed rock. The river gravel, which is rounded, has the lowest moduli and has failed prematurely whilst the ridge gravel (gap graded) which is of lateritic origin has had the highest moduli with an increasing trend with cycle count contrary to crushed rock trends. Figure 4 shows the matric suctions measured with the filter paper technique on the lateritic gravels showing high matric suctions that would contribute to the high resilient moduli. The rapid decrease of matric suction with DoS is to be noted for this material. The matric suctions of crushed rock materials are low and typically less than 50kPa (for example the metagreywacke material listed in Table 1). No premature failures of this lateritic material from the semi-arid North West Queensland has been reported to-date in spite of its high PI (=9%) and high Linear Shrinkage (=6%) by conventional paving material standards.

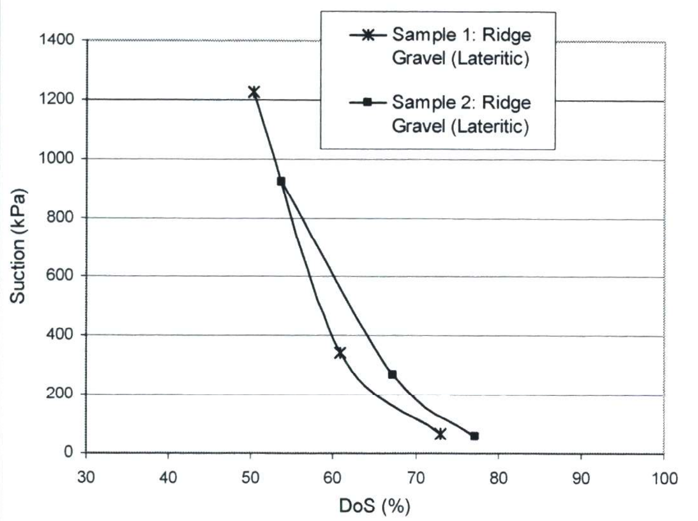

Figure 4: Matric Suction vs Saturation

Plastic Strain Response : Figure 5 shows the corresponding plastic strains measured in the RLT with cycle count. Similar to the resilient moduli, the two 'non-standard' materials again bound the plastic strain response. The rounded river gravel shows high plastic strain response with premature failure. On the other hand the lateritic gravel shows the lowest plastic strain i.e. the best rutting resistance. Additional testing to 1,000,000 cycles and beyond, and further testing at higher DoS levels may be necessary to establish whether this favourable rutting performance could be reliably sustained. As seen in Fig 5, materials with low weighted linear shrinkages (LS x% passing the 0.425mm sieve) tend to exhibit higher plastic strains; the river gravel with the highest plastic strain has a value of 22 whilst the next highest plastic strain is for the quartzite with a value of 25. Premature failures in the field have been more common with materials with low weighted shrinkage values.

Ishihara (1993), among others, has given some evidence to suggest that sands containing plastic fines exhibit higher resistance to liquefaction. Vuong (2001) has shown decrease in plastic strain with plasticity index. The plastic fines can contribute to strength through their matric suctions providing they are present in the right nature and quantities, without jeopardising the inter-particle direct contact. Such plastic fines with high initial suctions could be expected to prolong the development of positive pore water pressures under load and thereby play a role other than as mere inert fillers. This is supported by Skempton's (1954) B values (Table 1) for the DoS range reported in Fig.3 where the lowest was for the lateritic gravel (0.06) and highest for the river gravel (0.23). However, what optimum level of plastic fines ought to be present, needs to be investigated more carefully.

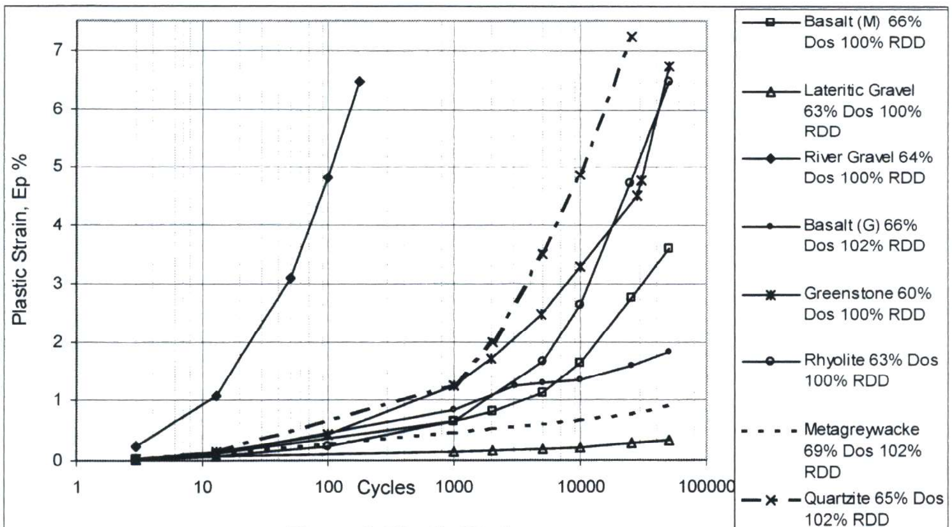

Figure 5. Plastic Strain

Figure 6 shows the plastic strains from the wheel tracker for four selected materials from Table 1 namely, metagreywacke, basalt(M), the ridge gravel and the quartzite, plotted along with the RLT results. The WT results generally mirror the RLT findings, and shows higher plastic strains. This is because the WT is a more onerous test as discussed earlier. The rounded river gravel (not shown in Fig. 6), which had the highest plastic strain in the RLT (as shown in Fig. 5), developed a plastic strain of about 15% within 10 passes in the WT. Based on the results shown in Fig. 6, the divergence between the RLT and WT appear to be more accentuated in the case of the quartzite. The ridge gravel shows a marked change in plastic strain between the RLT and the WT, however the

DoS level for the RLT is 63% but the DoS level for the WT is 70% and also the WT plastic strain has levelled off with increasing cycle count. The use of this dual methodology i.e. using the RLT and the WT as complementary tools has also been reported by Mundy and Dawson (2001).

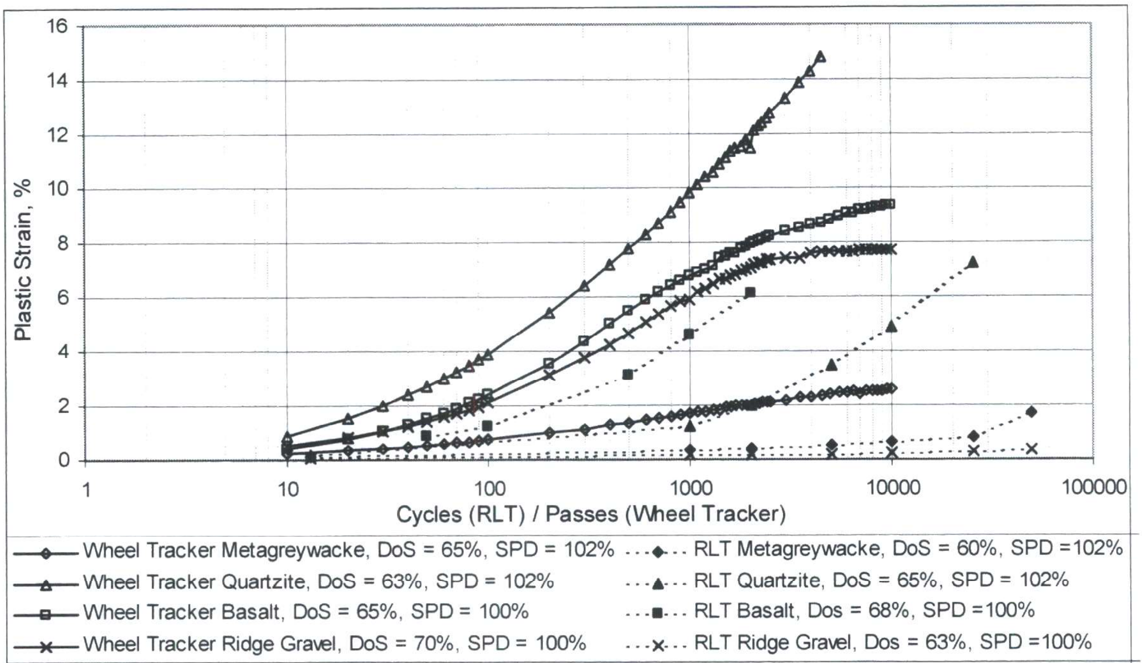

Figure 6: Comparison between Plastic Strains in RLT and WheelTracker

Material Ranking Issues: It appears from the foregoing that plastic strain is more sensitive to material properties than resilient modulus, (Fig .5). This is in agreement, for example, with Thompson (1991) & Vuong (2001). Werkmeister et al (2001) have adapted shakedown concepts for material ranking with plastic strains. The determination of an acceptable plastic strain is a difficult issue. QDMR currently adopts the tentative limits for plastic strains of 1.5% and 4% at 1,000 and 50,000 cycles respectively in the RLT. For the WT 2.5% strain at 10,000 passes is currently adopted. Preliminary assessment of shakedown criteria suggests that these limits may err on the high side. This issue is currently being investigated.

In view of the susceptibility of materials with low weighted LS values for excessive rutting and premature failure, caution is exercised with such materials and stricter construction DoS levels are usually recommended. Specification of a minimum weighted LS of 40 is currently under consideration.

The presented results clearly indicate the wide divergence of performance of crushed rock materials satisfying the conventional QDMR specifications. Performance based characterisation, for example by intercepting poor performers, enables the adoption of mitigation and development measures such as the specification of lower construction moisture levels; provision of pavement drainage with moisture sensitive aggregates; modification of the base at least in the top 100mm thickness e.g. cement modification in high rainfall areas where drying back may not be practicable; assessment of material under increased wheel loads ; assessment of the benefits of higher compaction levels ; assessment of non-standard materials (the performance assessment of the lateritic ridge gravel reported is an example); use of material in a lower layer e.g. as sub-base in view of the potential risk identified by these methodologies.

Construction Moisture Issues: QDMR has technical guidelines for DoS levels (QDMR, 1989) to be achieved prior to sealing. Currently a maximum DoS of 65% is stipulated prior to sealing. Given that a 1% change in moisture content is equivalent to about 10% to 15% in DoS in the dense states the base materials are placed, and the high sensitivity of plastic strain to DoS, the importance of accurate measurement of field moisture cannot be overstressed. As nuclear gauges are widely used for moisture measurement in the field in the back scatter mode, there is the potential for underestimation of moisture which could have disastrous consequences, unless frequent calibrations are undertaken. Another area of concern for construction moisture might arise from the use of coarse aggregates (generally of volcanic origin) with high water absorptivity e.g. rhyolite in the present study. Coarse aggregates with water absorption values generally in excess of 4% are treated with suspicion. However such materials are not common.

SUMMARY AND CONCLUSIONS

This paper has outlined the unbound granular characterisation methodology as practised by QDMR. The importance and usefulness of undertaking performance based characterisation (using such tools as the Repeat Load Triaxial (RLT) and the Wheel Tracker (WT)) has been discussed and these in turn have provided some insight into the understanding and management of geotechnical risk.

The application of soil mechanics principles through these technologies has enabled a basis for the demarcation of safe DoS levels for construction practice, approximate definition of plastic strain levels for material ranking and afforded an appreciation of the role of plastic fines in the mechanical behaviour of crushed rock with guidance for material development.

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