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Performance of some unbound roadbase materials from Queensland

Performance de certains matériaux granulaires de base du Queensland

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ABSTRACT

This paper summarises recent investigations into characterisation and performance of unbound roadbase materials carried out by Main Roads, Queensland (QDMR), on road projects across the state. Performance based tests such as the Repeated Load Triaxial (RLT) and the Wheel Tracker (WT) are the primary tools which are increasingly used by QDMR to overcome the limitations of simple specification type tests. This paper shows the inadequacy of current specification tests to rank material performance. The performance based tests show that the properties of the coarse aggregate alone are inadequate for sound performance; enable the contribution to mechanical behaviour by plastic fines with high matric suction to be assessed; further, and facilitates ranking of material behaviour. Simple shakedown analyses undertaken yield similar material rankings. Finally, some materials from the performance based characterisation are compared with Falling Weight Deflectometer (FWD) in-service pavement performance data.

RESUME

Cet article fait le bilan des enquêtes récemment menées par Main Roads, Queensland (QDMR) pour établir la caractérisation et la performance des matériaux de base sur des projets routiers répartis dans tout l'État. Dans le but de dépasser le type d'essais de simple caractérisation, QDMR a fait appel avant tout à des essais axés sur la performance tels que les essais triaxiaux à chargements répétés (RLT) et les essais d'orniérage (WT). Cet article souligne l'insuffisance des tests de caractérisation actuels en matière d'évaluation de la performance des matériaux. Les méthodes de caractérisation RLT et WT montrent que les propriétés du gros granulat ne suffisent pas, à elles seules, à assurer une bonne performance; ces méthodes permettent d'évaluer l'impact des particules de plastique à haute succion matricielle sur le comportement mécanique ; ce dispositif facilite en outre la classification du comportement mécanique. Les analyses préliminaires ont donné des classifications analogues des matériaux. Enfin, certains matériaux de l'enquête axée sur la performance sont comparés aux données relatives aux chaussées en service (FWD).

1 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). Most of these have been attributed to failures of the base material due to excess construction moisture. Performance based tests such as the Repeated Load Triaxial (RLT) and Wheel Tracker (WT) are often used by QDMR to better understand crushed rock behaviour. This paper shows the inability of current specification tests to rank material performance. It investigates the role of plastic fines to mitigate moisture sensitivity. The RLT and WT findings are compared with a FWD in-situ structural assessment of a motorway section.

2 TESTING PROGRAMME

This paper is based on test results stored in a database that contains data spanning more than a decade.

Most of the studies reported on crushed rock materials had been carried out under drained conditions e.g. Brown (1974). However, Barksdale (1972) states that densely-graded aggregate bases, (Queensland sourced base materials 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 pavement failures reported and the high rainfall experienced along the populated eastern coast (in excess of 2000mm), emphasis is placed on the **undrained** response.

A feedback controlled RLT testing system broadly conforming to AS1289.6.81 with a rectangular waveform generally of 0.33Hz is used to apply the deviator stress to specimens. A constant confining stress is applied to the specimen using water. Axial deformation of the sample is measured with three linear variable displacement transducers mounted externally. Permanent strain testing in the RLT was carried out under a cell pressure of 125kPa and a pulsating deviator stress of 625kPa under undrained conditions.

The WT used has a rolling wheel (simulating the rotation of principal stresses) with a 50mm by 18.5mm footprint undertakes channelised bi-directional travel (shown to be more damaging, Brown & Chan 1966). It applies the QDMR design tyre pressure of 750kPa at 0.69Hz and is performed in accordance with an "in-house" procedure. The RLT and WT play complementary roles.

Materials passing the 37.5mm sieve are used for roadbase materials. The fraction passing the 19mm sieve is used for compacted specimens. RLT samples have a nominal height of 200mm and a diameter of 100mm. The WT samples are 300mm by 300mm with 100mm thickness. Specimens were compacted to 100% Standard Proctor unless otherwise shown.

3 DISCUSSION OF RESULTS

3.1 *Specification Properties*

Table 1 shows some of the basic properties of the materials discussed in this paper as per Main Roads Standard Specification 11.05 (MRS11.05).

They are broadly representative of the majority of the lithologies used as roadbase materials in the QDMR network. The gradings of the materials are shown in Fig 1. As seen the

Source Rock Type	Basic Igneous	Acid igneous	Metamorphic	Metamorphic	Sedimentary	Metamorphic	Sampled from in-pavement	
Rock	Basalt	River Gravel	Greenstone	Metagreywacke I	Ridge Gravel I	Calc-silicate Metasediment	Metagrey-wacke II	Ridge Gravel II
10% fines (wet) kN	341	195	214	191	N/A	115	N/A	N/A
Wet/Dry Variation (%)	10	8	30	22		36		
Degradation Factor	70	75	54	72		65		
Flakiness (%)	14	14	24	20		28		
Fines								
LL (%)	19.2	20.6	20.4	18.4	21.2	22.6	19.2	21.2
PI (%)	0	1	2.2	0.4	9.4	5.4	2.4	5.4
LS (%)	1.2	2	2	2.2	6.4	4	3.2	3.4
Ratio 0.075/0.425	0.57	0.36	0.48	0.5	0.55	0.6	0.52	0.44
PI x %< .425mm	0	11	40	8	292	100	38	132
LS x %< .425mm	17.2	22	36	44	199	74	50	83
Grading								
$Cu = D_{60}/D_{10}$	48	13.6	82	53	340	117	45	66
Fuller's 'n' value	0.46	0.6	0.43	0.43	0.27	0.39	0.46	0.4
USC	GW-GM	GW	GW-GM	GW-GC	GC	GW-GC	GW-GM	GW-GC-GM
Compaction Conditions								
OMC (%)	8.6	5.4	6.7	7.2	6.6	7.3	5.1	5.7
MDD (t/m^3)	2.253	2.062	2.41	2.245	2.16	2.202	2.294	2.251
CBR (soaked)	135	105	78	120	N/A	78	98	68
APD (t/m^3)	2.895	2.659	2.945	2.725	2.641	2.718	2.712	2.677
Skempton 'B' value (Degree of saturation)	0.15 (66%)	0.23 (64%)	0.13 (60%)	0.20 (64%)	0.06 (63%)	0.21 (60%)	0.03 (50%)	0.13 (50%)

Table 1: Material Properties

*Modified compaction

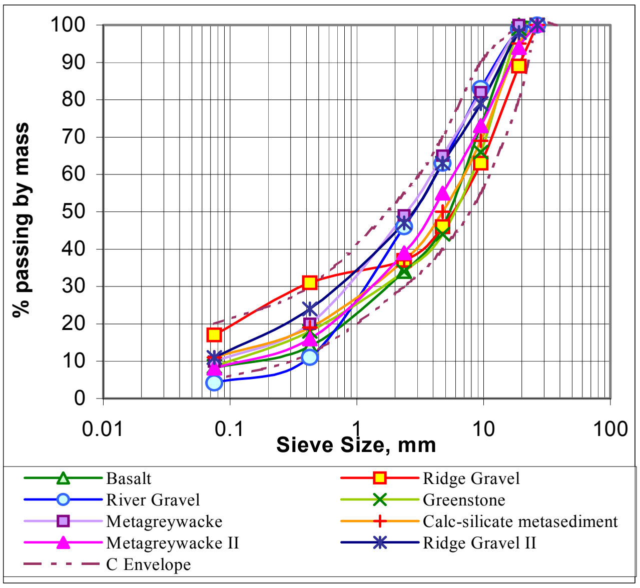


Figure 1: Gradings compared to Specification grading envelope

ridge gravel I (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 (0.35 – 0.5; Brown & Chan (1996)) for gradings that are attributed to give maximum compacted dry density. The constant head permeabilities of the majority of the tested materials are between 10^{-5}m/s to 10^{-8}m/s. These materials classify as GW/GC/GM. The two right columns in the Table give properties of the materials sourced from an existing pavement. Sections 3.2 and 3.3 discuss tests conducted in the 60-65% degree of saturation (DoS) range only. Section 3.5 discusses the results of the material sampled from pavement.

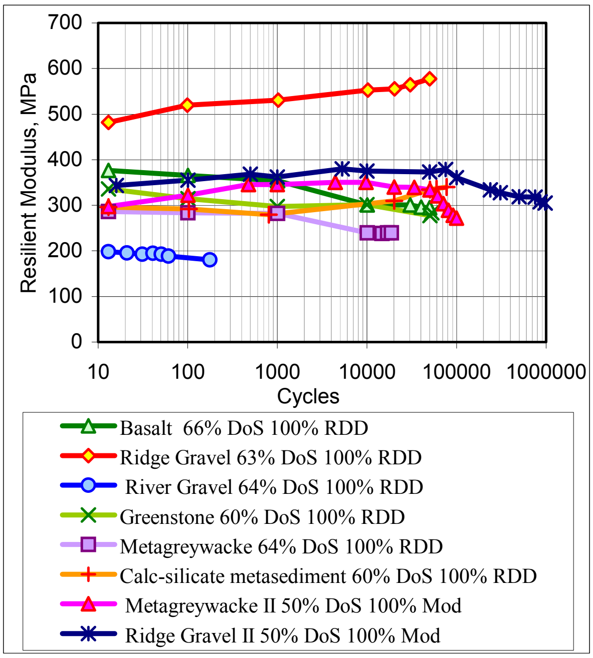
3.2 *Resilient Stiffness*

The behaviour of unbound granular materials is complex, because of their particulate nature. The 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.

Under repeated loading, these materials undergo resilient strains that are recovered after each cycle, and permanent strains that accumulate with the number of cycles. DoS has been used to correlate performance of these unsaturated materials.

Figure 2 shows the resilient moduli 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 I which is of lateritic origin has had the highest moduli with an increasing trend with cycle count, contrary to trends for the other materials.

Figure 3 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 50 kPa (for example the metagreywacke



material listed in Table 1). No premature failures of this lateritic material from the semi-arid North West Queensland have been reported to-date except for some flood inundated sections, in spite of its high PI (= 9%) and high Linear Shrinkage (= 6%) by conventional paving material standards.

Figure 2: Resilient Modulus

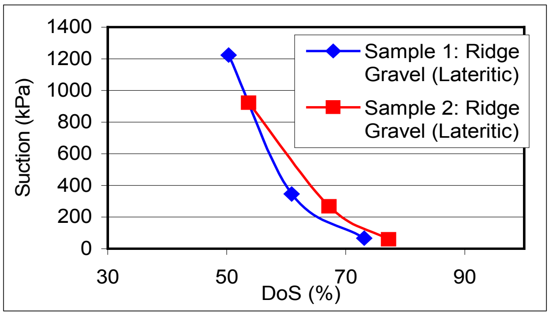


Figure 3: Suction results for two samples from Ridge Gravel I

3.3 *Plastic Strain Response*

Figure 4 shows the plastic strains measured both in the RLT & WT 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 ridge gravel I shows the lowest plastic strain i.e. the best rutting resistance. Additional testing to $1x10^6$ cycles and beyond may be necessary to establish whether this favourable rutting performance could be reliably sustained. Vuong (2001) has also shown decrease in plastic strain with increasing plasticity index.

As seen in Fig. 4 and Table 1, the basalt is the material with the highest fragmentation index (10% fines value), but does not yield the lowest plastic strain. It might appear that the strength of the coarse aggregate in itself may not be adequate for satisfactory performance. Generally, materials with low weighted linear shrinkages (LS x % passing the 0.425mm sieve) tend to show higher plastic strains in the RLT; the river gravel with the highest plastic strains has a WLS of 22, and high plastic strains are also seen in the greenstone (WLS of 36). Premature failures in the field have been more common with materials with low weighted shrinkage values.

The strength of the coarse aggregate, however, cannot be compromised as seen in the poor plastic strain results of the metasediment which has reasonably adequate plastic fines but the lowest 10% fines value as reported in Table 1.

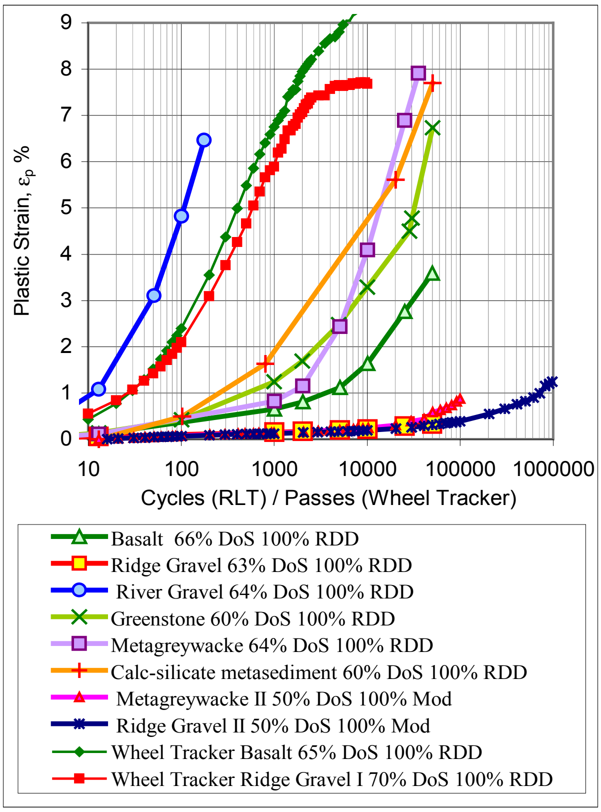


Figure 4: Plastic Strain from RLT and Wheel Tracker

Ishihara (1993), among others, has given some evidence to suggest that sands containing plastic fines exhibit higher resistance to liquefaction. Skempton's (1954) B values (Table 1) for the 60-65% DoS range reported in Figure 3 show that the lowest was for the lateritic gravel (0.06) and highest for the river gravel (0.23). However, the optimum level of plastic fines needs to be investigated more carefully.

The plastic strains from the WT are also shown in Fig. 4 for two selected materials from Table 1, namely, basalt and the ridge gravel I. They generally mirror the RLT findings, and show higher plastic strains than the corresponding RLT results. This is to be expected as the WT is a more onerous test as discussed earlier. The rounded river gravel (Fig. 4), which had the highest plastic strain in the RLT (Fig. 4), developed a plastic strain of about 15% within 10 passes in the WT (not shown).

The ridge gravel I 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 approach for material ranking based on plastic strain rate criteria using simple shakedown concepts as adapted by Werkmeister et al (2001) was used in this study. Figure 5 shows a shakedown analysis of the RLT results giving a ranking of materials similar to Fig. 4. In this methodology, plastic strain rate behaviour is categorised into three groups, viz: Range A-plastic shakedown Range; Range B –plastic creep shakedown range; Range C-incremental collapse shakedown range.

The lateritic ridge gravel I shows Range A behaviour which plots as almost a vertical line heading downwards towards very low strain rates and very little change in cumulative permanent strain. In contrast, the river gravel shows Range C behaviour where the strain rate is the highest and cumulative permanent strain increases steadily.

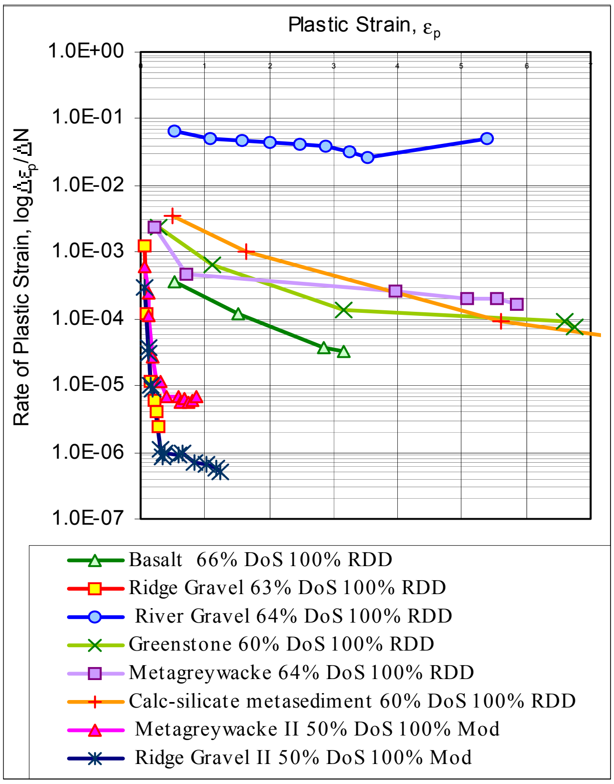


Figure 5: Shakedown Analysis: Rate of change of plastic strain vs plastic strain

3.4 *Material Ranking Issues*

This study uses plastic strain as a basis for material ranking. Plastic strains of 1.5% and 4% at 1,000 and 50,000 cycles respectively in the RLT and 2.5% strain at 10,000 passes in the WT are currently used as tentative guidelines in QDMR. These tentative limits need further work for confirmation.

In view of the susceptibility of materials with low weighted LS values to excessive rutting and premature failure in spite of having very high coarse particle strengths, caution is exercised with such materials and lower construction DoS levels are usually recommended, QDMR (1989). Specification of a minimum weighted LS of 40 is currently under consideration.

These results show the wide divergence of performance of crushed rock materials satisfying conventional QDMR type specifications. In contrast, performance based characterisation, enables performance risks to be envisaged and appropriate corrective actions to be implemented (Wijeyakulasuriya et. al 2004).

3.5 *Comparison with in-situ performance*

A preliminary investigation was undertaken to compare RLT results with an FWD assessment of an in-service pavement. Roadbase materials were sampled from two sections along a major motorway in South-East Queensland. Baran (2000) has documented the QDMR deflection bowl back calculation procedures, which undertake elastostatic multi-layered analysis.

The flexible pavement consists of 40mm of asphalt over 125mm of granular base underlain by 575mm of granular subbase. The average daily traffic corresponds to 22 x 10^3 vehicles per day (15% commercial). Some specification properties of the in-situ roadbase (metagreywacke II and ridge gravel II) are given in Table 1. This section of the motorway was built in 2000 and has started to show fatigue cracking due to heavy traffic. However, no base rutting was evidenced in the trenching used for sampling. The in-situ DoS values were around 40% for the metagreywacke and about 54% for the ridge gravel. The traffic loading to-date is estimated at 3x10^6 ESA.

Specimens compacted to modified standard compaction (to simulate measured in-situ densities) at DoS of around 50% were subjected to RLT testing. The ridge gravel was cycled to 10^6, while the metagreywacke was cycled to 10^5. Figures 4 & 5 do not show rutting in the granular base material under the observed in-situ DoS levels, confirming the field observations. This confirms a high shakedown load. The FWD based moduli (average values of 360MPa for metagreywacke, and 430MPa for ridge gravel) are underestimated in the RLT (Fig. 2). These tests do not readily allow the investigation of the influence of granular base layers on asphalt fatigue performance.

4 SUMMARY AND CONCLUSIONS

This paper has outlined the QDMR unbound granular characterisation methodology. The importance of performance based characterisation (using the Repeat Load Triaxial (RLT) and the Wheel Tracker (WT)) has been discussed. The contributory role of plastic fines is observed and needs further study. The material ranking framework thus provided has aided the management of risk associated with crushed rock usage as roadbase under traffic loads and high rainfall conditions prevalent in Queensland. Comparison with in-situ performance has shown the potential of these technologies as a viable tool for the practising professional.

ACKNOWLEDGEMENTS

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