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Session Report: Pavements and Fills

Burt G. Look
Foundation Specialists Pty, Brisbane, Australia

ABSTRACT: This report provides an overview of the reviewed papers presented in the Conference under the Theme of Pavements and Fills. The papers cover a breadth of topics which encompass both the old and new. Pavements cover both road and rail, while fill includes subgrades, embankment fills and mining stockpiles. The reader should refer to those papers for more details. This report is meant to encourage both discussion of those papers, highlight areas where further explanation is required and/or opportunity for further research. Quality control processes with simple testing and low cost procedures dominate. A move away from the traditional density testing to other approaches is also apparent. Modulus has a greater importance than strength in QC.

1 INTRODUCTION

This review discusses each of the 11 papers initially before providing an overview in aggregate. This report reflects both areas of interest for the reviewer and where some clarification or discussion is useful during the paper presentation.

This is not a paper review, as that had been carried out prior to this session report. Readers are encouraged to attend the presentations if the discussions on a particular paper herein is of interest and also to refer to the papers for specific details.

2 INDIVIDUAL PAPER DISCUSSION

2.1 *Verification of impact rolling compaction using various in situ testing methods (Scott et al.,2016)*

This paper uses a field based study to compare before and after compaction test results when an impact (non-circular) roller is used on trial. Various in situ testing methods, as well as instrumentation is used to measure the ground response and surface settlement measurements.

This trial is for a limited 1.5m depth and using a homogeneous material. The authors have carried out previous work in this area of using rolling dynamic compaction (RDC) and should have referenced more of their earlier work which covers other aspects (I am obviously following their research work). For example in Scott & Jaksa (2015), the authors show an average peak pressure of 120 kPa at 2m depth (Figure 1), yet this paper uses only 1.5m trial. Given this RDC approach has already been shown (by the authors) to extend to greater depths, then a question for the authors is the use of a lesser depth for this paper. Is this suggesting a target 200kPa pressure or does different depths apply to different materials?

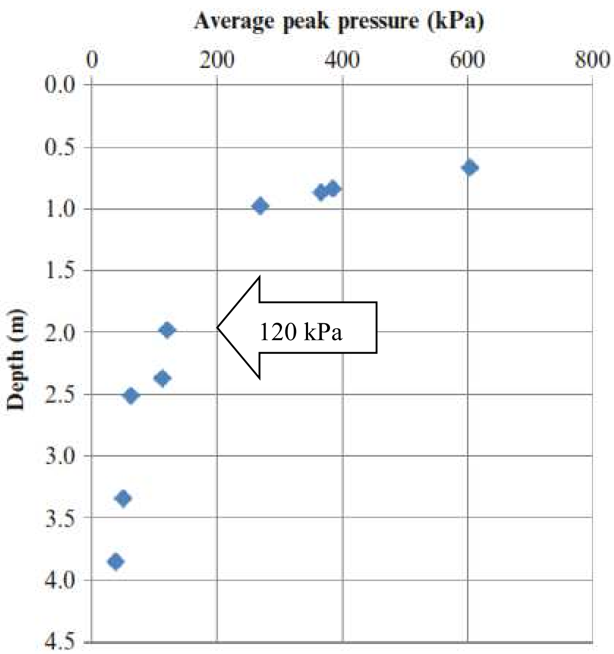

Figure 1. Average peak pressure vs depth ground (Scott and Jaksa, 2015).

The paper contributes to our understanding that deep lift compactions are achievable using RDC. No doubt this is just part of an ongoing research program. The benefits of deep lift compactions using modern equipment needs even more research before industry can promote its benefits on a wider basis rather than a project specific basis. Issues that still need to be addressed include.

(i) For traditional lifts, testing and reinstatement typically represent 15% of the compaction activity of placing, testing and re-instatement (Look, 2013). Deep lift compaction using RDC or heavy vibratory rollers have been possible for some time now, and this paper provides further proof of its effectiveness. However in practice the limitation is being able to provide quality control to that depth. Various techniques were used in the paper but industry needs verification processes which balances the benefit from deep lift compaction with not having increased testing + reinstatement offsetting the time benefit. (Figure 2).

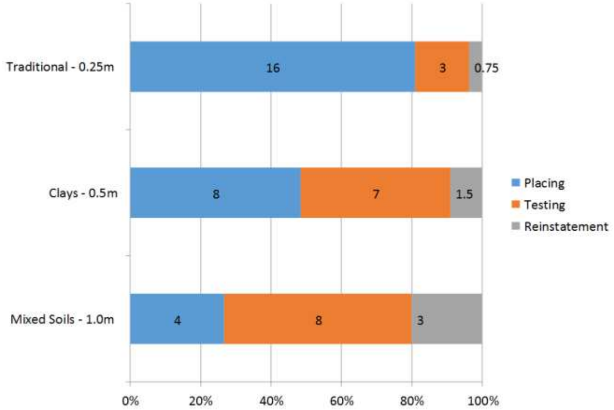

Figure 2. Typical Time (hrs) of various compaction activities for 4,000m^3 with 8 sand replacement tests (Look, 2012).

(ii) The paper shows 70 passes to achieve no further settlement. Although at a higher speed this is still a significant number as compared to traditional means of compaction (a factor of approximately 10). It would be informative for their further research to provide a cost and energy comparison as compared with other methods. For example a vibratory roller may not be able to achieve such a depth of compaction, but if 750mm (½ depth say) were achieved with 20 (say) passes, this may be a better production rate overall with less of an energy foot print. These equipment have a significant petrol consumption operational cost.

(iii) This paper supports the findings of Briaud and Saez (2012) who show the depth of influence from theoretical studies for various shape rollers (Figure 3). A match of that theory and this field practice does provide credence to the 1.5m depth. However in practice the more non round a roller, the less uniform for a given pass, which affects the quality control. This now leads to a similar question as 2) above, but for different reason. What is the *minimum* number of passes to achieve a similar "uniformity" of compaction similar to a round roller?

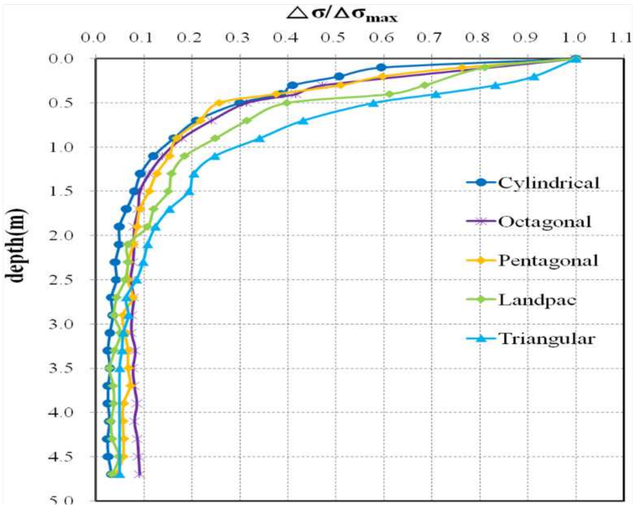

Figure 3. Depth of Influence for various shape rollers based on finite element analysis (Briaud & Saez, 2012).

2.2 *Suggested QC criteria for deep compaction using the CPT (Robertson, 2016)*

This paper is by one of the leaders in CPT use. This application is for quality control (QC) in deep compaction. However, while the CPT is popular in this application due to its low cost, the current methods of using CPT measurements for QC for deep compaction often apply only to clean silica sands and are not effective in soils with higher fines. This has frequently resulted in uncertainty on the effectiveness of the deep compaction. A suggested approach for QC for deep compaction is described based on the normalized equivalent clean sand cone resistance. There is also an associated webinar with more details freely down loaded at:- *www.greggdrilling.com/webinars/DwIgW/cpt-for-quality-control-of-ground-improvement-deep-compaction*

This is highly recommended reading as it answered some of the questions I had when writing this report. My first question was "what was a clean sand" as mentioned in the paper. The webinar provided the answer, and this is defined as $F_r < 0.5\%$. This also shows data from Kirsch and Kirsch (2010) with sandy soils with high fines content (> ~40%) and high CPT ($I_c < 2.6$) are generally not / less compactable (Figure 4). While ~40% is mentioned, this seems as an upper bound. Soils with 35% fines is considered "clays" in British Standards and would have been a closer match to the data of Figure 4.

My second question is on the clean sands at $F_r < 0.5\%$. Figure 5 shows soils as compactable and marginally compactible up to 1% and 1.5% friction ratio, respectively, then some discussion between these 2 different values would be helpful.

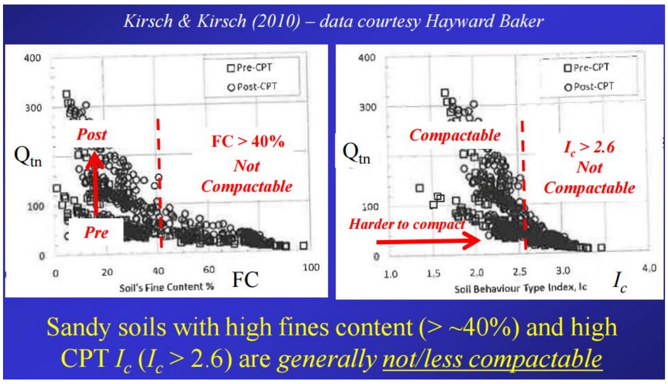

Figure 4. Compactibility slide (Robertson, 2015).

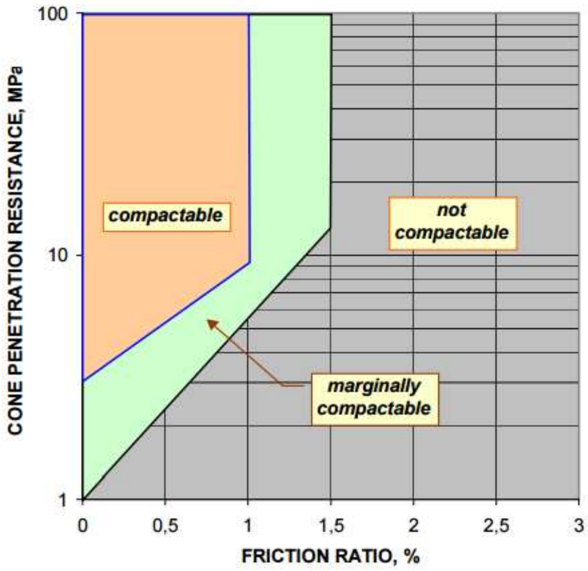

Figure 5. Soil classification for deep compaction based on CPT (Massarsch, 1998).

My final question is on time effects. Again the webinar provides some insights, but no recent data. Assuming there is less time effects with "clean" sands, then as the percentage fine increases, then some time effect is likely. Observing Pore Water Pressure dissipation is one approach, but further guidance on this aspect of likely increase in his future research would be appreciated.

2.3 *A new indirect tensile testing setup to determine stiffness properties of lightly stabilised granular materials (Gnanendran and Alam, 2016)*

A new IDT testing setup was developed in this study to determine deformations along the horizontal and vertical diameters of a cylindrical IDT specimen. The experimental program included the determination of IDT strength, stiffness modulus and Poisson's ratio for a lightly stabilized granular base material.

Table 1 provided the constants to be used in the equations provided in the paper. The authors should clarify why the 150mm gauge length has such a disproportionate change for the c_g and d_g constants.

Table 1: Values of constants for determination of elastic modulus and Poisson's ratio

Gauge length, g	a_g	b_g	c_g	d_g
37.5 mm (= D/4)	0.146	0.451	0.490	0.157
75 mm (= D/2)	0.236	0.780	1.075	0.314
100 mm (= 2D/3)	0.262	0.911	1.609	0.413
112.5 mm (= 3D/4)	0.268	0.952	1.970	0.457
150 mm (= D)	0.272	0.999	4.13	-0.04

The paper states "… an inaccurate Poisson's ratio with a difference of 0.1 from the actual value may increase/decrease the stiffness modulus by up to 25% resulting in an uneconomical and conservative pavement design". This is interesting as Poisson Ratio (in general geotechnical work) is not usually considered a governing parameter as compared to other material variables. The Poisson's ratios ranged from 0.18 to 0.26 for binder content variations of 1% to 3%

The IDT strength and modulus varied by a factor of 3.4 to 4, respectively for the 1% to 3% Binder content (Figure 6). The modulus was derived from both the IDT strength and Poisson ratio with the constants of Table 1. The assertion that the modulus is highly dependent on Poisson ratio is not immediately apparent from these figures. Showing the sensitivity by changing the values from 0.2 to 0.3 (say) would help illustrate the assertion of "being highly dependent on Poisson Ratio" to the reader.

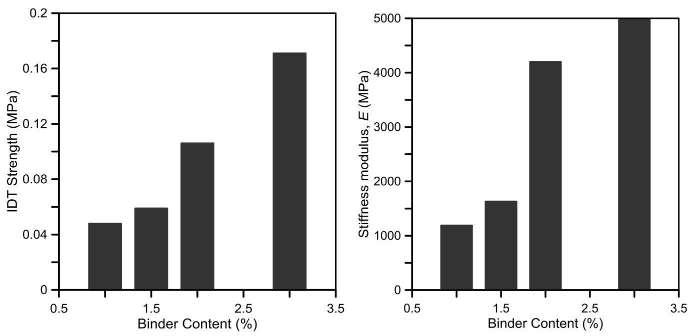

Figure 6. Variation of IDT strength and stiffness modulus with binder content.

2.4 *Geotechnical characterization of a heterogeneous unsuitable stockpile (Rengifo et al., 2016)*

This paper presents the geotechnical characterization of a heterogeneous stockpile in order verify the physical stability of the deposit and to optimize the closure configuration. This characterization was based on test pits, CPTU, MASW, field and laboratory tests. Based on the analysis of that information, strength parameters were proposed for slope stability analysis.

The amount of data provided by the CPTU tests, allowed statistical analysis for precise strength parameters for the different strata.

CPTU with dissipation tests were done, but the latter was not discussed. Given the phreatic level is shown at some depth the authors should describe its usefulness and rationale.

Laboratory tests were divide into coarse and fine materials. These 2 geotechnical units would have been clear enough to warrant that differentiation. This explanation would be useful as the reader is unclear if this was because of a clear differentiation in material stratigraphy, or is this location dependent, or simply for testing purposes. The UU triaxial and Atterberg limits are very similar with a very similar range for cohesion whether classified as coarse (35% average fines) or fine (64% average fines content). Which leads to the other consideration below.

Given the nature of mine waste stockpiles with a large quantity of coarse materials and with over 20% average material greater than 20mm fines (Figure 7) then an explanation on if large size samples were tested to derive the strength parameters, as scale effects may be different as compared to the fine samples tested.

The coarse material has friction angle of 32° to 34° based on field tests. Yet the fine material has an effective friction angle of 34° to 39° based on laboratory results. This is reverse of what one would expect but is a "normal" conundrum faced when correlations are adopted.

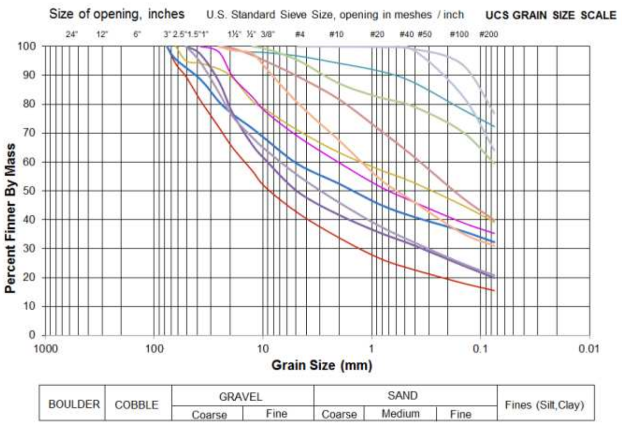

Figure 7. Grain size distribution of representative samples.

Equation 1 in the paper uses the exiting correlation for the N_{kt} value and even uses a conservative value in that range. Using a generic correlation seems out-of-place given the quantum of testing available to provide a site specific correlation. Correlations are used in the absence of site specific other testing, so some explanation is warranted.

2.5 *Control of soil compaction in pavement layers: A new approach using the dynamic cone penetrometer (DCP), (Belincanta et al. 2016)*

Compaction is widely used for the improvement of soil behavior. This paper presents field penetration tests data for compaction control and comparative laboratory testing for calibration of the penetration index (PI), the dry unit weight and the soil moisture content. In this way, the control can be performed in situ by measuring the PI and soil moisture content, as they are directly related to the degree of compaction of the layer. The results indicate that the PI value is inversely proportional to the degree of compaction and that it is strongly influenced by the soil moisture content.

A site specific relationship was developed for obtaining the degree of compaction at a given moisture content. This is not a universal equation as it is referenced to a poorly graded gravel with a soil of unit weight of 16.3 kN/m^3. The moisture content tested seems high at 20% to 32% for a gravelly material, and with CBR_{max} above 20%, but 10% to 13% at the optimum moisture content (Figure 8).

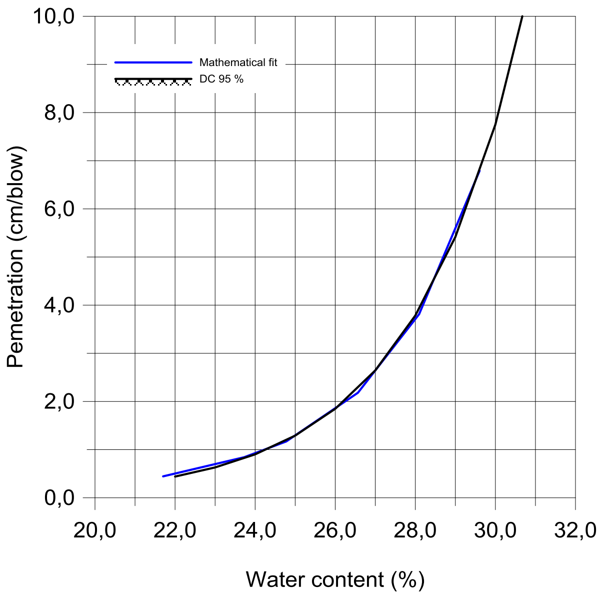

Figure 8. Compaction control as a function of the PI and moisture content, considering a degree of compaction greater than 95%.

The paper title is for soil compaction of pavement layers, but such a low CBR and high moisture content is inconsistent with the CBR required for pavement layers. These properties are more consistent with the subgrade material. Some clarification would be useful here.

In contrast, Burnham et al. (1997) provides the application of the DCP to pavement assessment procedures, with typical limit values of

- Silty clay material PI < 25 mm/ blow
- Select Granular material < 7mm/blow
- Special Gradation materials < 5mm / blow

These values are used when rehabilitation is required, but does not account for moisture variability which can be significant. Similar criteria can be found in various reports in the technical literature.

Some discussion on such an alternative approach which has a wider application and easier criteria as compared to this more refined criteria in the paper that accounts for moisture content but is very site specific.

The DCP can have a significant coefficient of variation (COV > 30%) as compared to density tests COV = 4%). While the DCP is certainly a more expedient tool, a discussion on how that greater variability accounted for in any assessment of compactions control would be useful.

2.6 *Use of the Light Falling Weight Deflectometer as a site investigation tool (Lacey et al., 2016)*

The Light Falling Weight Deflectometer (LFWD) is a surface based, dynamic plate load test that provides quick and direct measurement of the insitu modulus parameter of the near-surface. To demonstrate the potential use of the LFWD as an effective site investigation tool, two brands of LFWD were used to assess the insitu modulus of a residual soil and weak sedimentary rock profile. The performance of both LFWDs are compared to other 'traditional' site Characterisation techniques including DCP profiling and laboratory (soaked) CBR testing. Although strongly correlated, the two LFWD instruments will routinely produce different deflections.

I have been associated with the principal author for many years during his PhD studies where he used this tool demonstrating its usefulness. I would also refer the reader to Lacey et al (2015) which has a methodology using the LFWD to directly assess the improved modulus with geotextile inclusions rather than relying on manufacturer's values.

Over the past 50 years the construction industry has used density as the quality control test, yet in analysis and design the strength or modulus is used. The assumption is that the density controls relates to the strength and modulus values used. While that is the traditional approach the LFWD can provide a direct measurement of modulus and a better assessment of strength than a density inference can provide. Yet tradition rather than technology seems to govern.

Another barrier is that different LFWDs can provide different modulus This aspect is examined in this paper for 2 different LFWDs with one equipment giving approximately half the value of other although a strong correlation exists between the two and both instruments were consistent (Table 2).

Additionally, the limitations of the laboratory soaked CBR was discussed. That test does not apply for oversize a particles which are discarded during the test. Thus that test would incorrectly plateau at CBR 13% due to discarding the "rock" sizes.

Table 2. Typical DCP and insitu modulus material properties

	Insitu Modulus, $E_{LFWD-100kPa}$ (MPa)		
Blows / 100mm	Prima 100 LFWD	ZFG-2000 LFWD	Material Unit / Weathering State
3	16	8	SOIL (Fill / Residual Soil)
4	20	10	
5	24	12	
	43	21	Residual Soil to XW/HW Rock
10			
20	76	36	XW / HW Rock
25	92	42	
33	116	53	HW Rock

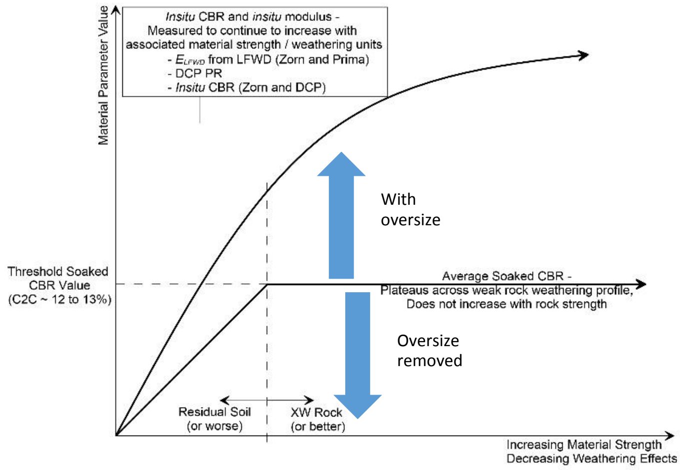

Figure 9. Comparisons between in-situ modulus and soaked CBR tests.

2.7 *Correlation between the results of the PLT and CBR tests to determine the elasticity modulus (Hajiannia et al., 2016)*

The CBR test is usually used to determine the relative strength of the subgrade soil and compacted layers. The Plate Load Test (PLT) yields more realistic soil elastic parameters, but it is costly. This paper presents a correlation between PLT and CBR test. Numerical modelling in ABAQUS and PLTs were used to develop a relation for determining the elasticity modulus using CBR test results. The relation was then checked through some PLTs in the site and CBR tests on the rebuilt specimens. The PLT was also used for the determination of the deflections due to loading, bearing capacity of foundations, and the soil elastic parameters. The modulus of elasticity yields an approximate estimation of the bed reaction coefficient (K_s). The related PLT load-deflection curves were predicted.

The CBR is a pseudo bearing failure test used to correlate to resilient modulus (M_r) and at a high strain of 2.5mm and 5.0mm. This does not occur in the linear elastic phase. A PLT load is a low strain for modulus test for measuring E in the linear elastic range. In fills and or to correlate to M_r the second cycle of PLT loading is typically used (not the first cycle). Hence the 2 tests are not directly comparable. However $E_{v2} \sim 2.3\ E_{v1}$ for comparison of 2nd and first cycle of loading.

Thus, the paper attempts to correlate with the linear elastic first cycle of loading of a PLT field test with a laboratory parameter (CBR) in the high plastic strain range. The PLT is a relatively low strain hence comparisons are being made between low and high strain tests. Figure 10 illustrates a few of these differences. The authors should clarify these considerations or mention the limitations of this approach in the paper.

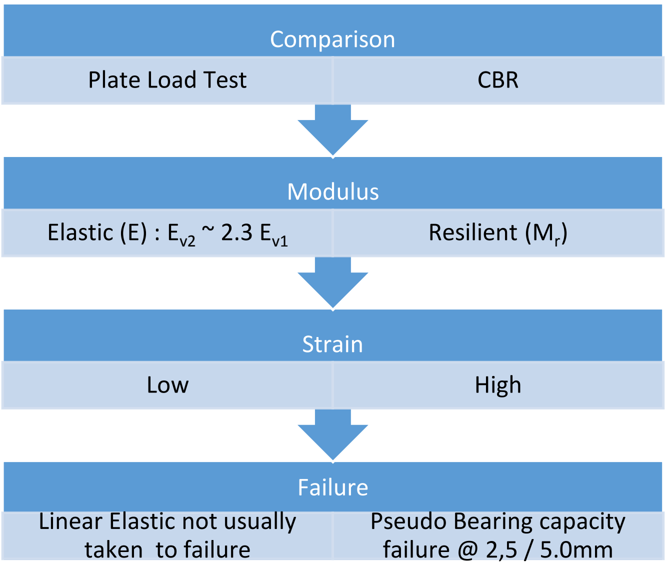

Figure 10. Comparisons between PLT and CBR tests.

This difference needs to be recognized in the FE analysis as a high strain modulus (lab) test is compared with a low strain modulus (field) test. The authors should clarify the rationale accordingly.

E/M_r ratios are material dependent (for example $E/M_r = 1.4$ to 0.3 for granular bases with stabilized layers and embankments below a granular base, respectively). Hence, while the procedure is interesting, any resulting relationship should not be applied outside of this case study due to that material dependency.

2.8 *Characterization of Railroad Track Substructures using Dynamic and Static Cone Penetrometer (Hong et al., 2016)*

A dynamic and static cone penetrometer (DSCP) is developed for characterization of rail-road track substructures. The DSCP consists of an outer rod for dynamic penetration in the ballast and sub-ballast layer and is an extendable inner rod for static penetration in the subgrade.

The DSCP is dynamically penetrated into the ballast and sub-ballast. In the subgrade, the inner rod with the mini cone is pushed. A dynamic cone penetration index is measured in the ballast and sub-ballast layer, and cone tip and friction resistances are obtained in the subgrade with a high resolution.

This is a hybrid of the cone penetrometer tests for penetrating ballast material. The dynamic component has a DSCP index associated. The standard DCP has an energy of 45J with a drop height of 575mm / 510mm and hammer weight of 8kg / 9kg. The drop height corresponds to the DCP but with no weight equivalent. The DCPI index is therefore new, and a discussion on why the index did not adopt the DCP standard would be useful as many existing correlations exist for the standard DCP.

The example experimental result (Figure 11) has 2 measurements (DCPI and cone tip resistance). As the main aim was for penetrating the ballast, the usefulness of the DSCP index needs further discussion.

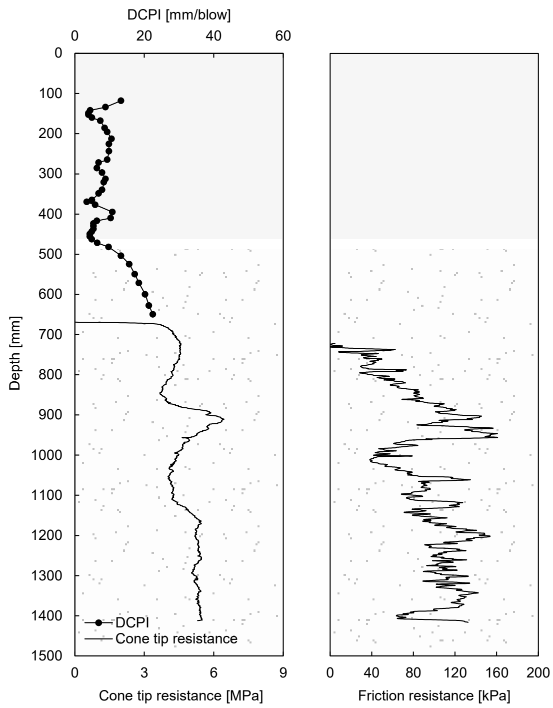

Figure 11. Experimental Results.

2.9 *Remedial measures to facilitate the construction of stable bridge approach fills-a case study (Diyaljee, 2016)*

This paper deals primarily with stability issues associated with the proposed 20 m high bridge approach fill. In November 1988 deep seated movements were noted at a depth of 22 m below the original ground from monitoring of slope indicators on the north approach. Remedial measures consisting of wick drains, stone columns, flatter approach fill head and side-slopes, and interceptor drains were then implemented.

The implementation of the remedial measures did not fully arrest the ground movements. As a result, the fill slope was modified. As the movements were taking place at depth it was decided to abandon the proposed 3-span bridge and construct a one-span Bailey bridge. This paper addresses the geotechnical investigation, evaluation and assessment of the bridge approach fills and the remedial measures implemented to minimize the movements

This is case study which is a bit dated except for a recent site observation in August 2013 which showed that the bridge is still serviceable after 23 years.

This provides an interesting reference case study to now assess if more recent understandings and/or design and construction techniques would have affected the decisions at the time. This would be in hindsight wisdom of course.

Monitoring of this bridge by survey hubs between June 2 and October 11, 1990 showed initial movements of 7 mm per day to July 4 and thereafter 1.2 mm per day. The bridge is still serviceable, but at 1.2mm / day, then over 100 mm had occurred in that period. One assumes that rate stopped over the next 23 years (or it would be over 10m movement). What was the final movement and hence tolerable movement for this type of bridge would be a key learning for industry.

Moulton et al. (1985) use intolerable movements of 100mm and 50mm for vertical and horizontal movements – but these were not for Bailey Bridges. These movements were significantly exceeded during the initial monitoring phase, and this case study shows the flexibility of Bailey Bridges.

2.10 *New and Innovative Approach to Ensuring Quality of Quarry Source Materials in Queensland Road Infrastructure Construction (Dissanayake and Evans, 2016)*

This paper discusses an approach that the Department of Queensland Transport and Main Roads (QTMR) adopts to manage the quality of road construction quarry products. A Quarry Registration System (QRS) was developed between QTMR and the Quarry Industry to address concerns that excessive testing was resulting in increased costs which were being passed on to the Department's construction projects. Guidelines allows the quarry management to self-assess their own testing frequencies and allows the Department as well as Quarry Industry to concentrate testing resources to where risks are highest. Testing frequency reductions of 90% have been realized in some cases with well managed quarries.

This paper is essentially about a management system for assessing quarries. Quality Assurance requires establishing the material variability (such as the coefficient of variation - COV). Each test listed would have a different COV (homogeneous vs heterogeneous) and the tests with the largest COV would represent the greatest quality "risk". Testing frequency is really only a subset of that risk profile (Figure 12). That level of technical detail would be useful to benefit the technical community. Reference to an associated paper Dissanayake & Evans (2015) provided no further detail, and with modest differences between the 2 papers.

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Figure 12. Frequency of Testing.

The list of tests required has been extended from the 2015 paper, but excluding 2 tests shown. Table 3 compares between the 2015 and 2016 papers. The rationale for this change would be useful. A ranking order (not shown), where some tests are mandatory, some are secondary and others as required would be useful discussion, as each test would not be given the same weighting by QTMR.

Table 3. Relevant source material Tests.

Source Rock Property Test ISC5 (2016)	Dissanayake & Evans (2015)
Petrographic Analysis	√
Wet 10% Fines Value	√
Wet/Dry Strength Variation	√
Degradation Factor	√
Particle Density (SSD)	√
Water Absorption	√
Bulk Density of Aggregate	--
Soundness (Sodium Sulfate)	--
Polish Aggregate Friction value	√
Weak Particles	--
Crushed Particles	--
Methylene Blue Value (MBV)	--
Sand equivalent	√
Light Particles	--
Particle Size Distribution	--
Material Passing 75μm	--
Material Passing 2μm	--
Organic Impurities	--
Sugar Presence	--
Sulfate Content	--
Chloride Content	--
	Alkali Silica Reactivity
	Alkali Carbonate Reaction

The original cost of testing was 1.5% of the road quarry materials. That testing has now been reduced in half with this QMS system.

The paper contributes to our understanding of the requirements for this quality management system, but gives little detail in terms of the quality control testing requirements, by only listing the type of test rather than acceptable, uncertain and unacceptable test boundaries. A few such indicators during the presentation would enhance our understanding of the QRS. Reference to specification and quality control details would also be required to be implemented and understood (Figure 13).

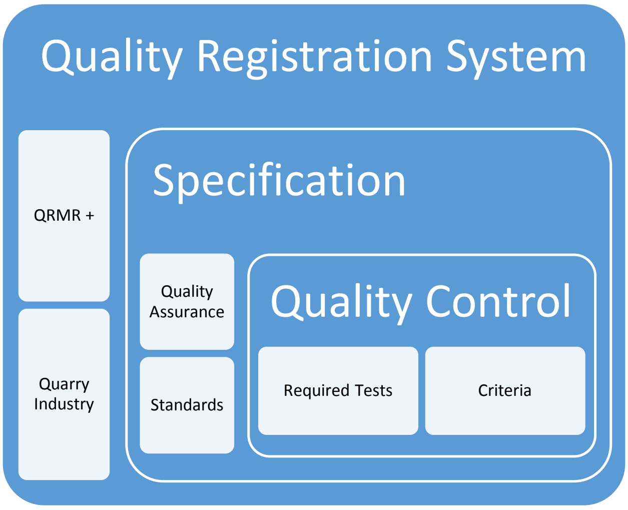

Figure 13. QRS and supporting documents.

2.11 *Evaluation of rockfill embankments by field tests in Siraf Refinery Complex site, Iran (Asghari-Kaljahi et al., 2016)*

Rock fill was used to fill valleys with up to 35m height required in some places. Trial embankments were used with lift thickness of 30, 45 and 60 cm and with 3 different compaction efforts. Differences between compaction percentages of 45cm and 60cm lift thickness showed the vibratory 15 ton roller compaction rate would be more effective in 45cm lifts. Various test were used to determine the compaction requirements. These tests consisted of field grading, large density, plate load test and surface seismic tests.

Constructing to "standard" maximum loose lift thickness of 300mm with heavy plant and 95% Standard compaction may be adequate for small to medium size project. But with large quantities of fill, a trial should be used to evaluate a best for project specification. The methodology and testing associated with development of such a specification is what is presented in this paper. This is a sound approach with some interesting insights.

The grading curve (Figure 14) should always be sue with caution in rock fill as soil testers sample a shovel into their sample bag. Hence, these curves show 200 to 300mm maximum. Clearly these are larger sizes (Figure 15). This highlights a common failure in industry (and this reporter has noted this in Australia on many projects) where soil testers do not place large sizes into their sample bag. Hence grading curves may be misleading with rock fills.

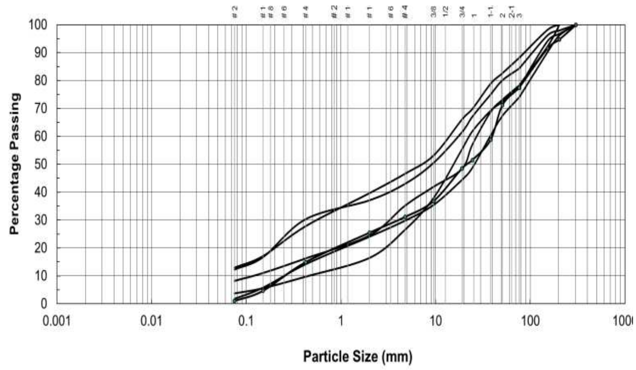

Figure 14. Particle size distribution curves rock-fill materials

Figure 15. Spreading of rock fill material.

At 8 passes, there was a reduced density as compared to 4 and 6 passes. The authors suggests it would be the result of particle breakage and loosening below the roller. This is an interesting result.

Both large in place density tests (water replacement) and the usual compaction test were carried out with 75mm and 20mm maximum sizes, respectively.

For the 20mm maximum size test, the maximum dry density was 21.6 kN/m^3 and 22.8 kN/m^3 for the uncorrected and corrected tests, respectively. A density change of 5.5% due to oversize. Even the large scale water replacement requires correction for oversize and that data would also be useful.

The modulus values seem high. Perhaps this was the first cycle of loading. For placed fill the second cycle of loading would be more relevant if that parameter was used in design. That said, the test as used is more as a comparative number to evaluate between passes / thickness rather than as a design value.

3 OVERVIEW

All of the papers in this session on "Pavements and Fills" could be in an alternative session heading, e.g. Sessions on "Case Histories", "Interpretation of in – situ tests", etc. Categorizing papers therefore ends in the trap of placing in another session heading. There are also associated papers in other sessions that could just as easily have been categorized into this session.

Readers interested in this topic areas should therefore look more bodily at other papers as well.

These discussions points are meant to be a prompt for possible discussion during the presentations, however the scheduling of this session report means that some presentations may have already occurred.

Quality Control (compaction) issues governs. Most are still wrestling with the basics of:-

- Density
- DCPs
- CBRs
- Gradings
- Oversize affects lab values

These simple issues affect our results. Standard CBRs, compactions and many lab testing cannot use large sizes and this is often over looked in our use of laboratory results. Issues that are trending in earthworks control are:-

- Light Falling weight Deflectometers
- Shear wave velocity
- Modulus Measurements
- Penetration testing
- Deeper Lifts

A clear shift to in situ testing is evident. Our comfort with the past approaches seem to hamper the progression and application of these tests. They provide a more direct measurement rather than density which "assumes" a permeability, strength or modulus has also improved without directly knowing by how much. A shift in quality control to these modern tools can be expected in time.

4 CONCLUSIONS

"Pavements and Fills" papers remain dominated by quality control processes with simple testing and low cost procedures a major consideration. A move away from the traditional density testing to other approaches is also apparent. Modulus has greater importance than strength.

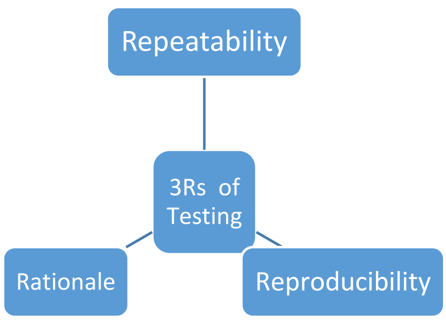

Figure 16. The 3Rs of Testing

Provided we confirm the 3Rs of testing (Figure 16), the traditional (and often outdated) approaches will continue to give way to the technology shown at this 5th International conference on Geotechnical and Geophysical Characterisation. "In Pursuit of best practice" is not just a conference theme, but an obligation we must continue with - forever.

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