DEEP DRY SOIL MIXING - PERFORMANCE AND QUALITY CONTROL ACCEPTANCE CRITERIA

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ABSTRACT

The variability in strength and compressibility of in situ deep soil mixing (DSM) of soft soils can be greater than the variability of the natural soil. To achieve economy, sustainability and performance of DSM, it is essential that construction be carried out using an appropriate QA/QC program.

This paper presents the results on the use of DSM columns to support several embankments over soft clay in the Ballina Bypass and Pimlico to Teven Pacific Highway projects. The DSM work was carried out under the Roads and Maritime Services (RMS) Specification, with QA/QC testing procedures developed specifically for these projects. Embankments with contrasting performance are compared to the quality control test results which comprised laboratory testing of core samples as well as in situ test results such as pull-out and push-in vanes and conventional cone penetration tests. Settlement monitoring indicated that where the acceptance criteria are met measured settlements under embankment loading were generally less than predicted settlements. The observed settlement in an area with high organic content was twice the predicted value, although in all cases, post construction settlement met the performance objectives.

Based on these results, recommendations on QA/QC testing procedures are made with the aim of improving economical and sustainable construction of the DSM ground improvement technique for soft soils.

1 INTRODUCTION

Between 2008 and 2012, the largest deep dry soil mixing (DSM) contract during that time in the southern hemisphere was carried out for the Ballina Bypass project. The project, comprising 12 km upgrade of the Pacific Highway located in northern New South Wales close to the border of Queensland, was commissioned by the Roads and Maritime Services (RMS) of NSW. The ground improvement contractor was Keller Ground Engineering (KGE) Australia. Seven types of ground improvement techniques were employed for the project, ranging from (i) low embankment strategy with nominal preloading, to (ii) surcharge with wick drains, (iii) light weight bottom ash fill, (iv) DSM, (v) vibro-replacement stone columns, (vi) dynamic replacement, and (vii) vacuum consolidation. Different techniques were chosen depending on time program, cost, and performance requirements relative to proximity to piled structures (e.g. bridge abutments) and post-construction settlement and differential settlement for either rigid or flexible pavements at different locations.

DSM was the most utilised ground improvement technique employed on the project, comprising a total of about 285 km of 0.8 m diameter columns to depths up to 18 m for the purpose of supporting bridge approach embankments and culverts with embankment heights of up to 6 m. DSM was chosen due to the very soft nature of the clay soil with moisture content practically at liquid limit, and the relative speed and economy of the system. Dry cement powder at rates of 160 kg/m^3 to 200 kg/m^3 were pneumatically discharged via a hollow stem auger, and mixed with the soil (using multiple horizontally mounted paddles on a single vertical stem) at high speed as the auger is withdrawn.

DSM was also used during 2012 to 2016 on the Pimlico to Teven (P2T) section of the Pacific Highway Upgrade project located immediately to the south of the Ballina Bypass project.

This paper describes the specification criteria adopted for the DSM work, performance of selected sites with contrasting performance following the works, and compares the Quality Control test data with the specification requirements. Based on the lessons learnt from these projects, suggested guidelines are provided to improve the specification for DSM work for future projects.

2 SUBSURFACE PROFILE AND CONDITIONS

The southern half of the Ballina Bypass alignment traverses some of the softest soft soils in Australia, and having depths of up to 30 m. Moisture contents are generally above 100% to 15 m depth, and the compression ratio, $C_c/(1+e_o)$ is typically between 0.4 and 0.6. Typical corrected cone tip resistance, q_t, moisture content, compression and recompression ratios, over consolidation ratio and undrained shear strength profiles at one of the bridge approach sites is shown in Figure 1 below.

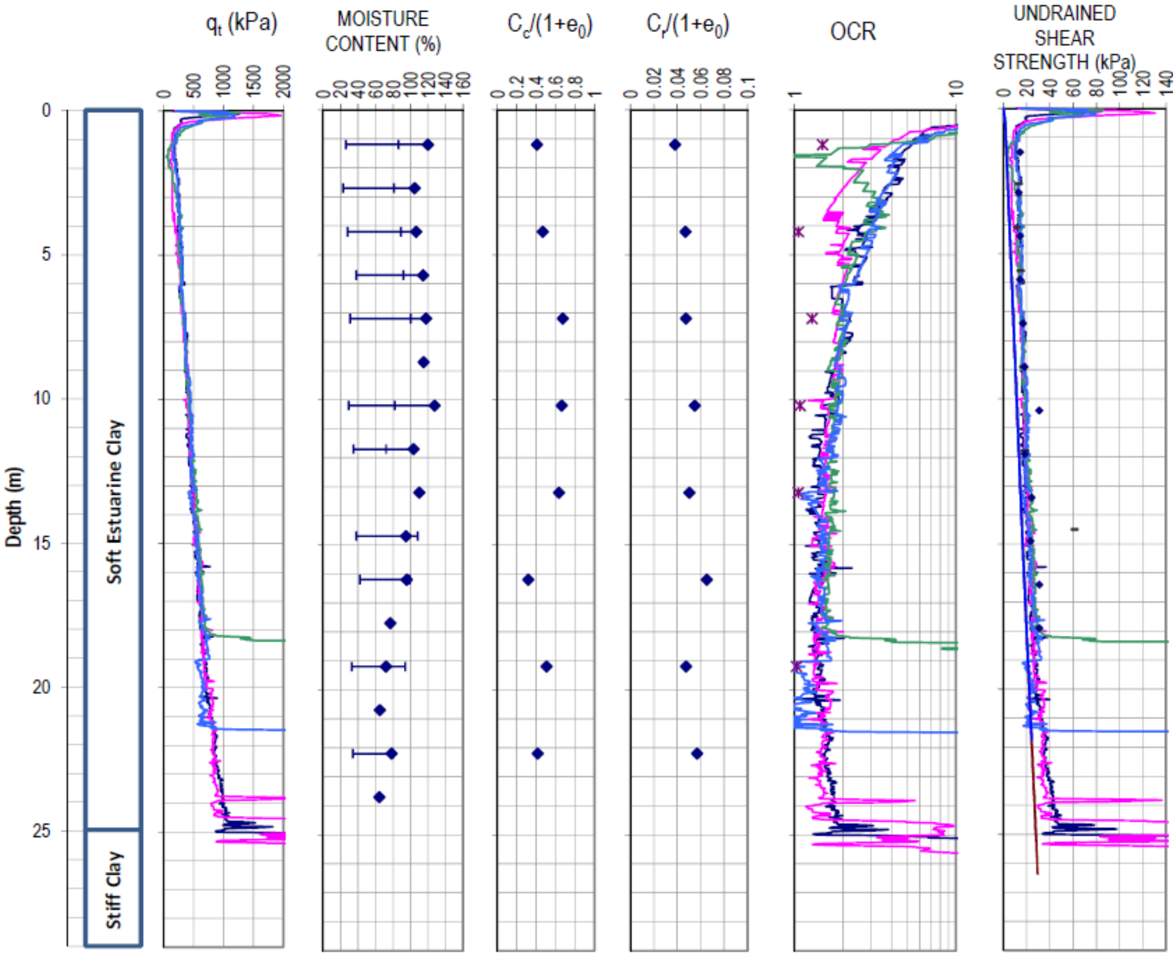

Figure 1: Typical subsurface profile and test results at the site

3 SPECIFICATION ACCEPTANCE CRITERIA AND QUALITY CONTROL

The DSM specification adopted for the project was based on European Standard (2005) and the Swedish Geotechnical Society (1997) for deep mixing.

The acceptance criteria for DSM columns at 28 days adopted were as follows:

- Design Shear Strength, $C_{col(d)}$ = 150kPa
- Design Elastic Modulus, $E_{col(d)}$ = 22.5MPa
- Not more than 10% of test results less than the specified design strength and stiffness
- Of the results that are less than the design values, they must not be less than 75% of the design values (i.e. min $C_{col\ (min)}$ = 112.5kPa, and $E_{col\ (min)}$ = 16.9MPa).

Shear strength was used as the main control parameter, although stiffness is actually the primary parameter of concern with respect to long-term settlement performance beneath the main body of the embankment. An empirical relationship E_{col} = 150 C_{col} (i.e. 75 UCS) was adopted for design purposes. This relationship was shown by testing correlation during construction to be rather conservative, as shown in Figure 2 based on test results from Zone 33, an area at Emigrant Creek Central – North Abutment (ECC-NA) with other zones also showing similar correlations.

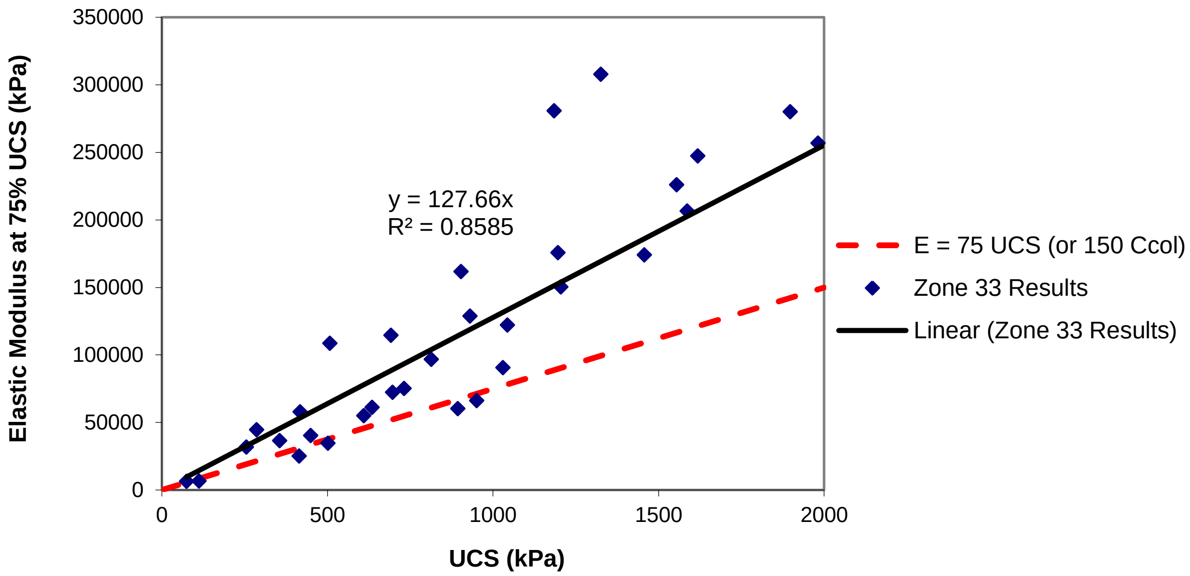

Figure 2: Relationship between measured E_{col} and unconfined compressive strength (UCS) of DSM columns

Strength was considered to be of greater importance for controlling the works due to stability requirements of the embankments. If the DSM column strength is less than the design value, over-stressing of the DSM columns may occur and the operating stiffness is likely to drop due to non-linear material behavior. Kamruzzaman et al (2009) report that when the yield stress of cement mixed soil is reached, the compression ratio can revert back to that of the original, uncemented soil.

From a design view point, the acceptance criteria were chosen to allow for potential variations in field strength and stiffness to guard against adverse performance (particularly long-term settlement performance) of the finished road and pavement.

The above criteria, when interpreted in statistical terms, imply that the mean column strength $C_{col(m)}$ and stiffness $E_{col(m)}$ in the field must be higher than the design values of $C_{col(d)}$ = 150kPa and $E_{col(d)}$ = 22.5MPa. This is illustrated in Figures 3 and 4, assuming a log-normal distribution of DSM column strength. Figure 3 is derived by adjusting the coefficient of variation, CV for a given mean column shear strength, $C_{col(m)}$, such that no more than 10% of the population fall below the design value of 150 kPa or no more than 1% of the values fall below 75% of the design strength (i.e. 0.75 x 150 = 112.5 kPa), whichever gives the lower CV. The 1% limit at 75% of the design strength was chosen as a nominal, small value to cater for a likely truncated distribution in the field compared to a theoretical distribution.

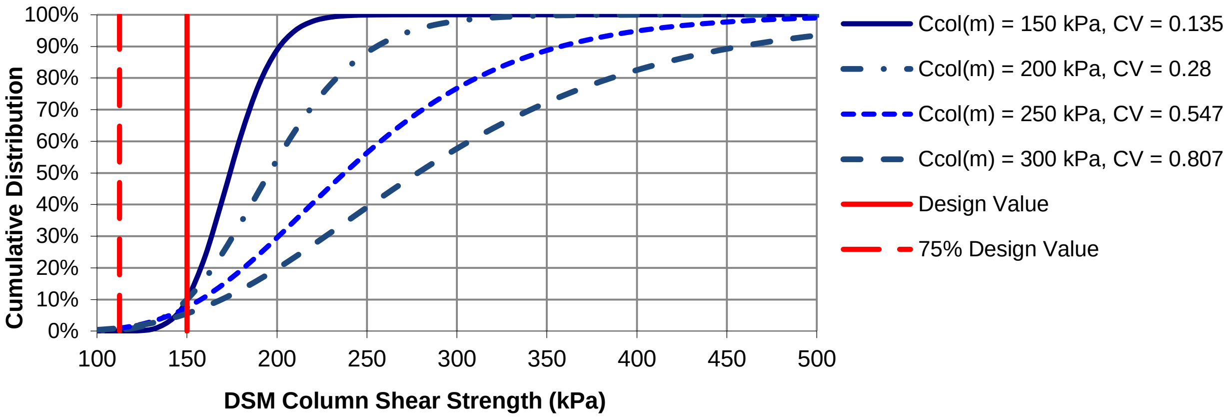

Figure 3: Statistical relationship of DSM columns shear strength to specification criteria

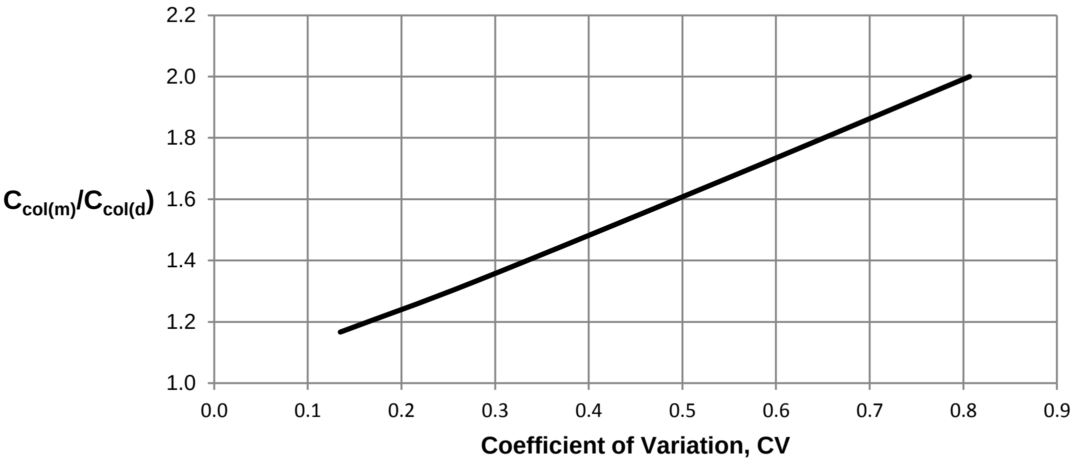

Figure 4: Required average DSM strength ratio as a function of CV for the specification adopted

Figure 4 shows a target mean column shear strength equal to the design strength will satisfy the specification requirements only if the CV is 0, which is not realistic. Filz and Navin (2006) and Adam and Filz (2007) reported that the CV ranged from 0.34 to 0.79, with an average of 0.56 from 14 data sets for ten deep mixing projects in the USA. Rather than the designer choosing a conservative strength for design, the specification adopted for the Ballina Bypass requires the DSM contractor to adopt a higher mean strength to meet the acceptance criteria depending on the uniformity of the product that can be achieved. Figure 4 suggests that the ratio $C_{col(m)}/C_{col(d)}$ increases linearly with increasing CV, and for the acceptance criteria specified for the project, the ratio $C_{col(m)}/C_{col(d)}$ would need to vary from approximately 1.5 to 2.0 for a CV value ranging from 0.4 to 0.8.

Discussions with the DSM contractor were made prior to finalisation of the project specification. To allow for uncertainties in pricing, the contractor provided a unit rate for cement content of 160 kg/m^3, with an increased rate for additional cement that had to be used to achieve the required strength to meet the specification.

The specification for the project required Quality Control testing comprising trial columns, vane pull-out testing, continuous coring and laboratory unconfined compressive strength testing. Further details of QC testing are provided in the discussions and recommendation section of this paper.

4 QUALITY CONTROL TEST RESULTS

A combination of vane pull out resistance tests (PORT) based on the Swedish method, and laboratory unconfined compression tests from cored samples were conducted on trial columns and production columns at each of the DSM sites. The field shear strength was interpreted by dividing the pull out resistance of the standard cast in vane (net area = 0.01 m^2) by a correlation factor N = 10 for the earlier Ballina Bypass work and N = 13 for the more recent P2T work (Liyanipathirana and Kelly, 2011 discuss factors affecting interpretation of the PORT test in more detail). The pull out test was considered more representative of the average strength of the DSM columns compared to the laboratory tests on cored samples as the former was able to test across a 0.6 m width of the columns as well as providing a continuous resistance profile over the column length. UCS testing of cored samples also suffered from potential bias on the better quality samples where low strength materials had poor sample recovery.

The field testing was carried out at different times after installation, with curing time ranging from 2 days to 34 days after installation of the columns. In order to compare the test results with various curing ages, and to the specification strength requirement at 28 days, a correction needs to be made to the test results. Figure 5 shows limited laboratory prepared samples that were tested at different curing times.

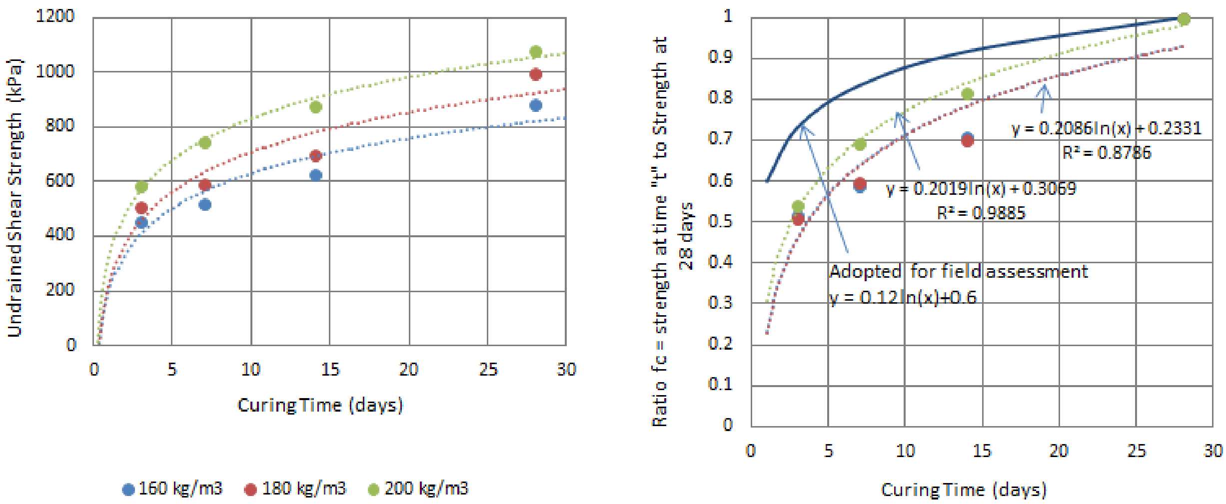

Figure 5: Relationship between strength and curing time for laboratory prepared samples at different cement contents (DSM Trial results from P2T project 2014)

A clear strength increase can be seen with curing age from Figure 5. The ratio of strength at time t to the strength at 28 days, f_c, has also been reported by Hodges et al (2008) as $f_c = 0.187 \ln(t) + 0.375$. However, the strength increase with time in the field is not as clear as the laboratory test results. The strength increase in the field was probably masked by the variability of the mixed soil. For this reason, this paper has been prepared using a more modest strength increase factor of $1/f_c = 1/(0.12 \ln(t) + 0.6)$ for correction of test results at time "t" to 28 day field strength. Typical histograms of test results (average strength corrected to 28 days over the length of the column obtained from PORT data) for selected sites are presented in Figure 6.

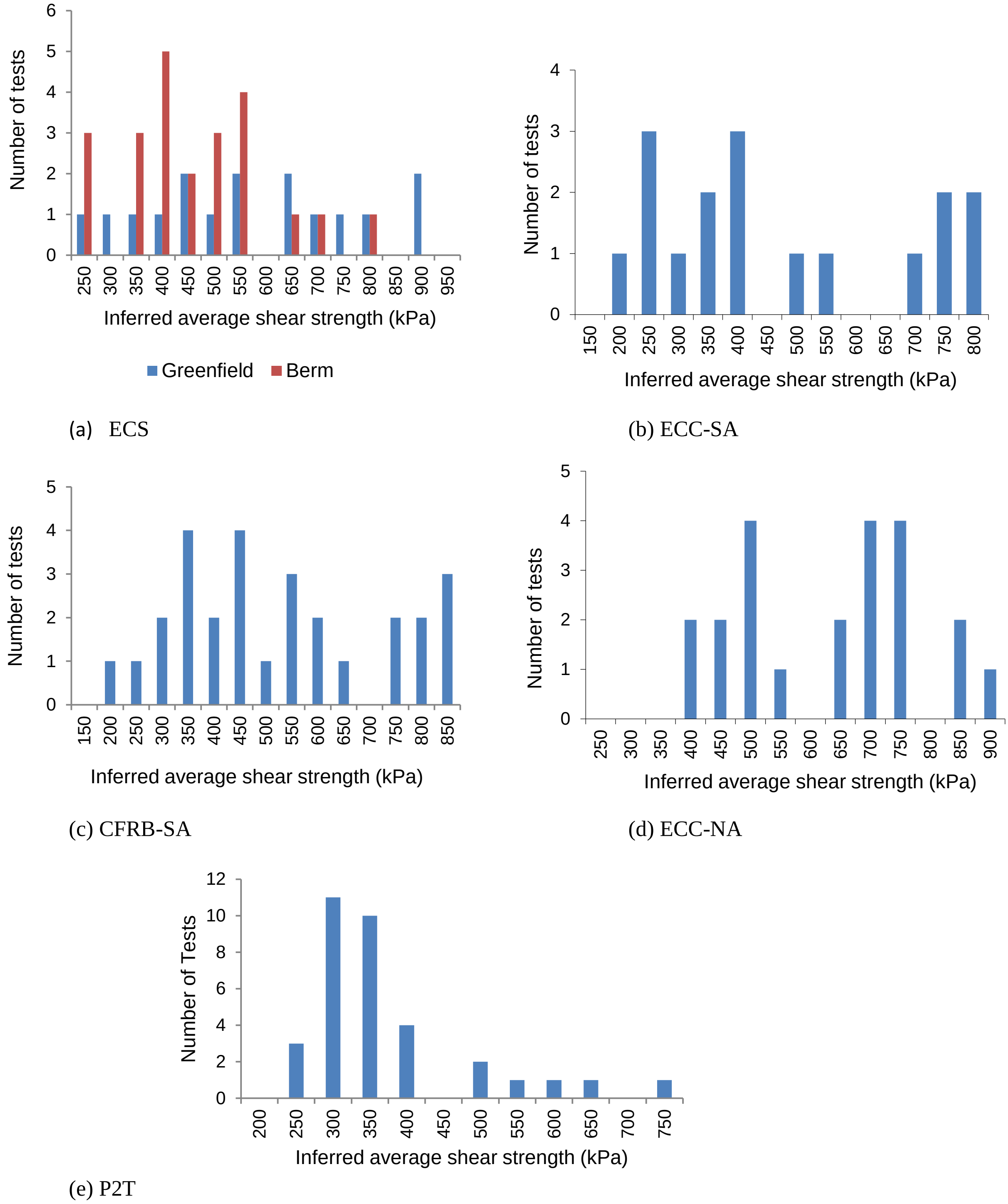

Figure 6: Histograms of inferred 28 day average column strengths for selected areas

A summary of the statistics from the inferred 28 day average shear strength results for the tested columns at the various sites are presented in in Table 1.

Table 1: Statistic Summary of Results

Statistics	ECC-NA	ECS	CFRB-SA	P2T*	ECC-SA
Cement Content (kg/m^3)	180	160	180	200	200
Mean shear strength (kPa)	518	481	513	405	360
Standard Deviation (kPa)	200	179	211	191	215
Coefficient of Variation	0.39	0.37	0.41	0.47	0.60
% ≤ 75% design strength	0	0	0	0.4	3.4
% ≤ design strength	0.1	0.1	0.2	2.3	9.6

* N = 13 for interpretation of PORT results at P2T, whereas N = 10 was used for the other sites

It should be pointed out that an N value of 13 was used for interpretation of the P2T PORT results whereas N = 10 was used for the Ballina Bypass sites. If N = 10 was used for P2T, the mean shear strength for P2T would have been the highest (i.e. 527 kPa). Taking this into account, the results in Table 1 show there is a general trend of increasing shear strength with increasing cement content. The exception to this trend are the results at ECC-SA. The site ECC-SA achieved the lowest average shear strength and greatest variability (CV=0.6) despite 200 kg/m^3 of cement was used. Of the five areas shown in Table 1, site ECC-SA fell outside the specification requirement with 3.4% of the theoretical cumulative log-normal distribution having shear strength less than 75% of the design strength, even though it just satisfied the less than 10% limit on design strength.

In general, it was found that higher cement content had to be used in areas where the soft soil contained higher moisture and organic contents.

5 SETTLEMENT PERFORMANCE

Results of settlement versus time under embankment loading for the various areas are presented in Figure 7 in terms of ratios of measured to predicted settlement. A ratio of less than 1 indicates over-prediction whereas a ratio higher than 1 indicates under-prediction. Settlement predictions were made using the equivalent stiffness method for DSM stabilised ground described in the SGF Report 4:95E (1997), based on the area replacement ratio of the DSM columns, and the elastic modulus of the DSM columns correlated with strength as described in Section 3 above.

It can be seen from Figure 7 that with the exception of site ECC SA, the embankment settlement was generally less than the design prediction. Site ECC SA on the other hand settled twice as much as the predicted settlement. It may be tempting to put this greater than predicted settlement to the fact that the test results at this site fell slightly short of the specification requirement in terms of the 3.4 % of the PORT results being less than 75% of the design shear strength. However, the authors believe that such a conclusion would be incorrect, and the reason for the increased settlement at this site is more likely to be caused by the higher organic content at this site which clearly showed lower average shear strength and higher variability compared to the other sites. However, the results are still acceptable in that the measured average field strength of 360 kPa exceeded the required $C_{col(m)}$ of 263 kPa (i.e. based on Figure 4: required strength = 1.75 times the design strength of 150kPa for a CV of 0.6). It should be noted that despite the fact that settlement at P2T following embankment construction was higher than the design prediction (typically between 100 mm to 150 mm was practically completed within 6 months. There were no embankment instability issues and the post pavement construction settlement was within acceptable limits.

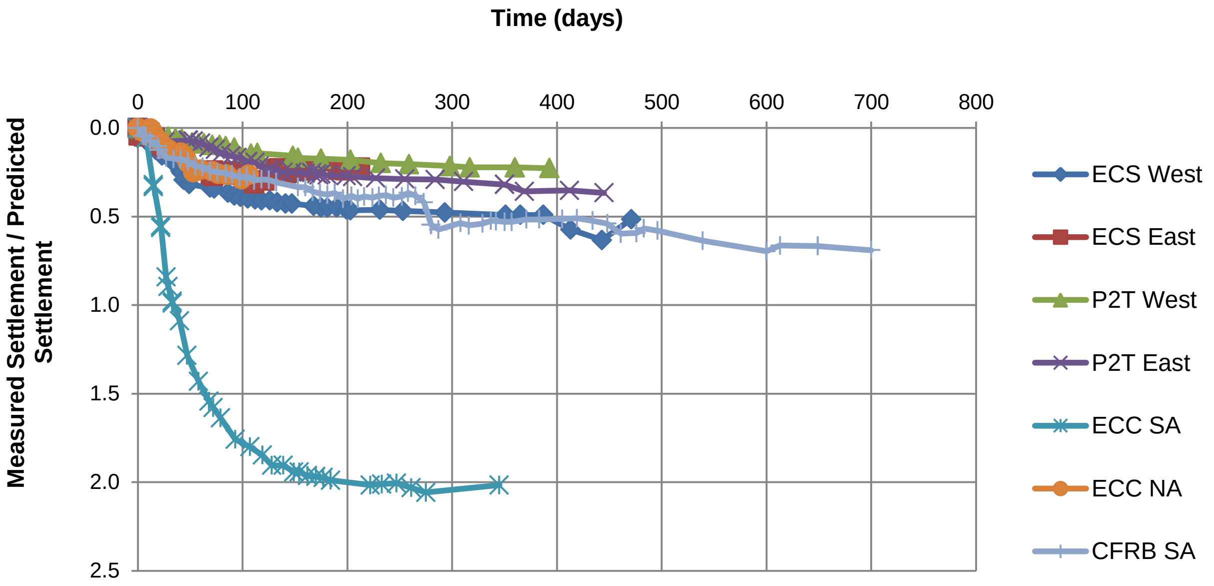

Figure 7: Measured settlement to predicted settlement ratios with time

It is not surprising that the embankment monitoring show that the majority of sites performed better than expected for the following reasons:

- The average strengths of the DSM columns installed are more than 2 to 3 times the design value. As the stiffness of the columns is proportional to strength, it is reasonable to infer that the column stiffness is also 2 to 3 times higher than the value adopted for design.
- The adopted correlation of E_{col} = 75 UCS (i.e. 150 C_{col}) for design is conservative, with the average E_{col} being more than 120 times UCS as shown in Figure 2. Similar results were obtained from recent laboratory testing in 2016 on Stage 2 of the P2T project.
- On the ECC SA abutment, it is possible that the higher than predicted settlements recorded can be due to the higher organic contents in the area resulting in higher compressibility of the mixed organic soils.

The results presented above demonstrate the variability of strength and stiffness in DSM columns but also that satisfactory performance can be achieved. It is reasonable to surmise that as long as lower strength and stiffness zones occur randomly and only locally (i.e. not concentrated in the same area), performance of the DSM would not be adversely affected.

6 LESSONS LEARNT AND GUIDELINES FOR FURTURE DSM SPECIFICATION

The results the projects described above showed that the use of statistical control on the acceptance criteria in the adopted DSM specification coupled with appropriate and adequate level of QC testing provided a reasonable means of achieving the desirable design outcome for the project. However, the statistics only told a part of the story. Experience from these projects highlighted the importance of conducting sufficient testing on soil properties (particularly humus and organic content) and trial columns prior to the main production works.

Based on the results of the Ballina Bypass project, changes were made to the DSM specification and acceptance criteria for the P2T project, with additional comments from the authors provided in square brackets [ ] as follows:

i. Sufficient laboratory trials with parallel 7, 14, and 28 days curing time should be carried out to assess likely binder content and strength gain with time. Samples need to be collected with depth to capture soil with the maximum moisture content because the water-cement ratio is a key indicator of strength. Note however that allowance must be made for low ratios of field strength to laboratory mixed sample strength. SGF Report 4:95E (1997) reported ratios in the range 0.2 to 0.5 for clay soils. The appropriate binder must be confirmed in field trials.

ii. Column penetration tests – Pull Out Resistance Test (PORT) or Push In Resistance Tests (PIRT) using the vane equipment shown in EN 1479-2005 shall be carried out to the full depth of 2% of production columns between 3 and 14 days after column installation. To maintain verticality of the penetration tests, preboring may be required. The DSM contractor shall submit a work method statement (detailing the type of equipment and column penetration testing and proposed interpretation procedure to be adopted) to the Principal for review and approval. The vane shall be pushed through the column at a constant penetration rate of 20mm per seconds ±20% with the penetration force continuously recorded. These tests are to be used as primary means for demonstrating acceptance. A minimum of two sets of parallel tests at 3, 7 and 14 days should also be carried out to assess strength gain in the field. Note that conducting the 14 day test may not be practical if the field shear strength at 14 days is over 1 MPa as the PIRT/PORT test will likely refuse. [Whether 2% is the right proportion of production testing of columns is an open question. The aims of the testing are to obtain a representative sample of the works for acceptance but also to allow a quick fix as problems are identified. A criterion tied to a minimum number of columns and/or a day or twos production could be considered. The criterion for proportion of tested columns is likely to be refined as more project experience is obtained over time in Australia].

iii. At least 20% of the column penetration tests should be carried out using the push in vane (PIRT) and parallel penetration tests shall be carried out in the unstabilised clay adjacent to these test columns (at least 0.5m from the edge of the column) to allow comparison of strength between stabilised and unstabilised materials to be made. If the strength of the DSM column is likely to cause refusal using the PIRT method, columns may be tested using the PORT method with the PIRT method used for untreated soil. [The purpose of this clause was to increase confidence in the N factor adopted to interpret column strength by taking measurements in the natural ground for which many independent strength tests have been performed. However, the amount of PIRT tests may be reduced if a conservative N value (i.e. > 10) is to be adopted for the interpretation of column shear strength].

iv. The results of the penetration testing should be reduced and reported using the method described by the Swedish Geotechnical Society – Report SGF Report 4:95E, with the exception that the N factor shall be assessed by reviewing the results of the stabilised versus the unstabilised tests. [This recommendation was a legacy of the Ballina Bypass. In practice, calibration of the N factor is difficult and use of a higher N value mitigates the risk of over-predicting column strength. The use of N=13 for the P2T project was based on the work of Liyanipathirana and Kelly (2011) but should not be seen as a value fixed in stone as several factors contribute to the resistance, as discussed by Liyanipathirana and Kelly (2011). Use of other values for N could be considered on a case by case basis if proved by the results of field calibration trials].

v. Column head exposure tests – Columns shall be excavated to a depth of 1m below the top of the column for visual inspection, and hand vane testing at 0.1m centres shall be carried out on two orthogonal lines across the diameter of the columns to assess the uniformity of the column strength.

vi. Plate load testing – This type of testing may be useful in assessing the stiffness of the DSM columns under applied axial loading. Testing shall be carried out using a 600mm minimum diameter plate at 0.5m depth below the head of the test columns, using quick set mortar to provide a level bedding for the plate. The plate load testing shall be carried out using at least two load cycles. The first load cycle shall be carried out to a maximum load of 50kN before unloading; the second load cycle shall then be carried out to the design working load of the column. The stiffness of the column shall be interpreted from the second load cycle.

The acceptance criteria shall be based on 28 day test results although earlier test results (e.g. 7 day or 14 day results) may be used for indicative purposes. Extrapolation from 7 day or 14 day results to 28 day results may be performed using data from the laboratory and field trials. Acceptance of PORT / PIRT tests shall meet the following acceptance, using an appropriate N factor calibrated against the UCS test results on a site specific basis:

i. From 0.5 m to 3 m depth – all results exceed the design strength;

ii. Below the upper 3 m – not more than 5% of the test section fall below the design strength provided these test results are equal to or greater than 90% of the design strength, and none of these sections are more than 1 m in length, and

iii. Acceptance of other forms of testing (e.g. UCS) shall allow for 10% of the test results falling below the design strength criteria provided these test results are equal to or greater than 75% of the target strength, to allow for variations inevitably associated with DSM, as well as testing methodology and interpretation.

Note that these acceptance criteria depend on a fundamental assumption that designers aim to minimise the number of columns and therefore that there is a low level of redundancy in the system. That is, if a randomly located column is weak, load will be redistributed to the surrounding columns and soil, and therefore it is important that the surrounding columns have sufficient strength and stiffness to withstand the increased load. If a design is developed such that there is a high level of redundancy in the system (i.e. larger DSM area replacement ratio than theoretically required) then the acceptance criteria can be modified to allow a higher proportion of columns to have a lower strength than the design strength. This higher proportion would need to be developed on a case by case basis.

The above DSM specification adopted for the P2T project was more stringent than the original specification adopted for the Ballina Bypass project which allowed 10% of results less than the specified design strength and stiffness. As shown in Figure 7, the P2T DSM supported embankments performed much better with measured settlement of only 20% to 30% of the predicted values. Therefore, it would appear that the specification and acceptance criteria adopted in the P2T project were on the safe side.

To make DSM an economical and sustainable ground improvement technique, it is necessary to avoid over-specifying the DSM design strength and stiffness requirements. It is also necessary to implement an adequate program of QC testing during construction to ensure design performance will be met. Ideally, field trials should be carried out ahead of production works to enable refinement of the DSM specification and acceptance criteria. Future refinement of DSM specification and acceptance criteria is likely to be possible as more project experience is obtained, including comparison of actual performance versus design predictions.

7 CONCLUSIONS

The results from the Ballina Bypass and Pimlico to Teven Pacific Highway Upgrade projects confirmed the shear strengths of dry deep soil mixing columns are within the expected range of variability reported in previous publish literature. Good DSM ground improvement performance in maintaining embankment stability and controlling post-construction settlement were achieved with adequate field trials and good workmanship.

The lessons learnt from these projects have been provided as guidelines for refinement of QC testing and acceptance criteria for future DSM projects. The settlement monitoring results reported above indicate that measured settlements were generally lower than design predictions. The presence of organics in the soil may cause greater variability in the strength and stiffness of the DSM columns, and potentially result in higher compressibility of the stabilised soils compared to stabilised inorganic soils.

Based on the above results, relaxation of DSM specification and acceptance criteria for inorganic soft soils would appear to be feasible and is desirable from economic and sustainability perspectives. On the other hand, greater care will be necessary for DSM stabilisation of organic soils. If possible, field trials should be carried out prior to production works prior to finalising the DSM specification and acceptance criteria.

Further research including comparison of field performance with design predictions will assist the refinement of specifications and acceptance criteria for future DSM projects.

8 ACKNOWLEDGEMENT

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