Field Assessment of the Strength and Deformation Properties of Limonite

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SUMMARY Detailed field investigations have recently been undertaken in the limonitic tropical soils of the South Pacific Region for a large industrial complex in New Caledonia. In situ investigations included piezo-Cone Penetrometer Tests (CPTU), Standard Penetration Tests (SPT), field and hand vane, pocket penetrometer tests, Menard pressuremeter testing and cross-hole seismic testing. Deformations of subsurface material under trial surcharge loading were measured using a magnetic extensometer. Several of the field techniques outlined were performed in close proximity allowing comparison of the interpreted soil properties through standard correlations. This paper compares and discusses the variations in soil properties assessed from different field techniques and compares them to properties assumed from observations of the soil under surcharge loading from a trial embankment.

1 INTRODUCTION

The site of a proposed large industrial development is located over 8000 hectares in the south eastern region of New Caledonia. New Caledonia is an island off the east coast of Australia, approximately 400 km long by 50 km wide, lying between 20 and 22 degrees latitude and 164 and 167 degrees longitude. The island experiences tropical weather patterns with an average annual rainfall of more than 3,100 mm (as recorded in the south east of the island). Extensive clearing of the land occurred during early European settlement and many slopes are deeply scarred by landslips and erosion. A relatively unique feature of the area are the dolines - dissolution cavities similar in nature to karsts in limestone terrain - that vary in size from less than a metre to tens of metres in diameter.

2 GEOLOGICAL SETTING

The site of the proposed industrial complex is underlain by the Late Eocene age Massif du Sud, Guillon (1). The Peridotite Massif du Sud is part of a larger structural unit, the Ultramafic Nappe that is present along both the eastern and western side of the island. Extensive faulting has occurred. Most of the large folds and faults follow the elongation of the island, striking at 110-130°, Brothers (2). Peridotite is a fine grained igneous rock, formed on the sea floor. It is dark grey green in colour and rich in iron and magnesium.

A soil profile that can extend to more than 50 m thickness has formed through the weathering and leaching of the Peridotite bedrock (Fookes (3)). Nickel and other minerals are leached from the upper layers of the Peridotite and concentrated lower in the weathering profile (Golder Associates (4)). Directly overlying the Peridotite is Saprolite, consisting of Peridotite gravel and corestones in a matrix of clayey silt soil. Limonite is present higher in the profile, overlying the Saprolite, where the Peridotite has been weathered completely into clayey silt. The Limonite has been leached of nearly all the silicate in the parent rock and is composed primarily of iron minerals. This extensive leaching has resulted in soils with high void ratio and moisture content that are stable in situ (Stocker (5)). The Limonite exhibits considerable structure, seen in the alignment of minerals, along which the Limonite will preferentially shear. A cemented ironstone cap (Ferricrete) has formed at the surface in most of the plateau areas. The Ferricrete is of variable thickness and has been reworked in some areas whilst in others it can be seen to be forming over a period of some months.

On slopes and in erosion gullies the soil profile can be thin to non-existent with exposed Peridotite rock. Fans or thin sheets of alluvium have formed in some areas where soil has been washed off ridges and where landslips have occurred.

The movement of the closely related ground and surface water has led to the formation of voids in the Peridotite rock and the weathered profile. Dolines are formed when the voids reach the ground surface leading to surface collapse.

3 FIELD TECHNIQUES

Obtaining suitable field investigation equipment was difficult in New Caledonia where their use is limited. This led to the trial of various techniques but the majority of the investigation was performed using standard drilling and sampling techniques. Standard Penetration Tests (SPT) were performed in situ and pocket penetrometer and hand vane tests were carried out on recovered undisturbed tube samples in the cohesive soils. Further investigations were performed adjacent to many of the boreholes including piezo-Cone Penetration Tests (CPTU) and vane shear testing. Testing was often performed in close proximity due to the high density of the field investigations. Limited pressuremeter and cross-hole seismic testing was also performed at selected locations.

Extensive laboratory testing was also performed on samples collected from the boreholes, but the results of these tests are outside the scope of this paper.

3.1 Standard Penetration Tests (SPT)

Standard penetration testing, while common in Australia, is not typically performed in New Caledonia. The local drillers were on a steep learning curve with equipment that was foreign to them and many errors were made in the early stages, including dropping sampling rods. Conveying the importance of accurate sample depths and cleanliness of the borehole was made more difficult by language differences. These difficulties reduced the confidence that could be placed in the results obtained from these tests. In soft cohesive soils the penetrometer often sank under the self-weight of the rods or the rods plus the hammer.

SPTs were used in the clayey silt Limonite to provide an indication of shear strength and Young's modulus based on the measured N value. Table 1 gives inferred soil consistencies based on the observed N values.

SPT N Blows per 300 mm	Shear Strength (kPa)	Consistency
0 to 2	< 12.5	Very Soft
2 to 4	12.5 to 25	Soft
4 to 8	25 to 50	Firm
8 to 15	50 to 100	Stiff
15 to 30	100 to 200	Very Stiff
> 30	> 200	Hard

Table 1. Inferred Shear Strength and Consistency based on SPT N values

Within the range of recorded SPT N values, Table 1 can be simplified to the following correlation between shear strength and N value.

$$c_u = 6.25N, \text{ kPa}$$

Several empirical correlations have been suggested (Blight (6)) to estimate the Young's Modulus, E, of a soil from the SPT N value. The correlation used for this paper was:

$$E = 1.6N, \text{ MPa}$$

3.2 Hand Vane

Undisturbed tube samples of approximately 50 mm internal diameter were taken at regular intervals in the Limonite. Selected samples were tested using a Pilcon hand held shear vane with either a 33 mm or 19 mm vane. The vane is imbedded in the soil and turned lowly, the dial reading is used to assess the shear strength of the soil. The shear strength is typically given by the following equation (Richards (7)):

$$S_v = T/K$$

where $K = pd^2H/2 + pD^3/6$

T = torque at failure

D = Diameter of the shear vane

d = diameter of rods

H = height of shear vane

For the vanes used the shear strength for a given dial reading could be read off a graph devised based on the constant K, and corrected for the vane calibration. The tube samples were transported from site to the laboratory at the end of each day where the hand vane tests were performed. Some disturbance of the Limonite may have occurred during sampling and transportation of the tubes on rough roads.

3.3 Pocket Penetrometer

The undrained shear strength of all tube samples collected was also assessed using a pocket penetrometer. Pocket penetrometer tests were performed in the field directly following the collection of the sample, reducing disturbance due to transportation. Several readings were taken and averaged to reduce the effect of the small scale of the test. Soil shear strength was assumed to be half of the measured pocket penetrometer reading.

Collection of the Limonite in an undisturbed state from the boreholes was often very difficult. Best recovery was achieved when tubes were inserted then left for several minutes to allow the sample to stabilise. The sample was rotated using a wrench on the rods at ground level before being lifted smoothly to the surface.

3.4 Piezo-Cone Penetrometer (CPTU)

A CPTU was performed adjacent to nearly all of the boreholes drilled during this field investigation. The CPTU test comprises pushing a probe into the soil at a steady rate and measuring the tip resistance, shaft friction and pore water pressure. The undrained shear strength can be estimated from the tip resistance, corrected for overburden, via the following equation from Robertson & Campanella (8):

$$c_u = \frac{q_T - s_{V0}}{N_K}$$

where: q_T = corrected tip resistance

s_{V0} = overburden pressure

N_K = cone factor, typically 15

The Young's Modulus can again be estimated using empirical correlations, with the following relationship (Stocker (5)), adopted in this study.

$$E = 20\, q_T$$

Malfunction of the electrical equipment due to the high temperatures and humidity experienced on site resulted in lost and erroneous data. The rate of penetration was controlled by the drillers and often varied markedly from the standard 20 mm/second. The results of many studies on the effects of penetration rate are summarised by Lunne et al (9) and indicate that increased penetration rates can lead to high cone tip readings.

3.5 Field Vane

Field vane testing procedures are similar to the hand vane described earlier except that the field vane test is performed in situ, typically at the bottom of a borehole. A torque wrench was used to rotate the rods as a Geonor was not available. The peak resistance to rotation was measured and, following failure, the residual shear strength of the soil was also measured. The calculation of the shear strength of the soil uses the same equation as the hand vane. Bjerrum (10) has recommended the following correction to account for the time dependency of shear vane testing variation with plasticity index, PI, of the clay.

$$S_u = lS_v$$

where $l = 1.7 - 0.54\log(PI)$

Standard testing procedures (AS1289 (11)) recommend that the vane be rotated 6 degrees per minute for peak shear measurement. In the absence of a Geonor the vane rotation speed was not as well controlled and was, in most cases, faster than standard procedures require. Effective refusal of the vane was achieved in very stiff and hard soils.

3.6 Menard Pressuremeter

Menard pressuremeter testing was performed in prebored boreholes to assess strength and modulus. This test comprises inserting a cylindrical probe into the ground and inflating the membrane of the probe in the radial direction. The measured change in volume of the probe with pressure allows soil strength and stiffness parameters to be assessed. Plotting the results as pressure versus volumetric strain, the gradient of the straight section of the plot is the shear modulus, G, as follows (Clarke (12)).

$$G = dp/d\,e_v$$

Young's modulus can be calculated by the following equation using an estimated Poisson's ratio, ν.

$$E = 2G(1+n)$$

Pressuremeter testing can be quite sensitive to disturbance of soil due to drilling and operator expertise. Local contractors supplied and operated the Menard pressuremeter for this investigation.

3.7 Cross-Hole Seismic

Cross-hole seismic testing can be performed on soil or rock to assess the in situ dynamic material properties including Poisson's ratio and Young's modulus. The test is performed in a pair of cased boreholes approximately 3 m apart. A source capable of generating shear and compression waves is lowered into one borehole and a geophone receiver is lowered into the other. The travel time is used to assess shear and compression wave velocities versus depth down the borehole. The following equation (McCann (13)) is used to calculate Young's modulus from velocity.

$$E = 2rV_s^2(1+n)$$

where r = mass density

Vs = shear wave velocity

n = Poisson's ratio

$$= \frac{0.5(V_p/V_s)^2 - 1}{(V_p/V_s)^2 - 1}$$

Vp = compression wave velocity

To increase the accuracy of the data interpretation the inclination of the boreholes from vertical was measured.

4 TRIAL EMBANKMENT LOAD INSTRUMENTATION

4.1 General

The trial embankment provided full scale in situ measurements of compressibility. Instrumentation was installed prior to the construction of the embankment, which reached a maximum height of thirteen metres. Instrumentation included a magnetic extensometer and several settlement plates to measure incremental settlements at depth and the total surface settlement.

4.2 Magnetic Extensometers

A magnetic extensometer was installed near the centre of the trial embankment site. Magnetic extensometers measure the settlement or heave of soil using targets placed at regular depths down a borehole and linked via a collapsible outer tube. A probe is lowered through a central access tube to measure the location of the targets and the differential movement is used to calculate incremental settlements at depth. As the trial embankment was placed, the extensometer was extended through the fill to allow uninterrupted monitoring. Extensometers can be quite difficult to construct and install in the correct manner. A planned second extensometer literally fell apart during installation and was abandoned. The operating extensometer required repair work from time to time to repair damage caused by earthmoving plant during placement of fill. This resulted in a reduction in the amount of useful data that could be collected.

4.3 Settlement Plates

Metal settlement plates with vertical rises were placed at the ground surface prior to the placement of the trial embankment. As the load was built up the vertical rises were extended. The relative level of the rises was measured at regular intervals to allow calculation of total surface settlement under the surcharge loading. Although care was taken during fill placement, risers were occasionally damaged by heavy plant, considerably reducing their accuracy.

5 FIELD MEASUREMENTS

5.1 Shear Strength

Shear strength values were calculated from field investigation results using the methods presented in Section 3. Figures 1 to 4 show the assessed variations in shear strength with depth at four field locations.

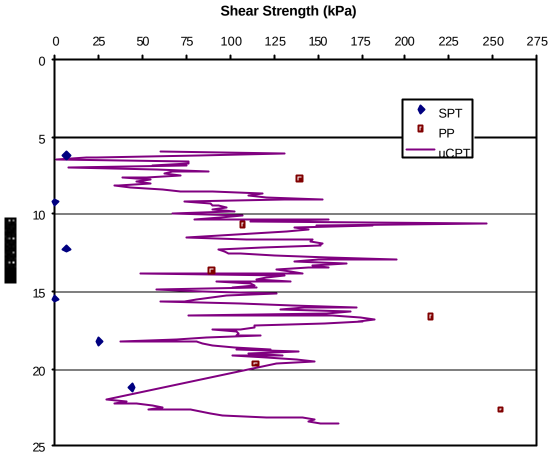

Figure 1. Field Location PL05

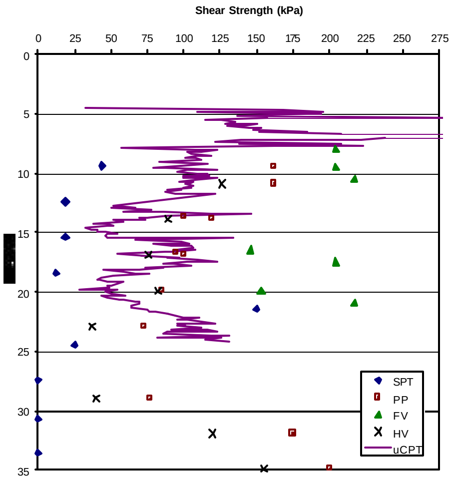

Figure 2. Field Location PL18

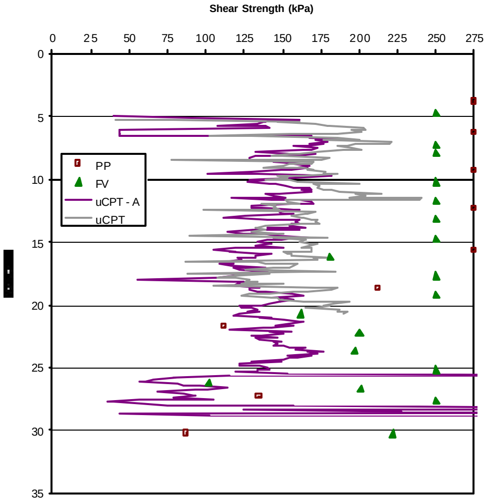

Figure 3. Field Location PL41

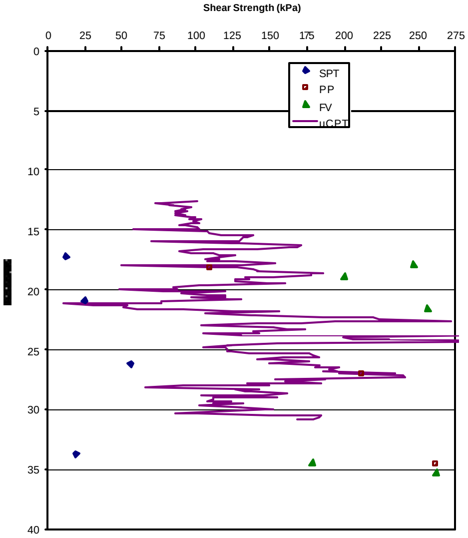

Figure 4. Field Location PL44

Figures 1 to 4 show that the assessed shear strength varies with the field technique used. For each of the field techniques the range of shear strength values obtained and their averages are given in Table 2.

Field Technique	Shear Strength (kPa)		
	Min	Max	Average
Standard Penetration Test	0	150	24
Pocket Penetrometer	72	>300	171
Field Vane Test	109	>250	215
Hand Vane Test	37	155	91
Piezo-Cone Penetration Test	1	1416	129

Table 2. Range and average of assessed Limonite shear strength.

5.2 Young's Modulus

The Young's modulus of the Limonite was estimated from the Standard Penetration Tests and piezo-Cone Penetration Tests using the empirical correlations outlined in Section 3. Young's modulus was also obtained directly from the strain measurements obtained through the pressuremeter and cross-hole seismic testing. The high values of Young's modulus measured using cross-hole seismic techniques are expected due to the small strain properties of the test. Stress-strain behaviour is typically non-linear with very high stiffness's measured at small strains. At intermediate and large strains, plastic behaviour dominates and the stiffness becomes more sensitive to strain and approaches a minimum value as the soil reaches failure (Matthews (14)). The assessed Young's moduli at field location PL05 are given in Figure 5.

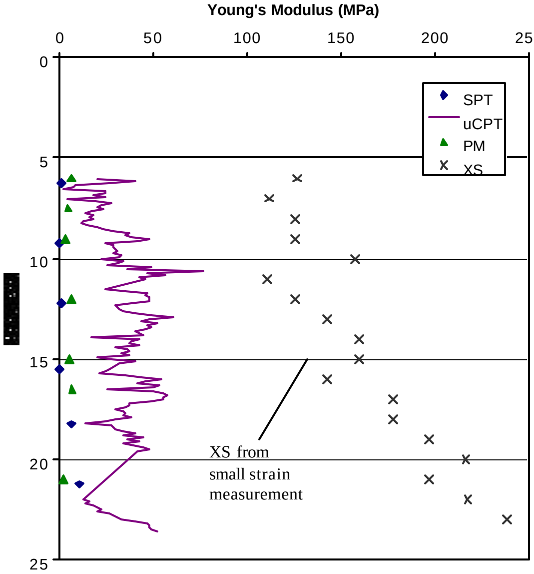

Figure 5. Field Location PL05

6 SURCHARGE LOAD MEASUREMENTS

Settlements under the trial embankment were measured from the settlement plates and movement of the extensometer at depth. At the maximum applied surcharge (350 kPa) Stocker (5) estimated the secant Young's modulus of the Limonite from the ratio of pressure to local strain. The computer program UNISETTLE was used to estimate the pressure profile for each depth increment and the local strain was based on measurements from the extensometer. It was observed that consolidation settlement occurred rapidly and could not be differentiated from elastic effects.

Two CPTUs were also performed at the site of the trial embankment, prior to fill placement, and the Young's moduli calculated from these are shown with the test load results in Figure 6.

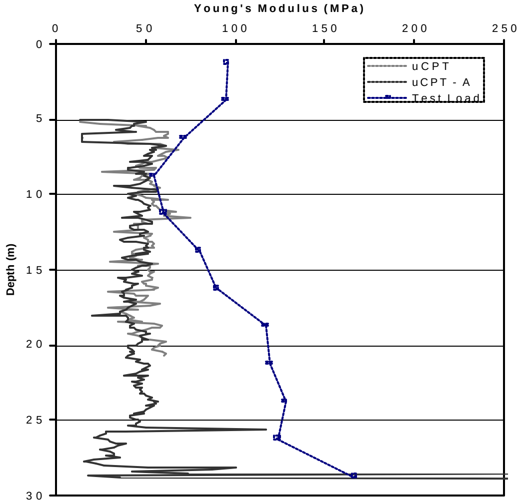

Figure 6. Young's Modulus of Limonite under the Trial Load

7 DISCUSSION

7.1 Shear Strength

Results of the SPTs indicate soil consistencies ranging from very soft to soft, lower than estimated from all the other field techniques. The average of these results is an order of magnitude lower than results of field vane testing. Such dramatically low values may be caused by breakdown of structure of the Limonite under the dynamic load applied during SPTs. The release of "trapped" water seen in the split spoon samples of SPTs supports this contention.

The measured shear strength from the hand vane testing also gave lower than expected results, with a greater variation. Measured consistencies ranged from firm to very stiff. The tube samples on which the tests were performed, although theoretically undisturbed, usually appeared to have undergone considerable change from their in situ state.

The field vane testing returned markedly higher shear strengths, averaging greater than 200 kPa. It would appear that the method of retrieving and transporting samples for hand vane testing caused enough disturbance to reduce the shear strength to approximately half of that for an in situ vane test.

Shear strengths calculated from pocket penetrometer readings averaged about 170 kPa, very stiff, and were on average only marginally lower than the strengths indicated by the field vane. Pocket penetrometer testing was performed in the same samples as the hand vane testing and consistently gave higher results by approximately 30 kPa. Pocket penetrometer testing was generally performed immediately following removal of the sample from the ground, with no transportation or time lapse.

The results of the CPTU were, on average, 90 kPa lower than the field vane testing. These differences are likely due to the disturbance associated with the UCPT and the effect of the assumed cone constant, N_K. Two anomalies witnessed during these tests may go someway towards explaining the low values.

Firstly, pore pressures were seen to increase to well over 1 MPa during testing only to drop instantly when tip advancement was halted to add rods. Stocker (5) surmises this is related to the structural collapse of the Limonite and that the tip measurement is actually the resistance due to some combination of peak shear strength near the cone tip and the residual strength of extremely distorted soil near the rear of the cone. With the residual strength being only a small fraction of the peak strength.

Secondly, there were several recordings of negative pore pressures, both spikes and sustained readings. It appeared that lenses of soil were dilating under the applied pressure.

7.2 Young's Modulus

Figure 7 shows the variation in Young's modulus with depth as assessed from all of the field tests and from back analysis of the trial embankment measurements.

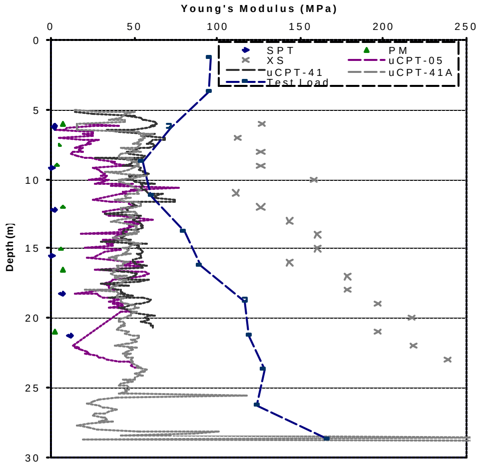

Figure 7. Young's Modulus from all Measurement Sources

The results from the cross-hole seismic testing and the test load indicate similar material behaviour with depth, although the numbers differ in magnitude. That is decreasing values to approximately 10 m then increasing with depth. The cross-hole seismic values are small strain moduli and are expected to be higher than the results from the other field measurement techniques. Small strain moduli measured through cross-hole seismic appear to be 2-3 times larger than those assessed from the trial embankment deformations. The cross-hole seismic moduli are from non-destructive techniques, unlike the other field methods that involve failure of the soil. It is also unlikely that the trial embankment caused failure within the Limonite. The absence of structural breakdown of Limonite led to higher values of modulus assessed from embankment measurements.

Young's moduli estimated from SPT N values are again very low and, as with the shear strength results, this is suspected to be due to the structural disintegration of Limonite. The preboring before pressuremeter testing may also have caused the breakdown of the Limonite structure close to the borehole walls and resulted in the low moduli values that can be seen in Figures 5 and 7. The CPTUs performed in the strata at the site of the trial embankment give higher Young's moduli than the CPTU at field location PL05, showing that there is some natural variation of stiffness across the investigation site. The CPTU assessed moduli appear to remain within a uniform range with depth. As outlined in Section 3.4 the Young's modulus is based on the measured cone tip resistance via an empirical correlation. The reduced strength of collapsed Limonite will therefore lead to a reduced estimate of Young's modulus. The assumed ratio of 20 may also be adjusted to vary Young's values.

8 CONCLUSIONS

Due to the high void ratio and open structure of the Limonite, formed through the extensive weathering of Peridotite in a tropical environment, standard field investigation techniques and empirical correlations used in transported soils and less leached soils were found to be of limited value.

SPT N values should only be used with extreme care as a source of information on the properties of Limonite. Even following testing procedures perfectly, their use may need to be limited to the collection of samples for logging and laboratory testing.

Use of hand vane and pocket penetrometer tests in undisturbed tube samples should be applied with consideration of the sensitivity of Limonite to disturbances during sampling and transportation. All testing should be performed immediately following sampling to reduce these effects.

Field vane testing appears to provide a realistic indication of shear strength but the results of laboratory testing should be used to correct for plasticity. The field vane is limited in its use in very stiff and hard Limonite where refusal occurs.

The assessment of Limonite properties from CPTUs may need to be limited to cone tip results as sleeve friction and pore pressures are measured in collapsed Limonite behind the tip. The use of any CPTU results should be based on site specific correlations derived on the basis of other field techniques and assessment of behaviour under surcharging.

Menard pressuremeter testing gave low indications of stiffness compared to other methods due to the initial disturbance caused by borehole drilling. Self-boring pressuremeters may give better results but it is likely that some disturbance of the Limonite will still occur and effect the results.

Cross-hole seismic testing appears to give a reliable estimate of the dynamic small-strain Young's modulus of Limonite. Its expense and the applicability of small strain measurement to design may limit the general use of this technique.

Instrumentation of a trial embankment provides results of a large scale field test not involving disturbance to the Limonite, allowing the measurement of modulus values without soil collapse. Time and cost constraints may restrict the use of this method but the measurements obtained can be used to develop site specific correlations and to gain a better understanding of the soil response to loading. Attention needs to be paid to the quality of the settlement data being received as small measurement errors can lead to big differences in calculated properties.

9 ACKNOWLEDGEMENTS

I would like to thank our client for graciously allowing this paper to be published. I want to acknowledge the extensive collation and analysis of Limonite data that was performed by Peter Stocker (GAP, Brisbane) and is presented in Reference 5. I have inspected only a very small subset of the data. Thanks to Pierre Legeron (GAP, Brisbane) who passed on as much data as he could find. Finally, thanks to the field engineers who worked in difficult conditions to obtain field data.

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A number of the referenced documents are project reports and are not available for general viewing.