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Report on
Geotechnical Investigation

Proposed Southern Special Purpose Precinct
Cobaki

Prepared for
Leda Developments Pty Ltd

Project 82508.01 (Rev 1)
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Integrated Practical Solutions

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# Report on Geotechnical Investigation
# Proposed Southern Special Purpose Precinct
# Cobaki

## 1. Introduction

This report presents the results of a geotechnical investigation undertaken for the proposed Southern Special Purpose Precinct (SSPP) at Cobaki. The investigation was carried out at the request of Leda Developments Pty Ltd in accordance with Douglas Partners (DP) Pty Ltd's proposal GLD130091 dated 17 May 2013.

This report should be read in conjunction with the '*Notes About this Report*' in Appendix A along with any other explanatory notes and should be kept in its entirety without separation of individual pages or sections.

The objective of the investigation was to determine subsurface conditions and the potential effect in relation to settlement, of the extension of the filling and preloading associated with the works for the missing link earthworks operations.

In order to comply with the above, the investigation was targeted to assess the subsurface conditions of the site and provide specific comment on the following:

- earthworks, including site preparation, stripping, compaction, reuse of excavated materials and trafficability;
- likely settlements under proposed filling and development loads and preload program options including preload heights and times; and
- suitable foundation options including high level and piled foundations, bearing pressures and negative skin friction on piles.

The investigation comprised the drilling of two test bores and pushing of seven piezecone tests (CPTu) as well as laboratory testing, engineering analysis and reporting. Details on the field work are presented in this report, together with comments and recommendations on the issues listed above.

## 2. Previous Investigation

A broadscale investigation was undertaken of the overall site in February 2010 by Cardno Bowler Pty Ltd and the results are reported in the report ref: 9243gs.10, '*Broadscale Geotechnical Investigation, Cobaki Lakes Development*' prepared for Leda Developments Pty Ltd. This report only undertook a single CPTu in the area of the proposed SSPP. The subsurface profile, interpreted from the CPTu testing indicated that the soft compressible clay (marine clay) may be expected from surface to a depth of 7.0 m underlain by stiff or stronger clays and loose sands to approximately 18.0 m depth where weathered rock was encountered.

## 3. Site Description

The SSPP site is located near the south west corner of the estate and covers an area of some 4.2 ha. Based on the supplied survey plan from Leda Developments P/L, (unknown drawing number), the existing suface levels across the site varies from RL 0.0 m to 0.5 m AHD.

The site, at the time of the investigation, was low lying, flood prone land that was vacant, had ponding water at the surface and was covered with thick grass up to 1m high in parts.

In order to gain access to test locations, Leda Developments Pty Ltd, arranged for the construction of several access tracks and working platforms, prior to arrival on site.
Figures 1 and 2 below indicate typical site conditions encountered during the investigation.

**Figure 1 – Site looking north west from southern boundary of the SSPP**

**Figure 2 – Site looking east from near western boundary of the SSPP**

## 4. Regional Geology

Reference to the Geological Survey of Queensland 1:100,000 series *'Murwillumbah'* Sheet indicates that the site may be underlain by compressible alluvial soils generally comprising *'esturing deposits, mainly mud, silty sand, clay and gravel'*. The alluvial soils overlie bedrock strata from the Neranleigh Fernvale Beds which consist of *'greywacke, argillite, quartzite, chert, shale, sandstone and greenstone'*.

The investigation encountered compressible alluvial soils over weathered rock at depth, and as such, are in general agreement with the above described geology.

## 5. Field Work Methods

The field work was undertaken between 3 and 10 October 2013 and comprised:

- two test bores to 19.5 m depth (Bore 4) and 24.5 m depth (Bore 7); and
- seven piezocone tests (designated CPTu01 to 03, 05, 06 and 08) to refusal depths of between 12.9 m and 25.0 m.

The approximate test locations are indicated on Drawing 1 attached in Appendix B. Also in Appendix B is a plan showing existing site contours. This plan is designated as Drawing 2. The test locations were nominated by DP and surveyed by Michel Group Services survey teams. The coordinates of each test location were provided by DP to Michel Group Services. Pegs were installed by the survey team indicating test locations prior to mobilisation to site. No site levels were recorded at any of the test locations.

Given the *'boggy'* nature of the site, access tracks were constructed by the client to each test location.

The bores were drilled using a truck mounted Hydrapower Scout drilling rig utilising 100 mm diameter solid flight auger techniques to 0.75 m depth, then advanced using rotary washbore drilling and casing techniques to the bore termination depths of 19.5 m (Bore 4) and 25.4 m (Bore 7). Standard penetration tests (SPT's) and/or *'undisturbed'* push tube samples were carried out at regular depth intervals to provide an indication of soil strength and also to provide samples for laboratory testing. Strata identification was undertaken through observation of auger cutting, washbore returns and the SPT samples. On completion, the bores were backfilled with drill spoil.

The CPTu testing comprised advancing a 35 mm diameter cone (with a following pore pressure transducer) and 130 mm long friction sleeve, into the ground at a constant rate of approximately 20 mm per second by hydraulic thrust from a purpose built test rig. Cone and friction sleeve resistance and pore pressure readings are measured continuously with depth and captured at 20 mm intervals on a computer for processing and plotting. The CPTu's were terminated at depths of between 12.9 m and 25 m. More information regarding this testing is given in the notes in Appendix A.

The field work was undertaken by or under the supervision of experienced geotechnical personnel from DP who positioned the test locations, operated the CPTu equipment, logged the bores, and collected samples for visual and tactile assessment and for laboratory testing.

## 6. Field Work Results

The subsurface conditions observed in the bores are described in detail on the log report sheets in Appendix C. The soil types encountered in the CPTu's have been inferred using published correlations, local experience and comparison with the test bores, and are shown on the CPTu report sheets also in Appendix C. These should be read in conjunction with the general notes preceding them in Appendix A which define the classification methods and descriptive terms used.

In general, at the investigation locations, compressible alluvial clays were encountered at surface which graded into stiff or stronger clays with depth, underlain by weathered sandstone. Layers of loose to medium dense sands were noted throughout each profile. The soil profiles encountered at each investigation was consistent with the regional geology.

A summary of the subsurface conditions encountered at each test location are shown in Table 1 below.

**Table 1: Summary of Subsurface Conditions/Depth Range (m)**

| Test No | Platform Fill | Topsoil | Sand (Loose to medium dense) | Compressible Clay (Soft) | Clay (Firm or stronger) | Weathered Sandstone |
|---|---|---|---|---|---|---|
| CPTu01 | 0.0-0.7 | - | - | 0.7-9.1 | 9.1-18.4 | - |
| CPTu02 | 0.0-0.3 | - | 0.3-4.0<br>20.3-21.3 | 4.0-5.1 | 5.1-20.3 | - |
| CPTu03 | 0.0-0.8 | - | 0.8-1.2<br>10.6-10.9<br>11.4-12.9(ii) | 1.2-8.4 | 8.4-10.6<br>10.9-11.4 | - |
| CPTu05 | 0.0-0.5 | - | 14.5 - 19.0<br>21.1-22.0 | 0.5-10.2 | 10.2-14.5<br>19.0-21.1 | - |
| CPTu06 | 0.0-0.8 | - | 19.95-23.10 | 0.8-7.1 | 7.1-19.95 | |
| CPTu08 | 0.0-0.7 | - | 0.7-3.95<br>18.4-20.4 | 3.95-4.45 | 4.45-18.4<br>20.4-25.04 | - |
| Bore 4 | 0.0-1.0 | 1.0-1.5 | 18.0-19.0 | 1.5-9.0 | 9.0-18.0 | 19.0-19.5 |
| Bore 7 | 0.0-1.0 | 1.0-1.5 | 2.0-4.5 | 1.5-2.0 | 4.5-23.0 | 23.0-24.75 |

Note i) All above depths recorded from fill platform level at the time of the investigation
ii) Indurated sand

Groundwater was encountered at an approximate depth of 1.0 m in Bore 4, which is at the approximate existing natural surface level, given the placed construction platform thickness. Based on previous experience with works on this site, by the author, groundwater across this low lying area could be expected to be within 1.0 m below surface level.

Based on the results of the investigation, Table 2 below indicates the soft compressible (marine) clay thickness encountered at each investigation location.

**Table 2 – Summary of Soft Marine thickness**

| Test No | Soft Marine Clay Thickness (m) |
|---|---|
| CPTu01 | 8.4 |
| CPTu02 | 1.1 |
| CPTu03 | 7.2 |
| CPTu05 | 9.7 |
| CPTu06 | 6.3 |
| CPTu08 | 0.5 |
| Bore 4 | 7.5 |
| Bore 7 | 0.5 |

Drawing 3 in Appendix B indicates an inferred compressible clay thickness contour plot across the site.

## 7. Laboratory Testing

Two multi-stage oedometer tests with particle density were undertaken on two *'undisturbed'* tube samples recovered from Bore 4. The oedometer loading stages comprised loading stresses between 6 and 1800 kPa, unloading to 1800 kPa and 100 kPa.

Based on the proposed fill height of 4m and the depth ranges tested for the oedometer samples, the pertinent stress range for compressible clay parameters selection has been assessed as being in the 100kPa to 200kPa range. Given this, Table 3 below indicates the parameters achieved based on the laboratory oedometer testing.

**Table 3 – Summary of Oedometer Test Results**

| Bore No | Depth (m) | Description | Particle Density (t/m$^3$) | $M_v$ (MPa$^{-1}$) | $T_{90}$ $c_v$ (m$^2$/yr) | $c_\alpha \times 10^{-3}$ |
|---|---|---|---|---|---|---|
| 4 | 1.5-1.9 | CLAY – grey | 1.45 | 0.66 | 0.64 | 10.0 |
| 4 | 4.5-4.9 | CLAY – grey | 1.52 | 0.58 | 1.05 | 8.3 |

Detailed laboratory test results are shown in Appendix D.

## 8. Proposed Development

The site covers some 4.2 ha. The existing site levels vary between RL 0.0 m and 0.5 m AHD. It is understood that the existing site level is to be raised by up to 4.0 m.

Details on building types, locations and structural loads were not known at the time of reporting of this investigation.

## 9. Comments

### 9.1 Appreciation of Ground Conditions and Potential Hazards

At the investigation locations, generally below the newly constructed access platforms, soft, alluvial, compressible marine clay was encountered from surface grading into firm or stronger consistency with depth. Layers of alluvial sand were also noted. The alluvial soils were underlain by weathered sandstone.

The soft marine clay thickness varied from 0.5 m (Bore 7 and CPTu 08) up to 9.7 m (CPTu05) at the tested locations.

Weathered sandstone was encountered at depths of 19.0 m and 23.0 m at Bores 4 and 7 respectively.

In relation to site hazard, excluding, any environmental issues, the major geotechnical issue (hazard) that would effect the development and that must be addressed in detail, is consideration of consolidation settlement from fill plus building loads. Therefore, this report addresses this potential hazard.

Given the presence of the compressible clays and the proposed structural fill heights, this will have implications on the timing of the development given the settlement magnitude and rates anticipated from the induced levels under proposed structural fill plus and expeceted building loads.

Given the site is a flood plain and there are no steep slopes, the details associated with *'hillside'* construction are not relevant for this site and as such have not been addressed further in this report.

### 9.2 Settlement and Surcharge

Primary settlement occurs under loading as pore pressures, generated by the loading, are dissipated. In the soft compressible clays, consolidation settlement is time and load dependent. In the loose to medium dense sand, settlement is expected to be elastic and occur essentially upon loading.

As the thickness of soft compressible soils encountered during the investigation varies, consolidation settlement is predicted to vary across the site.

A settlement analysis has been undertaken using *'Settle 3D'* software to model settlement of the development assuming a controlled filling layer up to 4.0 m thick, to raise site levels, with an average 20 kPa development loading (total 100 kPa loading – equivalent to 5.0 m of fill) Analysis was initially undertaken for the controlled filling and development loading assuming no surcharge and the results of the primary and secondary settlements are shown below in Table 4 below for various marine clay thicknesses.

For the purposes of this assessment and given the authors previous experience with construction on this site, a value of $M_v$ (coefficient of compressibility) of 1 $\times10^{-3}$ kPa $^{-1}$ has been adopted for theoretical consolidation settlement calculations. Further, for all calculations, two way drainage has been assumed. Predicting settlement times becomes difficult, as simulating the field conditions in the laboratory is generally not possible. In alluvial deposits, drainage in the settlement process occurs vertically as well as horizontally through shell layers and sand layers. By contrast, in the laboratory test, drainage is restricted to vertical only. Therefore the co-efficient of consolidation, Cv, in the field, may be several times the laboratory value. It is generally accepted that the coefficient of consolidation may be between 5 to 10 times of that indicated by laboratory tests. In this instance, a Cv of $5m^2$/yr has been assumed for all calculations. This value is consistent with the local experience of the author.

Included in the total estimated settlement magnitude shown in Table 4 below, is the predicted elastic settlements anticipated in the sand layers. Allowing for the worst case scenario of a 4.5 m loose thick sand layer, as encountered in CPTu05, up to 60mm of elastic settlement could be expected.

**Table 4 – Estimated Settlements for Various Marine Clay Thicknesses**

| Marine Clay Thickness (m) | Primary Estimated Consolidation (mm) | Secondary Estimated Consolidation (mm)$^{(i)}$ | Estimated Total Target Settlement (mm) |
|---|---|---|---|
| 0.5 | 50 | 15 | 125 |
| 2 | 200 | 40 | 300 |
| 4 | 400 | 55 | 515 |
| 6 | 600 | 65 | 725 |
| 8 | 800 | 70 | 930 |
| 10 | 1000 | 75 | 1135 |

Note i) 30 years design life

The magnitude of total settlement shown is theoretical and could be expected to vary depending on the physical characteristics of the underlying soils at a given location. Secondary consolidation, once all pore pressures are dissipated at the completion of the primary consolidation and where the load is then transferred to the *'soil skeleton'*. The consolidation which occurs at this point is termed secondary or creep settlement. The above secondary consolidation settlement values were based on a design life of 30 years. The effect of secondary settlement may be minimised by leaving surcharge filling in place until predicted settlement due to structural fill plus building loads, from primary consolidation and partial secondary consolidation, have been achieved. This timing is likely to be based on accepted tolerable residual movement, which is not known at this time.

Subsequent analysis was then carried out to determine the height of surcharge and period of time required to induce the total settlements as presented in Table 4. The results of the above assessment are shown in Table 5 below.

**Table 5 – Summary of Estimated Consolidation Rates Based on Various Surcharge Thickness and Marine Clay Thickness**

| Marine Clay Thickness (m) | Estimated Total Target Settlement (mm) | No Surcharge (5.0 m total fill) Estimated Time to achieve $T_{90}$ (months)(ii) | 1m surcharge (6.0 m fill total) Estimated Time to Achieve Target Settlement (months) | 2m surcharge (7.0 m fill total) Estimated Time to Achieve Target Settlement (months) |
|---|---|---|---|---|
| 0.5 | 125 | <1 | - | - |
| 2 | 300 | 2 | 1.5 | 1 |
| 4 | 515 | 9 | 5.5 | 3 |
| 6 | 725 | 21 | 11 | 8 |
| 8 | 9360 | 33 | 19 | 14 |
| 10 | 1135 | 66 | 40 | 23 |

Notes i) $T_{90}$ – Time to 90% primary consolidation
ii) Secondary settlements still remaining

Due to the variability of the physical characteristics of compressible alluvial soils encountered during the investigation, consolidation rates are expected to vary from the above depending on the parameters of the soils at a given location and therefore, should be considered an estimate only.

It is proposed that surcharge fill material be placed on the structural fill to accelerate consolidation due to final fill plus building loads. The maximum height of fill/surcharge placed must be controlled to prevent bearing capacity failure in the underlying soft soils. Based on the shear strength of the soft soils, a maximum initial fill height of 4.0 m is recommended, with fill batter slopes not steeper than 3H:1V. It is also recommended that to reduce the effects of the surcharging and any potential future effects from surcharge on neighbouring structures, the toe of the surcharge should extend 5m beyond the construction footprint.

## 9.3 Excavatability

Based on the conditions within the bores, it is estimated that excavation in the alluvial soils may generally be undertaken by small to medium sized excavation plant, such as backhoes, drotts to 8 to 15 tonne hydraulic excavators.

Rock breakers may be required for removal of buried concrete if encountered.

It should be recognised that the above excavatability estimates are based on materials encountered at the test locations only and that excavation conditions may prove more difficult (or easier) between, beyond and at depths greater than the test locations.

The results of the investigation and previuos experience across the site indicated groundwater 1.0 m depth below ground level. If excavations are proposed below this depth, measures to control inflow may be required.

## 9.4 Subgrade Preparation

The surface conditions at the time of the field work, generally comprised *'boggy'*, vegetated ground where some stripping of vegetation and earthworks will be required to enable trafficking of the site and construction of structural fill platforms. Given this, it is recommended that the following site preparation be undertaken:

- strip the grass and organic root zone and allow the surface to dry to promote some surface crust, where possible. If the area is especially wet, slashing and removal of larger vegetation can be adopted.
- following stripping and drying, place a maximum thickness bridging layer of say 500 mm to 600 mm of well graded granular material of a maximum particle size of 150 mm over the stripped surface, with not more than 20%, by volume , of the layer containg particle sizes of 150 mm. The initial lift should not exceed 500 mm and should be static rolled using a minum 12 tonne smooth roller. A minium of 8 passess is recommended. Subsequent lifts should not exceed 300 mm loose thickness with the same compaction treatment as just discussed. A total fill thickness of this lower couse (bridging layer) should not exceed 1.0 m.
- the use of geofabric may be required on the newly stripped surface, prior to the placement of the bridging layer, if placement of the bridging layer becomes problematic;
- after the initial lift, subsequent lifts preferable with granular material, can be placed and compacted as discussed below;
- select granular filling should be placed for the structural fill, as soon as possible, in layers not exceeding 300 mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% standard. Any clay fill should be placed within $\pm$2% of optimum moisture content;
- the top 1.0 m of the finished platform should comprise of minimum CBR 15% material with a maximum particle size of 37.5 mm. Not more than 20% of a layer thickness, by volume, should comprise material at 37.5 mm. For material below 1.0 m depth from finished surface level and above the lower course (bridging layer) material, a maximum particle size should not exceed 75 mm and again, should comprise not more than 20% of the layer thickness, by volume. Any granular material must be well garded. For a crushed rock material, a well graded material would be required. Too much same size material may not allow for sufficient interlock of particles during compaction; and
- Level 1 testing of filling, in accordance with AS 37989-2007 (Ref 1), is recommended for the placement of controlled filling for raising site levels.

The above procedures will require geotechnical inspection and testing service, by DP, during construction.

## 9.5 Re-Use of Materials

Organically rich surface material removed during the site strip would not be suitable for re-use in the controlled filling but could be used as a topsoil cover or for landscaping areas. From existing and previous works on the site, stripping depths would be expected to vary between 0.1 m and 0.5 m. However isolated areas of deeper stripping may be required. Any firm to stiff clay and sand/clay filling won from excavation would be considered suitable for reuse provided moisture and compaction are carefully controlled, and the particle size, reactive movements, workability and trafficability issues associated with the soils are acceptable.

Where fill is placed under controlled conditions, there is potential for *'creep'* of the filling materials as the filling settles over time under self weight. Such settlement will be minimal beneath areas which have been surcharged. In the remaining areas, settlement is expected to be in the order of approximately 0.5% to 1% of the fill thickness over a period of 10 to 20 years for well compacted clay fill and less for granular filling.

## 9.6 Surcharge Construction, Placement and Monitoring

Following preparation and raising of site levels, the proposed surcharge should be constructed and monitored as detailed below:

- Fill Selection: The fill used to construct the controlled filling (and surcharge which should also be suitable for future use in construction of raising site levels in future stages of development), and should preferably consist of low plasticity, gravelly sandy clay with a low dispersion potential or non plastic granular material;
- Placement of Surcharge: The fill used for the surcharge should be block tipped, dozed and track-rolled in layers not exceeding 0.5 m, and 0.25 m for the outer 5.0 m/shoulders of the surcharge. The surface of the surcharge should be free draining and drum rolled, as should the final surface, at the end of each day, to seal the surface should rain be likely;
- settlement plates on a nominal grid spacing of 30 m should be installed on the ground surface following placement of the grid bridging layer and prior to placement of the controlled filling;
- Staging: It is recommended that DP review the final surcharge design and construction times to ensure the soft soils are allowed to strengthen before full height is reached. It is also recommended that the strength gain of the clays be checked (by CPTu) when the structural platform has reached 4.0 m in height. The settlement plate results will also provide an indication of potential strength gain in the clay soils;
- settlement plates should be monitored (by the client's surveyor) weekly during controlled filling and initial surcharge placement, increasing to fortnightly once full height is reached. Results of the survey should be regularly provided to DP for analysis and assessment; and
- monitoring/surveying of any adjacent structures and/or services should also be undertaken during controlled filling and surcharging to ensure significant movements of these structures are not occurring, if applicable.

  Given the potential magnitude of settlement, the top 5 m of the fill platform should be placed in a controlled manner, as structural fill, as described in section 9.4 of this report. Subsequent filling may be placed as surcharge as described above. Once the initial 4 m of fill is placed, results of settlement plate monitioring and additional field work, as discussed, will determine the thickness and timing on when additional surcharge can be placed. Once it has been determined that surcharge can be placed without causing shear of the underlying compressible soils, the nominated thickness of surcahge will be provided. This process will be repeated as required. During the structaul fiill and surcharge placement, settlement plate monitoring should continue as described above.

## 9.7 Foundations and Site Classification

The investigation indicates that ground conditions at the proposed finished floor level following surcharging, are likely to comprise up to 4.0 m of controlled filling overlying alluvial clay and sand soils which would have strengthened through consolidation.

Foundation selection will need to consider future applied loads and acceptable settlements; however, shallow pad or strip footings bearing on controlled filling may be suitable for settlement-tolerant structures; otherwise piled foundations would be more suitable for heavily loaded, settlement-intolerant structures.

### 9.7.1 Site Classifications

Final site classifications will be a function of the soil profile and fill material properties that comprise the building pad at a given location. As these details were not known at the time of the preparation of this report, confirmation on site classifications can not be provided at this time. However, depending on tolerable movements and provided the material that comprises the structural pad is essentially a non plastic granular material, it is likely that rafts, of category 'H1'should be achievable. This assumes that all fill is placed in accordance with the recommendations made in this report and that appropriate consolidation settlement has been achieved.

Once final fill levels and targeted consolidation settlements have occurred, additional detailed field, laboratory testing, analysis and reporting should be undertaken to assess final site classifications at a given location.

### 9.7.2 High Level Footings

Provided that site preparation is carried out in accordance with the recommendations in this report, it is considered that lightly loaded, settlement-tolerant, upper level footings (pads or strips) to a maximum width of 1 m founded a minimum of 0.3 m into the controlled filling, can be designed using the allowable value of 100 kPa

For high level pad or strip footings founded in the controlled filling it is considered that settlements under such applied loading will be less than 1 % of footing width (ie. approximately 20 mm for 2 m wide pads).

### 9.7.3 Piled Foundations

Pile foundations may be used to support settlement-sensitive or highly loaded structures. The most suitable pile types are expected to be driven timber or precast concrete piles founded in the weathered sandstone as encountered below 19 m and 23 m depth in Bores 4 and 7 respectively.

Where piles are driven to refusal with the hammers regularly used by piling contractors (ie. 'Junttan' or 'Banut' rigs), the capacity of the pile can be considered as a structural member. The design ultimate resistances given in Table 6 are commonly assigned when precast concrete piles are driven to refusal using the abovementioned piling hammers and when the capacity is verified by dynamic testing.

**Table 6: Precast Concrete Pile Capacities**

| Pile Section | Factored Single Pile Design Ultimate Resistance * |
|---|---|
| 275 mm x 275 mm | 1000 – 1200 kN |
| 350 mm x 350 mm | 2200-2500 kN |
| 400 mm x 400 mm | 3200-3500 kN |

Note i) * assuming no eccentricity of load
ii) assumes f' c of 60 mPa

Some predrilling may be required through the existing fill in order to avoid piles *'skewing'* off line or becoming '*hung up'*.

As an alternative to driven precast piles, cast insitu continuous flight auger (CFA) piles could be considered, although these are likely to be significantly more expensive and experience indicates there is a potential for difficulties in socketing in the rock at depth and achieving satisfactory base cleanliness. The ultimate geotechnical strength ($R_{d,ug}$) of CFA piles can be calculated using the unfactored, ultimate shaft adhesion and end bearing values given in Table 7.

**Table 7: CFA Pile Design Parameters**

| Material Description | Unfactored Ultimate Shaft Friction - Compression (kPa) | Unfactored Ultimate End Bearing (kPa) |
|---|---|---|
| soft silty clay | 10 | NR |
| Firm or stronger clay | 50 | 450 |
| Loose medium dense sand(II) | 50 | 1500 |
| Extremely low strength sandstone | 125 | 1750 |

Note i) NR – Not recommended.
II) Where encountered at depths of greater than 10 m below existing site level.

Where limit state methods are used to design the piles, the ultimate unfactored geotechnical strength ($R_{d,ug}$) must be multiplied by a suitable geotechnical strength reduction factor ($\phi_g$) to obtain the design geotechnical strength ($R_{d,g}$). The geotechnical strength reduction factor is dependent upon several factors which were unknown at the time of preparation of this report, including incorporation of pile testing into the construction sequence (if any) and method of pile testing. As a guide, where the average risk rating is assessed to be high, there is no pile testing and the system has low redundancy, a $\phi_g$ value of 0.45 would apply. Guidance on the choice of $\phi_g$ factor is provided in Section 4 of AS2159–2009 (Ref 2).

If working stress methods are used in pile design, the ultimate geotechnical strength ($R_{d,ug}$) values given in Table 7 will need to be divided by a factor of safety of 2.5 to calculate the maximum single pile working load.

Suitable pile types and pile capacities should be confirmed by the piling contractor. It would ultimately be the piling contractor's responsibility to confirm the pile capacity and pile length for the pile type proposed.

### 9.7.4 Negative Skin Friction

Due to the potential for consolidation settlement of the loose sands and compressible soft clays, it is recommended that allowance be made for the effects of negative skin friction on the shafts of piles; however, this would only apply to areas which have not been surcharged in accordance with the recommendations made in this report. It is therefore recommended that, should piled structures be proposed on this site, advice is sought with regard to negative skin friction from DP prior to design.

### 9.7.5 Verification of Design Pressures

It is essential that foundation excavations (where applicable) be inspected by experienced geo-technical personnel to ensure the design parameters adopted are suitable for the ground conditions and to ensure that there is no soft or loose material remaining at the base of the excavations or smear on the side walls. Ground conditions can vary, and it is essential that adequate provision be made throughout the project to vary foundations to suit differing ground conditions.

## 10. References

1. Australian Standard AS 3798 – 2008 *'Guidelines on Earthworks for Commercial and Residential Developments*', Standards Australia.
2. Australian Standard AS 2159 – 2009 *'Piling – Design and Installation*', Standards Association of Australia

## 11. Limitations

DP has prepared this report for this project at the Southern Special Purpose Precinct, Cobaki in accordance with DP's Proposal GLD130091 dated 17 May 2013. This report is provided for the exclusive use of Leda Developments Pty Ltd and their consulting engineers for this project only and for the purpose(s) described in the report. It should not be used for other projects or by a third party. In preparing this report, DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the components set out in this report and to their

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# Appendix A

About this Report

# About this Report

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## Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

## Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

## Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

## Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

## Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

# About this Report

## Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

## Information for Contractual Purposes

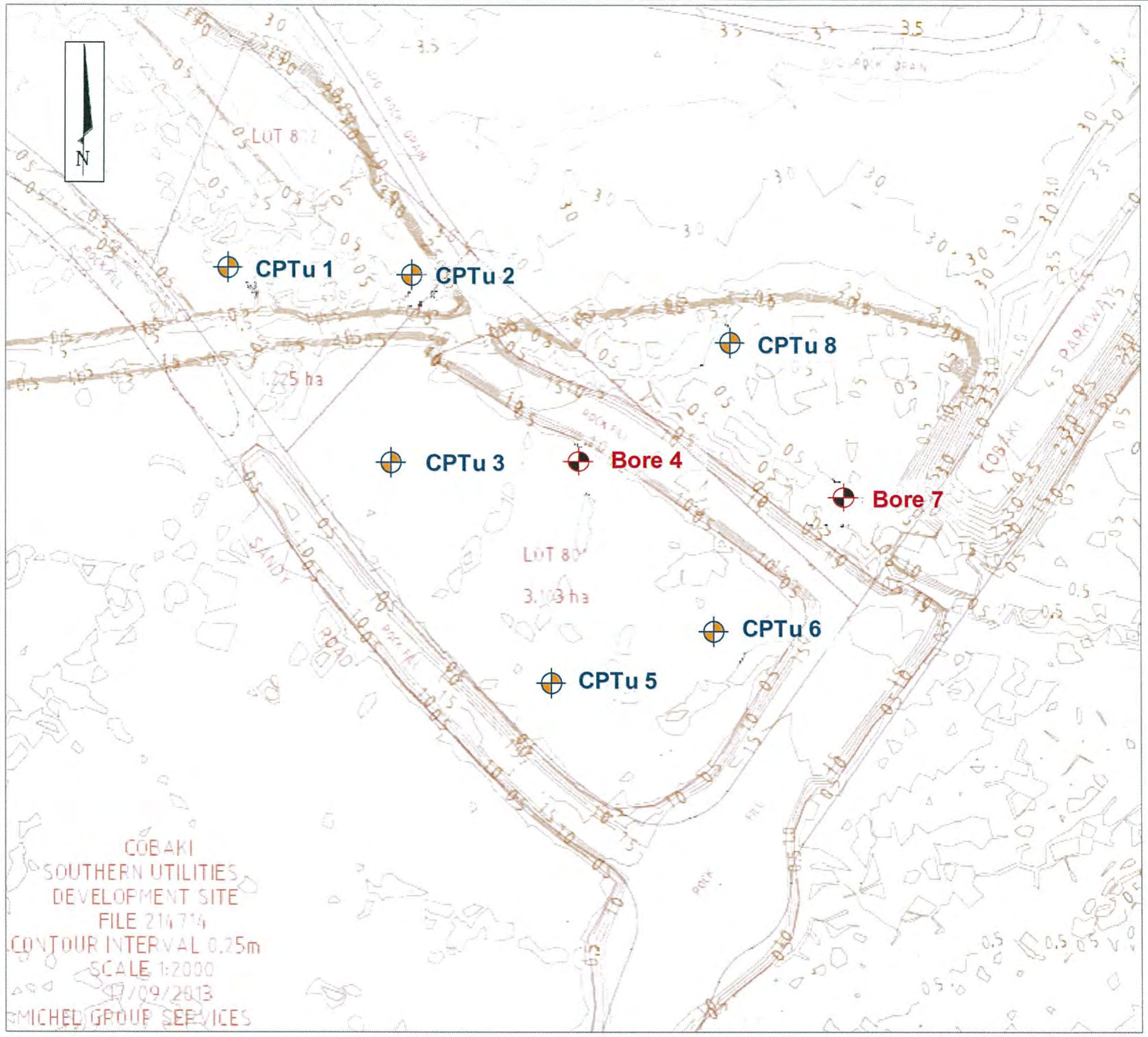
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

## Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

# Appendix B

Drawings 1 to 3





Site Locality

**Legend:**

- Borehole number and location
- CPT number and location

**Notes:**

1. Test locations are approximate only and are shown with reference to existing and proposed site features.
2. Drawing Cobaki Utilities Southern Site Drawing provided by the client

GPS co-ordinates:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CPTu 1:** | E 47219.9523<br>N 81234.6877 | **CPTu 2:** | E 47300.1053<br>N 81230.9084 | **CPTu 3:** | E 47292.1153<br>N 81150.7293 | **Bore 4:** | E 47373.0445<br>N 811505429 |
| **CPTu 5:** | E 47360.0510<br>N 81055.0718 | **CPTu 6:** | E 47432.5889<br>N 81077.2487 | **Bore 7:** | E 47485.9405<br>N 81135.9908 | **CPTu 8:** | E 47436.3441<br>N 81202.6136 |

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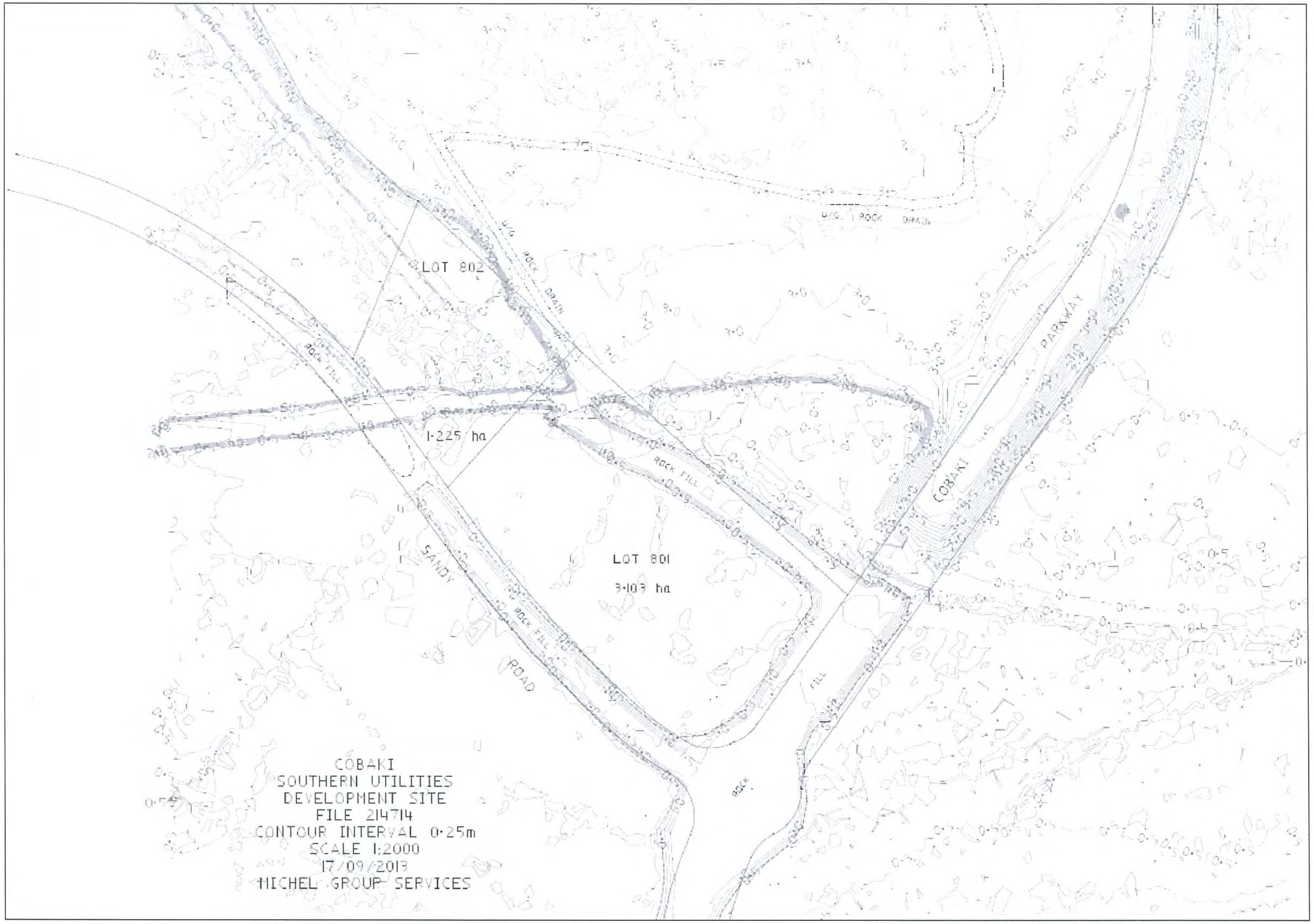
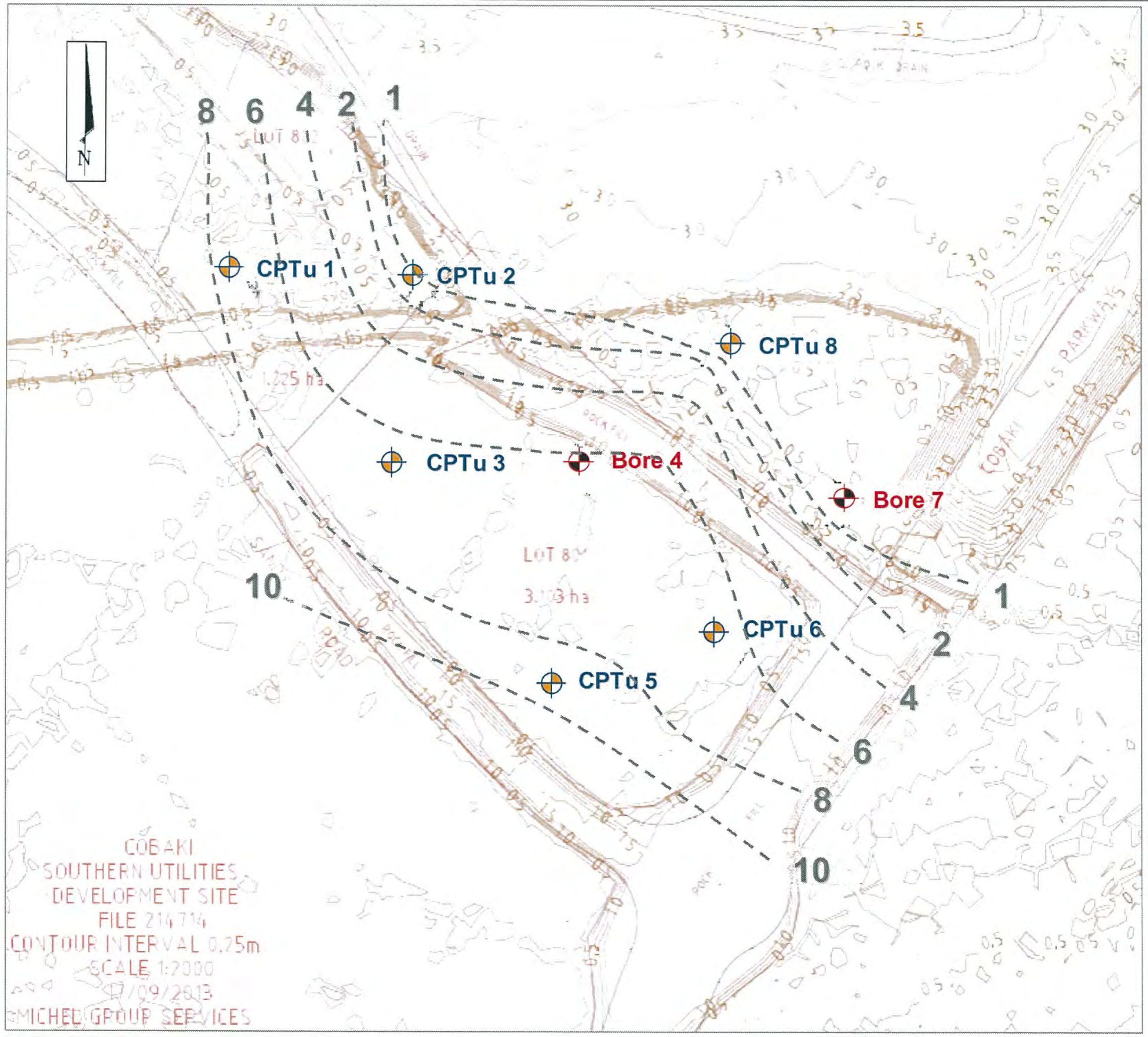
| | | | |
|---|---|---|---|
| CLIENT: | Leda Development Pty Ltd | | |
| OFFICE: | Gold Coast | DRAWN BY: | OD |
| SCALE: | As shown | DATE: | Oct 2013 |

TITLE: **Test Location Plan**
**Proposed Southern Special Purpose Precinct**
**Corner Sandy Lane and Cobaki Parkway, Cobaki Lakes**

| | |
|---|---|
| PROJECT No: | 82508.01 |
| DRAWING No: | 1 |
| REVISION: | 0 |




| | | | | |
|---|---|---|---|---|
| **Douglas Partners** Geotechnics / Environment / Groundwater | CLIENT: Leda Developments Pty Ltd | | TITLE: **Contour Plan** | PROJECT No: 82508.01 |
| | OFFICE: Gold Coast | DRAWN BY: OD | **Proposed Southern Special Purpose Precinct** | DRAWING No: 2 |
| | SCALE: As shown | DATE: Nov 2013 | **Corner Sandy Lane and Cobaki Parkway, Cobaki Lakes** | REVISION: 0 |





Site Locality

**Legend:**

- Borehole number and location
- CPT number and location
- Inferred soft compressible clay thickness (m)

**Notes:**

1. Test locations are approximate only and are shown with reference to existing and proposed site features.
2. Drawing Cobaki Utilities Southern Site Drawing provided by the client

GPS co-ordinates:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **CPTu 1:** | E 47219.9523<br>N 81234.6877 | **CPTu 2:** | E 47300.1053<br>N 81230.9084 | **CPTu 3:** | E 47292.1153<br>N 81150.7293 | **Bore 4:** | E 47373.0445<br>N 811505429 |
| **CPTu 5:** | E 47360.0510<br>N 81055.0718 | **CPTu 6:** | E 47432.5889<br>N 81077.2487 | **Bore 7:** | E 47485.9405<br>N 81135.9908 | **CPTu 8:** | E 47436.3441<br>N 81202.6136 |

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| | | | |
|---|---|---|---|
| CLIENT: | Leda Development Pty Ltd | | |
| OFFICE: | Gold Coast | DRAWN BY: | OD |
| SCALE: | As shown | DATE: | Oct 2013 |

TITLE: **Inferred Soft Compressible Clay Contour Plan**
**Proposed Southern Special Purpose Precinct**
**Corner Sandy Lane and Cobaki Parkway, Cobaki Lakes**

| | |
|---|---|
| PROJECT No: | 82508.01 |
| DRAWING No: | 3 |
| REVISION: | 0 |

# Appendix C

Results of Field Work

# BOREHOLE LOG

**CLIENT:** Leda Developments Pty Ltd
**PROJECT:** Southern Special Purpose Precinct
**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki Lakes

**SURFACE LEVEL:** --
**EASTING:** 473730495
**NORTHING:** 811505429
**DIP/AZIMUTH:** 90°/--

**BORE No:** 4
**PROJECT No:** 82508.01
**DATE:** 9/10/2013
**SHEET** 1 OF 2

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| | | PAD FILLING | | | | | | | |
| | 1.0 | CLAY - very soft, brown, slightly silty clay. Wet. Abundant live rootlets | | | | | | | |
| | | | | $U_{50}$ | 1.5–1.9 | | pp = 0<br>Push tube sank under own weight | | |
| | | ....becoming soft with trace of shell fragments | | $U_{50}$ | 3.0–3.4 | | pp = 40 | | |
| | | ....with trace sand | | $U_{50}$ | 4.5–4.9 | | pp = 35 | | |
| | | ....with trace of fine, angular siltstone gravel | | | | | | | |
| | | ....becoming firm | | $U_{50}$ | 6.0–6.4 | | pp = 55 | | |
| | | ....with trace wood fragments | | $U_{50}$ | 7.5–7.9 | | pp = 80 | | |
| | 9.0 | ....becoming stiff, grey sandy clay, with some partially decayed leaf matter | | $U_{50}$ | 9.0–9.4 | | pp = 130 | | |

**RIG:** GOT 2008 **DRILLER:** DE Drilling **LOGGED:** AS **CASING:** 1.3m
**TYPE OF BORING:** Auger 0.0 - 0.75, washbore 0.75 - 19.5m
**WATER OBSERVATIONS:** Water observed at 1.0m
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | | | |
|---|---|---|---|---|---|
| A | Auger sample | G | Gas sample | PID | Photo ionisation detector (ppm) |
| B | Bulk sample | P | Piston sample | PL(A) | Point load axial test Is(50) (MPa) |
| BLK | Block sample | $U_x$ | Tube sample (x mm dia.) | PL(D) | Point load diametral test Is(50) (MPa) |
| C | Core drilling | W | Water sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | ▷ | Water seep | S | Standard penetration test |
| E | Environmental sample | ▼ | Water level | V | Shear vane (kPa) |

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# BOREHOLE LOG

**CLIENT:** Leda Developments Pty Ltd
**PROJECT:** Southern Special Purpose Precinct
**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki Lakes

**SURFACE LEVEL:** --
**EASTING:** 473730495
**NORTHING:** 811505429
**DIP/AZIMUTH:** 90°/--

**BORE No:** 4
**PROJECT No:** 82508.01
**DATE:** 9/10/2013
**SHEET** 2 OF 2

| RL | Depth (m) | Description of Strata | Graphic Log | Sampling & In Situ Testing: Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| | | ....becoming stiff, grey sandy clay, with some partially decayed leaf matter *(continued)* <br> ....becoming light grey and orange brown in colour <br> ....with trace of stiff light grey clay | | $U_{50}$ | 10.5 – 10.9 | | Not enough sample recovered to test | | |
| | 11 | ....with trace of fragments | | | | | | | 11 |
| | 12 12.0 | CLAY - very stiff, orange brown and pale grey clay with trace fine sand | | $U_{50}$ | 12.0 – 12.4 | | pp = 250 | | 12 |
| | 13 | | | | | | | | 13 |
| | | | | $U_{50}$ | 13.5 – 13.9 | | pp = 310 | | |
| | 14 | | | | | | | | 14 |
| | 14.5 | ....with trace of black silty clay | | | | | | | |
| | 15 15.0 | CLAY - very stiff, dark brown and black silty clay with some wood fragments and organic odour | | $U_{50}$ | 15.0 – 15.4 | | pp = 320 | | 15 |
| | 15.8 | ....becoming grey | | | | | | | |
| | 16 | | | | | | | | 16 |
| | | | | $U_{50}$ | 16.5 – 16.9 | | pp = 210 | | |
| | 17 | | | | | | | | 17 |
| | 18 18.0 | SAND - loose, blue grey, medium grained sand with some clay | | $U_{50}$ | 18.0 – 18.4 | | Sample tube sank under own weight | | 18 |
| | 19 19.0 | SANDSTONE - extremely low strength, grey, sandstone | | | | | | | 19 |
| | 19.503 | - low strength <br> Bore discontinued at 19.5m | | S | 19.5 – 19.53 | | 30 refusal | | |

**RIG:** GOT 2008 **DRILLER:** DE Drilling **LOGGED:** AS **CASING:** 1.3m

**TYPE OF BORING:** Auger 0.0 - 0.75, washbore 0.75 - 19.5m

**WATER OBSERVATIONS:** Water observed at 1.0m

**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | | | |
|---|---|---|---|---|---|
| A | Auger sample | G | Gas sample | PID | Photo ionisation detector (ppm) |
| B | Bulk sample | P | Piston sample | PL(A) | Point load axial test Is(50) (MPa) |
| BLK | Block sample | $U_x$ | Tube sample (x mm dia.) | PL(D) | Point load diametral test Is(50) (MPa) |
| C | Core drilling | W | Water sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | ▷ | Water seep | S | Standard penetration test |
| E | Environmental sample | ▼ | Water level | V | Shear vane (kPa) |

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# BOREHOLE LOG

**CLIENT:** Leda Developments Pty Ltd
**PROJECT:** Southern Special Purpose Precinct
**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki Lakes

**SURFACE LEVEL:** +1.0 m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 7
**PROJECT No:** 82508.01
**DATE:** 10/10/2013
**SHEET** 1 OF 3

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| | | PLATFORM FILLING | | | | | | | |
| 0 | 1.0 | TOPSOIL - soft, brown and grey, slightly sandy, silty clay topsoil | | | | | | | |
| | 1.5 | CLAY - very soft, dark brown and grey, slightly silty, clay. Wet. Abundant live rootlets | | | | | | | |
| -1 | 1.7 | SAND - dense, (indurated) brown and red brown, silty, fine grained sand. | | S | 2.0 – 2.25 | | 27,30 refusal | | |
| -3 | | ....becoming medium dense and orange brown | | S | 3.5 – 3.95 | | 5,10,9 N = 19 | | |
| -4 | 4.5 | CLAY - firm, light grey, slightly silty clay | | $U_{50}$ | 5.0 – 5.4 | | pp = 160 | | |
| -6 | | ....becoming very stiff and light blue grey | | $U_{50}$ | 6.5 – 6.9 | | pp = 250 | | |
| -7 | | ....becoming orange brown mottled light blue | | U | 8.0 – 8.4 | | pp = 290 | | |
| -8 | | ....becoming stiff | | S | 9.45 – 9.5 | | 4,6,10 N = 16 | | |

**RIG:** GOT 2008 **DRILLER:** DE Drilling **LOGGED:** AS **CASING:** 1.3m
**TYPE OF BORING:** Auger 0.0 - 0.75, washbore 0.75 - 24.5m
**WATER OBSERVATIONS:**
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | | | |
|---|---|---|---|---|---|
| A | Auger sample | G | Gas sample | PID | Photo ionisation detector (ppm) |
| B | Bulk sample | P | Piston sample | PL(A) | Point load axial test Is(50) (MPa) |
| BLK | Block sample | $U_x$ | Tube sample (x mm dia.) | PL(D) | Point load diametral test Is(50) (MPa) |
| C | Core drilling | W | Water sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | ▷ | Water seep | S | Standard penetration test |
| E | Environmental sample | ▼ | Water level | V | Shear vane (kPa) |

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# BOREHOLE LOG

**CLIENT:** Leda Developments Pty Ltd
**PROJECT:** Southern Special Purpose Precinct
**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki Lakes

**SURFACE LEVEL:** +1.0 m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 7
**PROJECT No:** 82508.01
**DATE:** 10/10/2013
**SHEET** 2 OF 3

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| -9 | | CLAY - firm, light grey, slightly silty clay *(continued)* | | | | | | | |
| -10 | 11 | ....with some fine and medium gravel sized inclusions of very low strength sandstone | | S | 11.0 - 11.45 | | 3,5,6 N = 11 | | 11 |
| -11 | 12 12.0 | CLAY - firm, dark grey, silty clay with some decayed organic matter | | S | 12.5 - 12.95 | | 1,3,4 N = 7 | | 12 |
| -12 | 13 | | | | | | | | 13 |
| -13 | 14 | ....becoming stiff, blue grey mottled, orange brown and red brown clay | | S | 14.0 - 14.45 | | 5,8,12 N = 20 | | 14 |
| -14 | 15 | | | S | 15.5 - 15.95 | | 3,5,8 N = 13 | | 15 |
| -15 | 16 | | | | | | | | 16 |
| -16 | 17 | | | S | 17.0 - 17.45 | | 5,6,10 N = 16 | | 17 |
| -17 | 18 | ....becoming pale brown<br>....becoming red brown and grey | | S | 18.5 - 18.95 | | 1,6,7 N = 13 | | 18 |
| -18 | 19 | | | | | | | | 19 |
| | 20.0 | | | | | | | | |

**RIG:** GOT 2008 **DRILLER:** DE Drilling **LOGGED:** AS **CASING:** 1.3m
**TYPE OF BORING:** Auger 0.0 - 0.75, washbore 0.75 - 24.5m
**WATER OBSERVATIONS:**
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | | | |
|---|---|---|---|---|---|
| A | Auger sample | G | Gas sample | PID | Photo ionisation detector (ppm) |
| B | Bulk sample | P | Piston sample | PL(A) | Point load axial test Is(50) (MPa) |
| BLK | Block sample | $U_x$ | Tube sample (x mm dia.) | PL(D) | Point load diametral test Is(50) (MPa) |
| C | Core drilling | W | Water sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | ▷ | Water seep | S | Standard penetration test |
| E | Environmental sample | ▼ | Water level | V | Shear vane (kPa) |

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# BOREHOLE LOG

**CLIENT:** Leda Developments Pty Ltd
**PROJECT:** Southern Special Purpose Precinct
**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki Lakes

**SURFACE LEVEL:** +1.0 m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 7
**PROJECT No:** 82508.01
**DATE:** 10/10/2013
**SHEET** 3 OF 3

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| -19 | | CLAY - stiff, orange brown and blue grey, slightly sandy clay. Sand fraction is fine grained | | S | 20.0 – 20.45 | | 5,5,5 N = 10 | | |
| -20 | 21 | | | | | | | | 21 |
| | 21.5 | SAND - medium dense, orange brownfine to coarse grained sand | | S | 21.5 – 21.95 | | 6,9,14 N = 23 | | |
| -21 | 22 | | | | | | | | 22 |
| -22 | 23 23.0 | SANDSTONE - extremely low strength to very low strength, grey, fine to coarse grained sandstone | | S | 23.0 – 23.25 | | 24,30 refusal | | 23 |
| -23 | 24 | | | | | | | | 24 |
| | | | | S | 24.5 – 24.75 | | 17,30 refusal | | |
| | 24.75 | Bore discontinued at 24.75m | | | | | | | |
| -24 | 25 | | | | | | | | 25 |
| -25 | 26 | | | | | | | | 26 |
| -26 | 27 | | | | | | | | 27 |
| -27 | 28 | | | | | | | | 28 |
| -28 | 29 | | | | | | | | 29 |

**RIG:** GOT 2008 **DRILLER:** DE Drilling **LOGGED:** AS **CASING:** 1.3m
**TYPE OF BORING:** Auger 0.0 - 0.75, washbore 0.75 - 24.5m
**WATER OBSERVATIONS:**
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | | | |
|---|---|---|---|---|---|
| A | Auger sample | G | Gas sample | PID | Photo ionisation detector (ppm) |
| B | Bulk sample | P | Piston sample | PL(A) | Point load axial test Is(50) (MPa) |
| BLK | Block sample | $U_x$ | Tube sample (x mm dia.) | PL(D) | Point load diametral test Is(50) (MPa) |
| C | Core drilling | W | Water sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | ▷ | Water seep | S | Standard penetration test |
| E | Environmental sample | ▼ | Water level | V | Shear vane (kPa) |

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# CONE PENETRATION TEST

**CLIENT:** Leda Developments Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:**

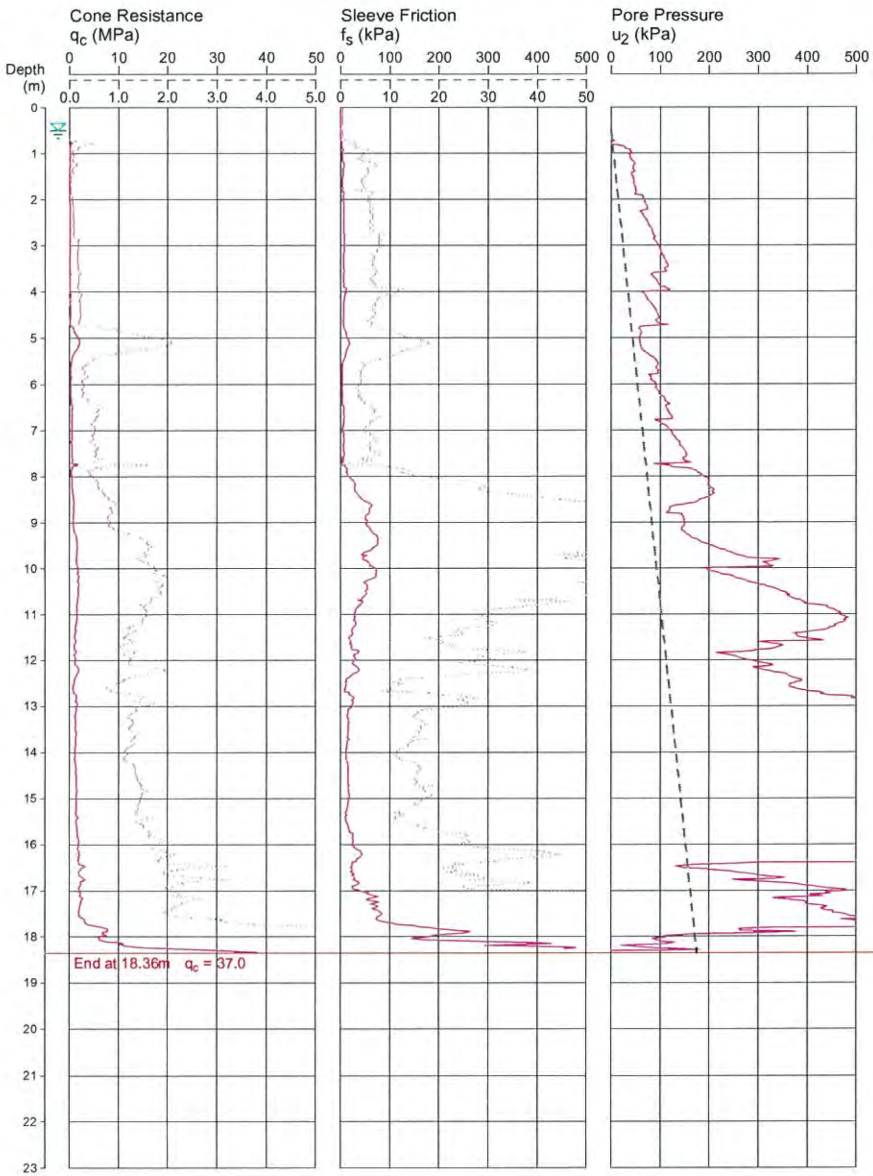
**COORDINATES:** 47219.9523E 81234.6877N

## CPTu 01

Page 1 of 1

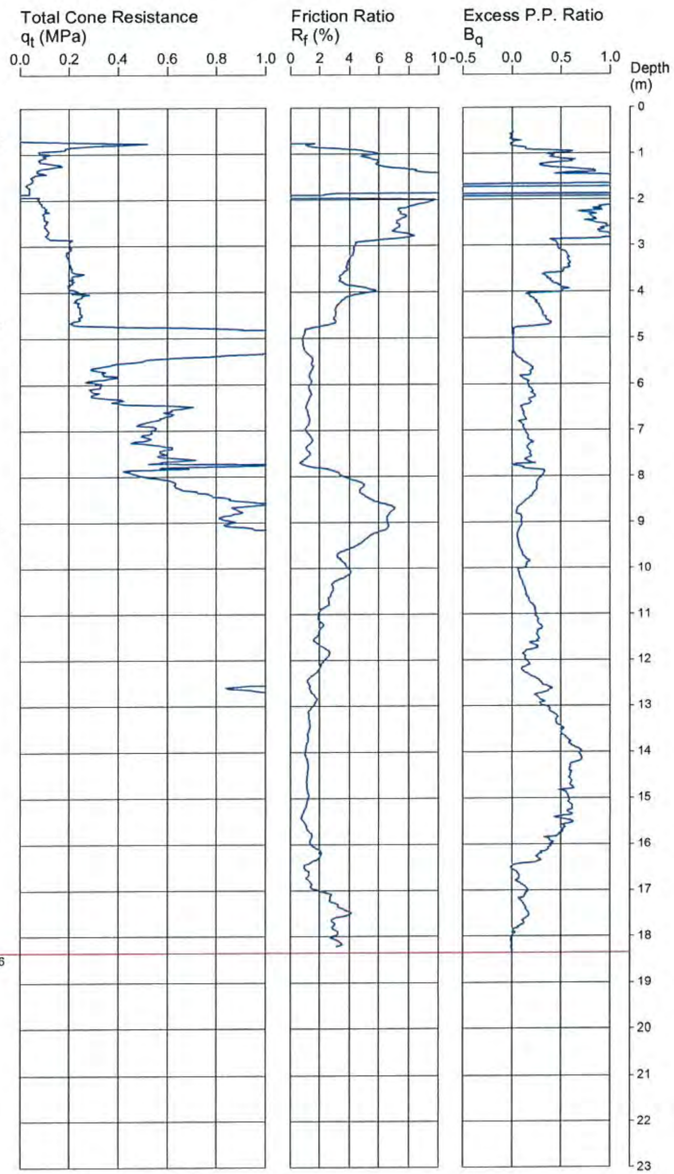
**DATE** 3/10/2013

**PROJECT No:** 82508.01




Soil Behaviour Type

| Depth (m) | Soil Behaviour Type |
|---|---|
| 0.70 | PAD FILLING |
| 4.70 | CLAY - slightly silty clay, very soft to soft |
| 9.10 | SILTY CLAY - soft to firm. Bands of sand throughout |
| 18.36 | SILTY CLAY: stiff<br>- grading very stiff<br>- grading stiff<br>- grading very stiff<br>- grading hard |




**REMARKS:** Dissipation Test at 4.0m

**Water depth after test: 0.50m depth (assumed)**

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82508-01 draft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2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# CONE PENETRATION TEST

**CLIENT:** Leda Developments Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:**

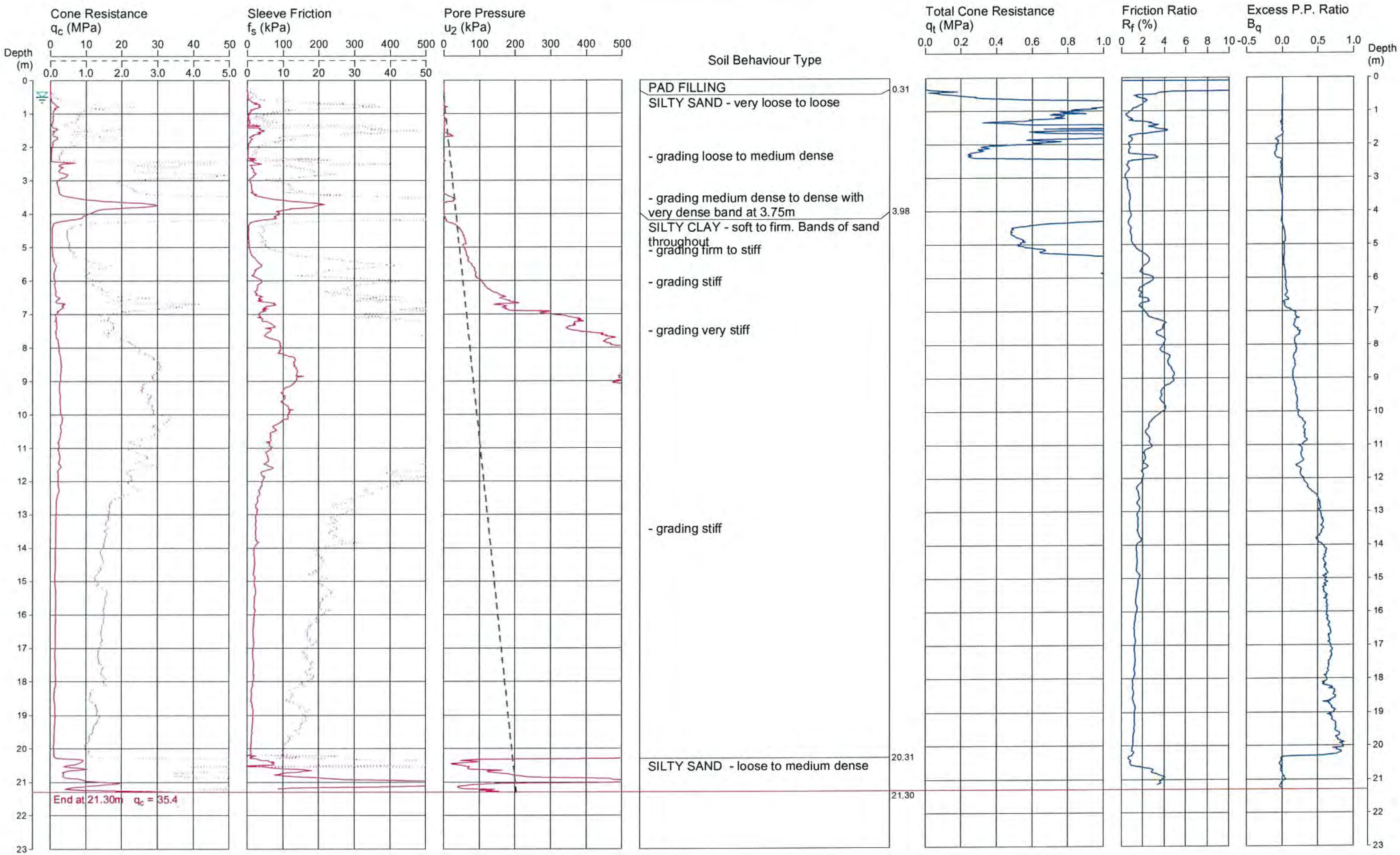
**COORDINATES:** 47300.1053E 81230.9084N

## CPTu 02

Page 1 of 1

**DATE** 4/10/2013

**PROJECT No:** 82508.01




**REMARKS:** Dessipation test at 7.52m

**Water depth after test: 0.50m depth (assumed)**

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82508-2A draft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2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# CONE PENETRATION TEST

**CLIENT:** Leda Developments Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:**

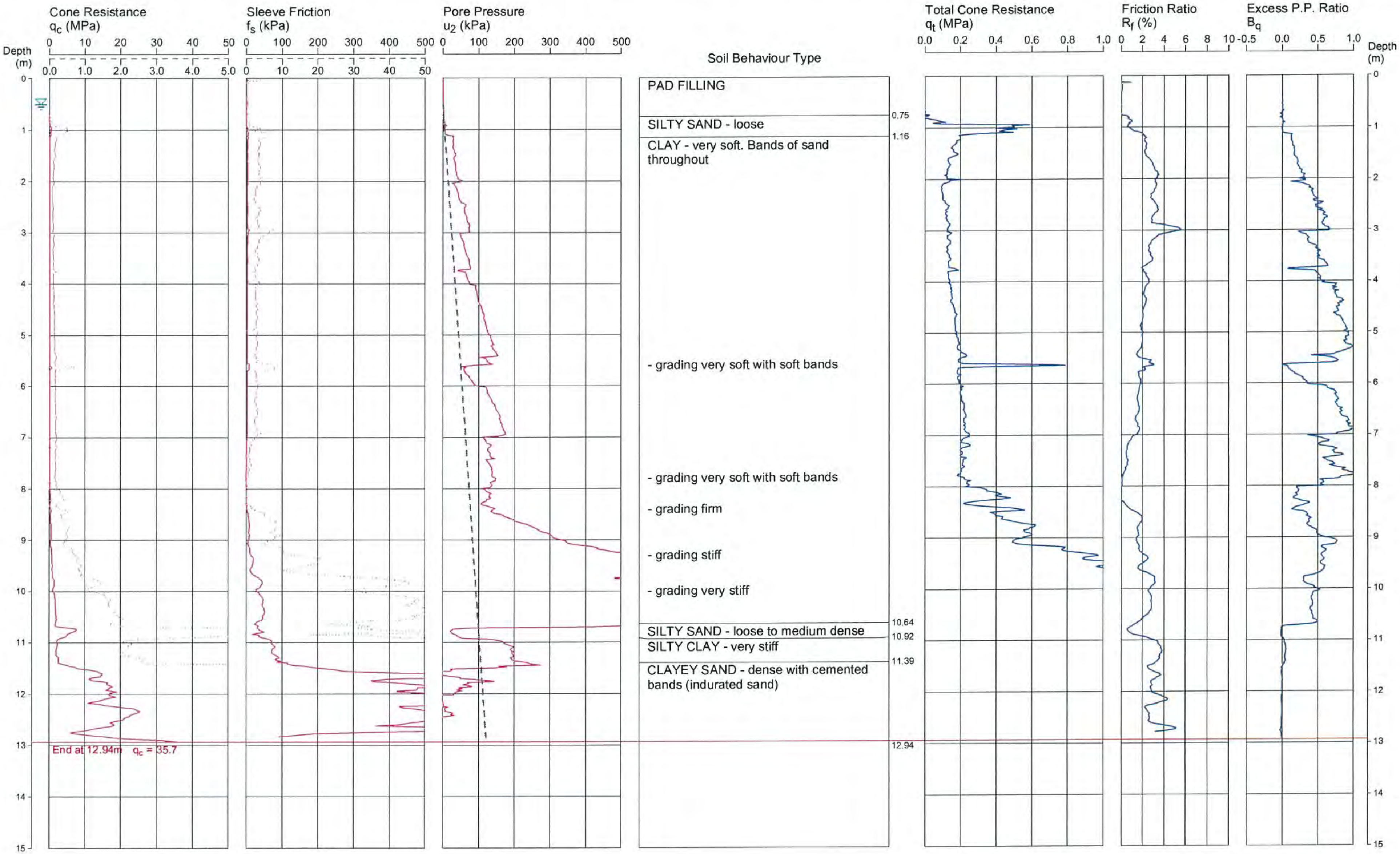
**COORDINATES:** 47292.1153E 81150.7293N

## CPTu 03

Page 1 of 1

**DATE** 4/10/2013

**PROJECT No:** 82508.01




**REMARKS:** Dissipation test at 3.02m

**Water depth after test:** 0.50m depth (assumed)

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82508-03 draft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2


Douglas Partners
Geotechnics / Environment / Groundwater

# CONE PENETRATION TEST

**CLIENT:** Leda Development Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:** 1.0m AHD

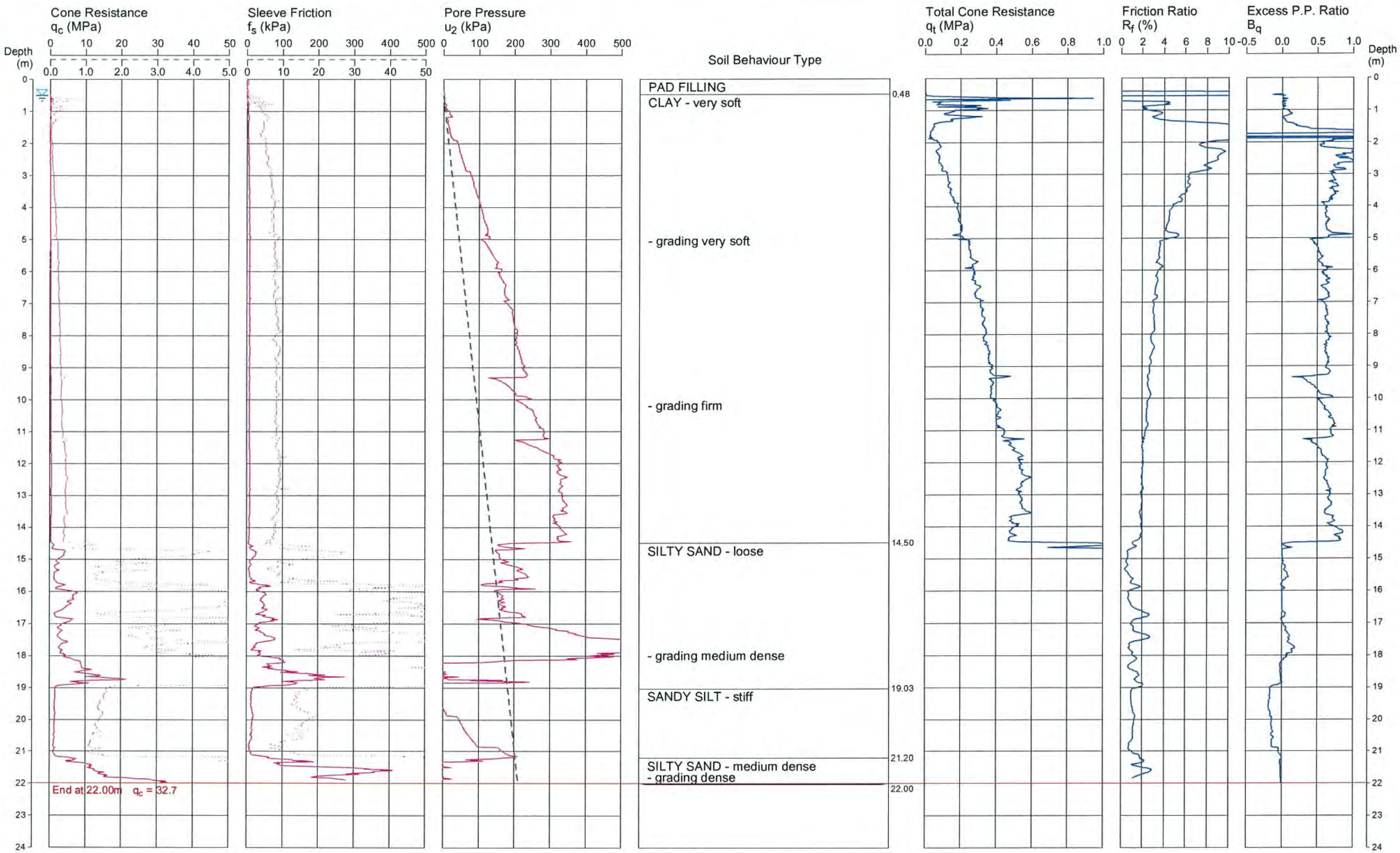
**COORDINATES:** 47360.051E 81055.0718N

# CPTu 05

Page 1 of 1

**DATE** 3/10/2013

**PROJECT No:** 82508.01




**REMARKS:**

**Water depth after test: 0.50m depth (assumed)**

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82508-05 draft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2



Douglas Partners
Geotechnics / Environment / Groundwater

# CONE PENETRATION TEST

**CLIENT:** Leda Developments Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:**

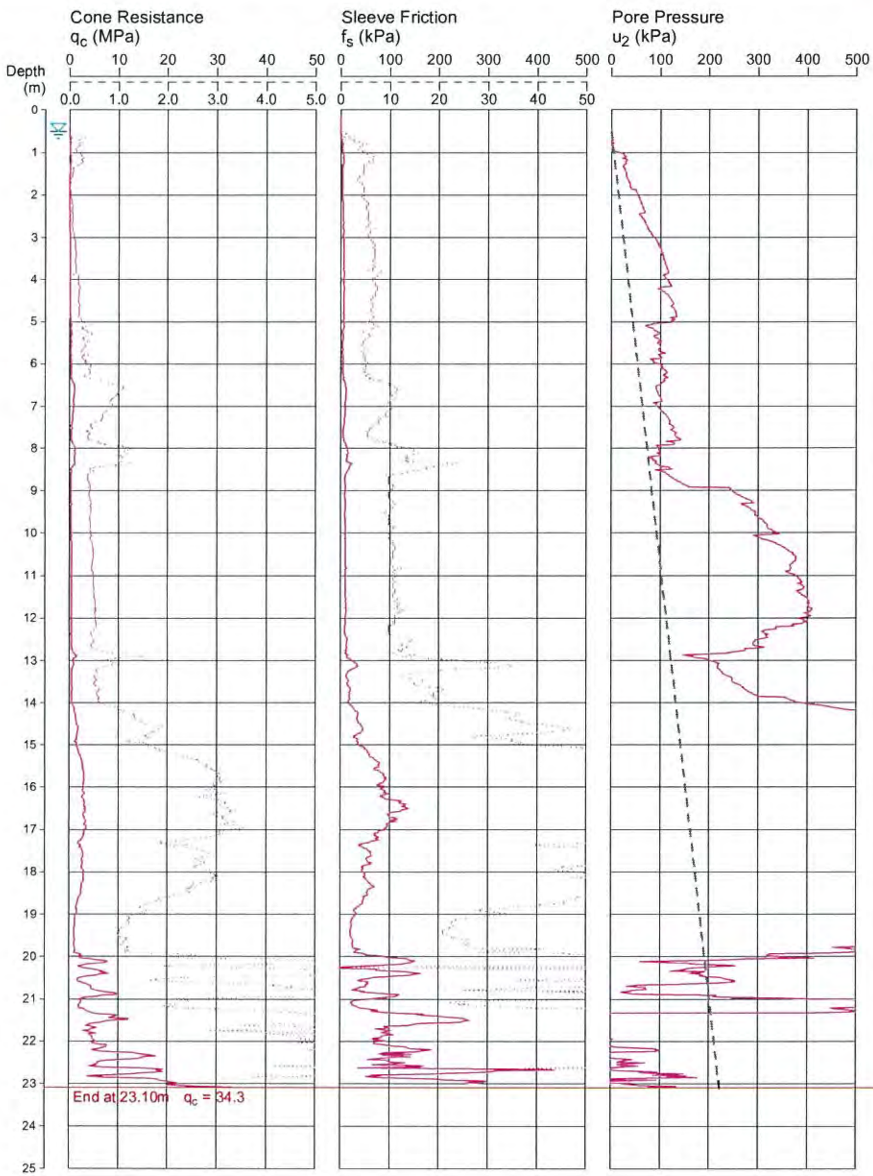
**COORDINATES:** 47432.5889E 81077.2487N

## CPTu 06

Page 1 of 1

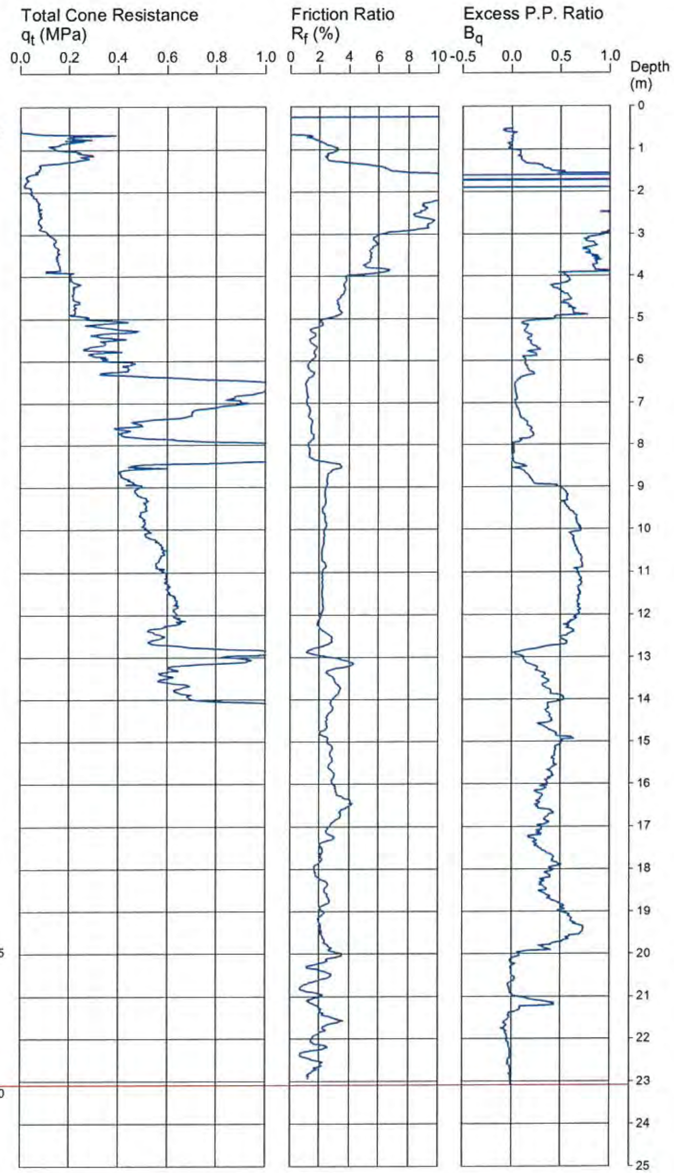
**DATE** 3/10/2013

**PROJECT No:** 82508.01




Soil Behaviour Type

| Depth (m) | Soil Behaviour Type |
|---|---|
| | PAD FILLING |
| 0.60 | CLAY - very soft. Bands of sand throughout |
| | - grading soft |
| | - grading stiff |
| | - grading firm |
| | - grading stiff |
| | - grading soft |
| | grading firm |
| | - grading stiff |
| | - grading very stiff |
| | - grading stiff |
| 19.95 | SILTY SAND - loose to medium dense |
| | - grading medium dense |
| 23.10 | |




**REMARKS:** Dissipation Test at 8.52m

**Water depth after test:** 0.5m

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82508-06 draft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2


Douglas Partners
Geotechnics / Environment / Groundwater

# CONE PENETRATION TEST

**CLIENT:** Leda Developments Pty Ltd

**PROJECT:** Southern Special Purpose Precinct

**LOCATION:** Cnr Sandy Lane and Cobaki Parkway, Cobaki

**REDUCED LEVEL:**

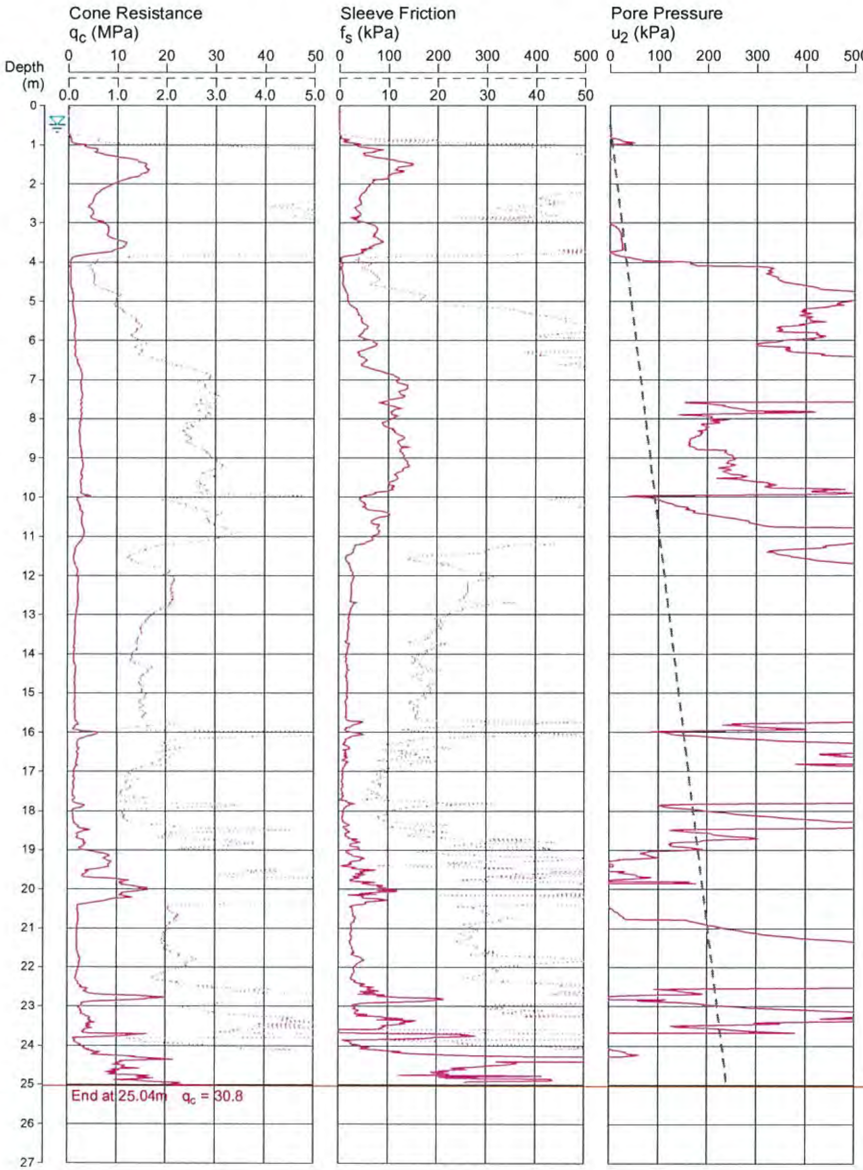
**COORDINATES:** 47436.3441E 81202.6136N

## CPTu 08

Page 1 of 1

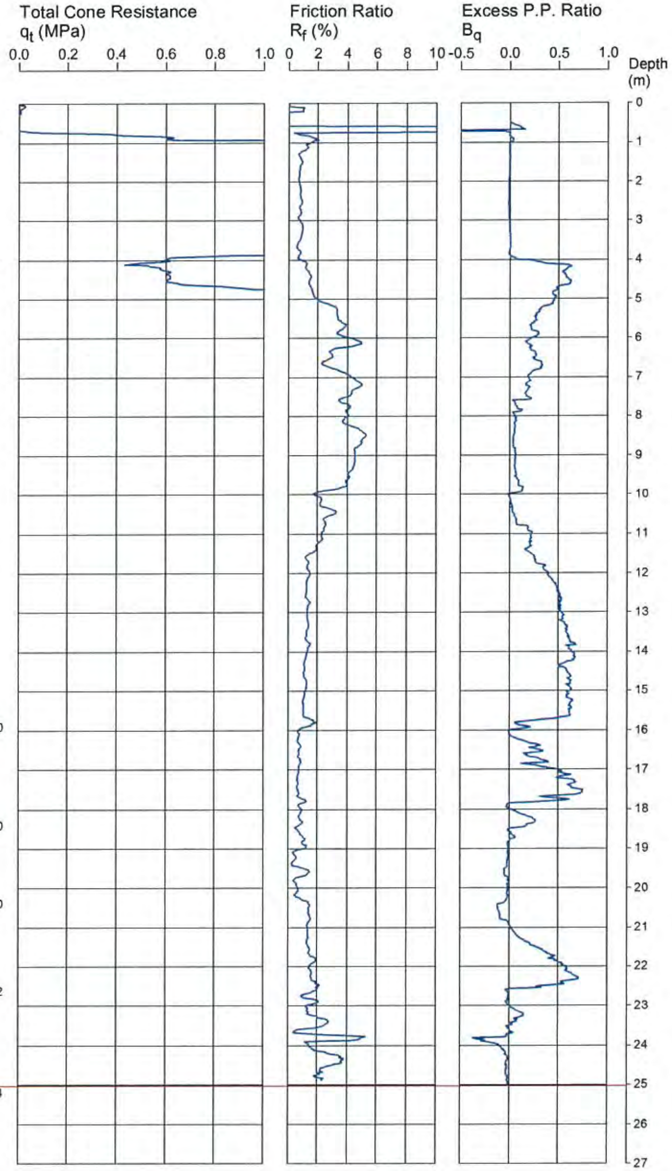
**DATE** 4/10/2013

**PROJECT No:** 82508.01




Soil Behaviour Type

| Soil Behaviour Type | Depth (m) |
|---|---|
| PAD FILLING | 0.71 |
| SAND - medium dense with some gravel | 3.95 |
| CLAY - soft to firm<br>- grading stiff<br><br>- grading very stiff<br><br>- band of clayey silt/sand silt 8.0m to 10.5m | 15.90 |
| SILTY CLAY - very stiff with some sand lenses | 18.40 |
| SILTY SAND - loose to medium dense | 20.40 |
| SILTY CLAY - very stiff | 22.62 |
| CLAYEY SILT - hard with some sand lenses | 25.04 |




**REMARKS:**

**Water depth after test:** 0.50m depth (assumed)

**File:** P:\Geotech Projects\82508.01, Cobaki, Southern School site\Field\CPT Warren\82545 CPT8 daft.CP5
**Cone ID:** CONE1624 **Type:** 5 Piezocone

ConePlot Version 5.9.2


Douglas Partners
Geotechnics / Environment / Groundwater

# Appendix D

Results of Laboratory Tests

Trilab
Soil Rock Calibration

**Brisbane**
346A Bilsen Road,
Geebung
QLD 4034
Ph: +61 7 3265 5656

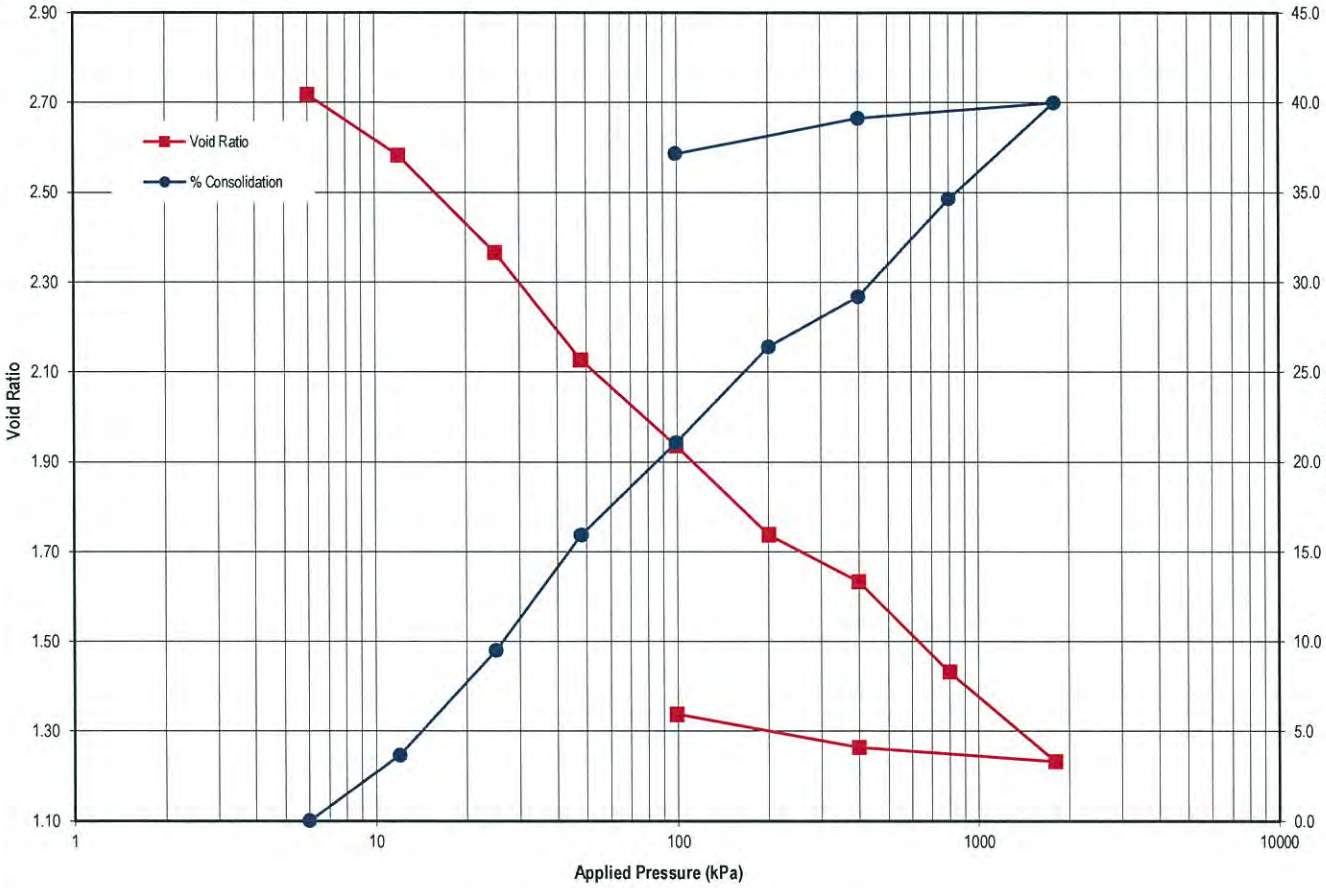
**Perth**
2 Kimmer Place,
Queens Park
WA 6107
Ph: +61 8 9258 8323

# OEDOMETER TEST REPORT

**Test Method: AS1289.6.6.1, 3.5.1**

| | | | |
|---|---|---|---|
| **Client:** | Douglas Partners Pty Ltd | **Report No.:** | 13100301-OED |
| **Project:** | Southern Special Purpose Precinct | **Test Date:** | 17/10/2013 |
| | | **Report Date:** | 6/11/2013 |
| **Client Id.:** | Bore 4 | **Depth (m):** | 1.50-1.90 |

**Description:** CLAY- grey




| | | | | | |
|---|---|---|---|---|---|
| Wet Density ($t/m^3$): | 1.45 | Initial Moisture (%): | 98.2 | Test Condition: Inundated on load | |
| Particle Density ($t/m^3$): | 2.73 | Initial Voids Ratio: | 2.720 | Initial Degree of Saturation (%): | 99.2 |
| Undisturbed sample supplied by the client | | Remarks: | Tested as Received | | Page 1 of 2 |

REP03102

Accredited for compliance with ISO/IES 17025.
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/National Standards.

Tested at Trilab Brisbane Laboratory.

**Authorised Signatory**

C. Channon

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

**Laboratory Number 9926**

Trilab
Soil Rock Calibration

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Ph: +61 7 3265 5656

**Perth**
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Queens Park
WA 6107
Ph: +61 8 9258 8323

# OEDOMETER TEST REPORT

**Test Method: AS1289.6.6.1, 3.5.1**

| | | | |
|---|---|---|---|
| **Client:** | Douglas Partners Pty Ltd | **Report No.:** | 13100301-OED |
| **Project:** | Southern Special Purpose Precinct | **Test Date:** | 17/10/2013 |
| | | **Report Date:** | 6/11/2013 |
| **Client Id.:** | Bore 4 | **Depth (m):** | 1.50-1.90 |
| **Description:** | CLAY- grey | | |

## *TEST RESULTS*

| Stage | Load (kPa) | Cc | Cv ($m^2$/yr) $t_{50}$ | Cv ($m^2$/yr) $t_{90}$ | Mv ($kPa^{-1}x10^{-3}$) | $C_a$ x $10^{-3}$ | % Consolidation |
|---|---|---|---|---|---|---|---|
| 1 | 6-12 | 0.451 | 0.14 | 0.32 | 6.081 | 9.27 | 3.6 |
| 2 | 12-25 | 0.681 | 0.27 | 0.50 | 4.659 | 13.11 | 9.5 |
| 3 | 25-48 | 0.843 | 0.30 | 0.47 | 3.087 | 11.77 | 15.9 |
| 4 | 48-99 | 0.607 | 1.47 | 1.26 | 1.198 | 14.70 | 21.0 |
| 5 | 99-201 | 0.644 | 0.44 | 0.64 | 0.661 | 10.02 | 26.4 |
| 6 | 201-401 | 0.347 | 1.16 | 2.12 | 0.190 | 8.27 | 29.2 |
| 7 | 401-801 | 0.672 | 0.56 | 1.51 | 0.192 | 10.72 | 34.6 |
| 8 | 801-1799 | 0.568 | 0.91 | 1.78 | 0.082 | 9.97 | 40.0 |
| 9 | 1799-401 | 0.048 | 1.75 | 8.98 | 0.010 | 1.44 | 39.1 |
| 10 | 401-99 | 0.122 | 0.32 | 0.62 | 0.108 | 4.18 | 37.1 |

Remarks: Tested as Received



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Tested at Trilab Brisbane Laboratory.

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Laboratory Number 9926

Trilab
Soil Rock Calibration

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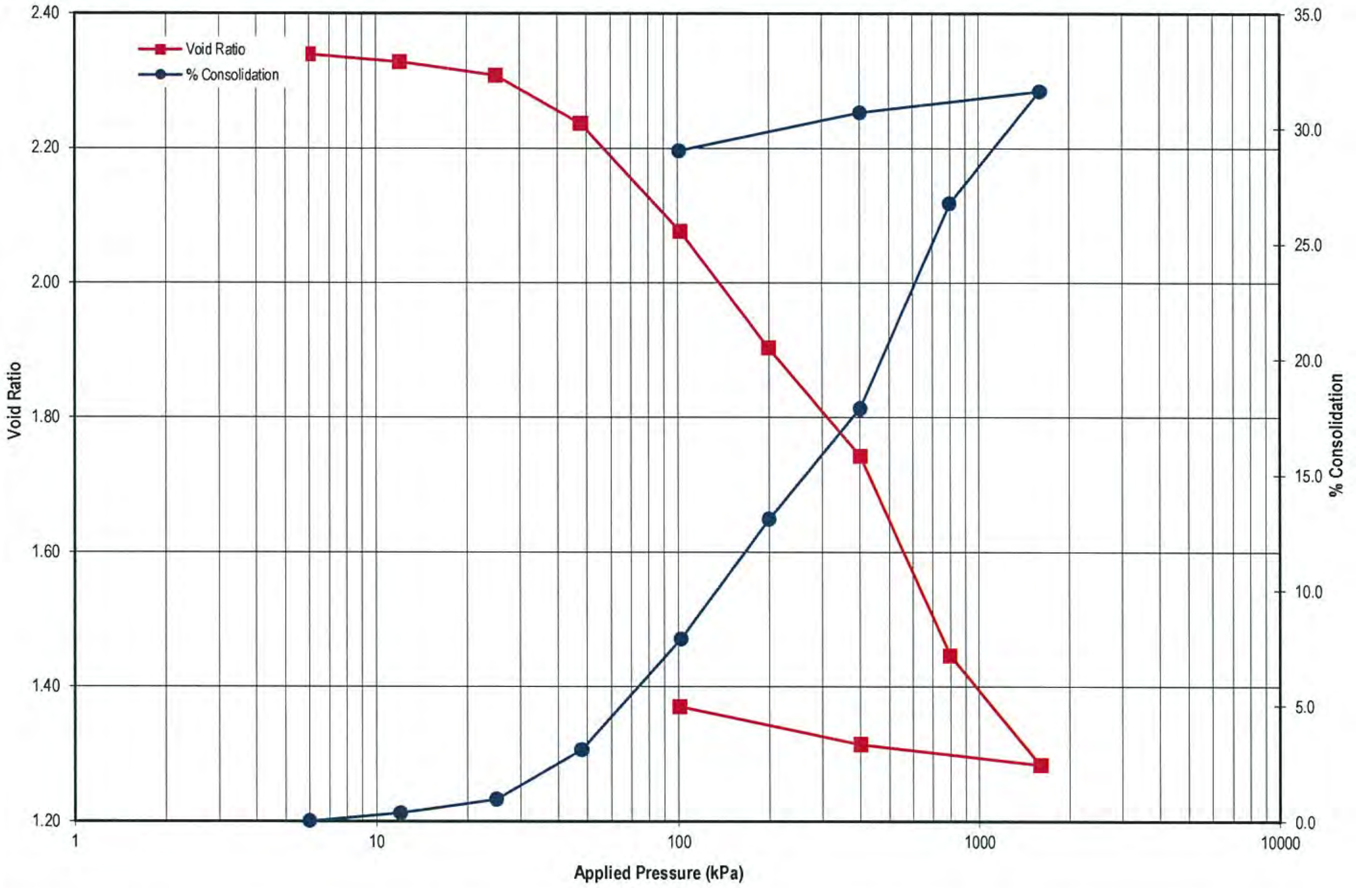
Ph: +61 8 9258 8323

# OEDOMETER TEST REPORT

**Test Method: AS1289.6.6.1, 3.5.1**

| | | | |
|---|---|---|---|
| **Client:** | Douglas Partners Pty Ltd | **Report No.:** | 13100302-OED |
| **Project:** | Southern Special Purpose Precinct | **Test Date:** | 17/10/2013 |
| | | **Report Date:** | 7/11/2013 |
| **Client Id.:** | Bore 4 | **Depth (m):** | 4.50-4.90 |

**Description:** CLAY Grey




| | | | | | |
|---|---|---|---|---|---|
| Wet Density ($t/m^3$): | 1.52 | Initial Moisture (%): | 83.2 | Test Condition: Inundated on load | |
| Particle Density ($t/m^3$): | 2.77 | Initial Voids Ratio: | 2.340 | Initial Degree of Saturation (%): | 99.5 |
| Undisturbed sample supplied by the client | | Remarks: Tested as Received | | Page 1 of 2 | |

REP03102

Accredited for compliance with ISO/IES 17025.
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/National Standards.

Tested at Trilab Brisbane Laboratory.

**Authorised Signatory**

C. Channon

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

**Laboratory Number 9926**

Trilab
Soil Rock Calibration

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Perth
2 Kimmer Place,
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Ph: +61 8 9258 8323

# OEDOMETER TEST REPORT

Test Method: AS1289.6.6.1, 3.5.1

| | | | |
|---|---|---|---|
| **Client:** | Douglas Partners Pty Ltd | **Report No.:** | 13100302-OED |
| **Project:** | Southern Special Purpose Precinct | **Test Date:** | 17/10/2013 |
| | | **Report Date:** | 7/11/2013 |
| **Client Id.:** | Bore 4 | **Depth (m):** | 4.50-4.90 |
| **Description:** | CLAY Grey | | |

## TEST RESULTS

| Stage | Load (kPa) | Cc | Cv ($m^2$/yr) $t_{50}$ | Cv ($m^2$/yr) $t_{90}$ | Mv ($kPa^{-1}x10^{-3}$) | $C_a$ x $10^{-3}$ | % Consolidation |
|---|---|---|---|---|---|---|---|
| 1 | 6-12 | 0.039 | 2.76 | 19.19 | 0.592 | 0.00 | 0.4 |
| 2 | 12-25 | 0.061 | 7.63 | 7.50 | 0.452 | 1.73 | 0.9 |
| 3 | 25-48 | 0.253 | 4.03 | 9.64 | 0.943 | 4.81 | 3.1 |
| 4 | 48-102 | 0.489 | 0.76 | 1.68 | 0.917 | 3.46 | 7.9 |
| 5 | 102-200 | 0.593 | 0.92 | 1.05 | 0.575 | 8.35 | 13.1 |
| 6 | 200-402 | 0.528 | 1.48 | 2.70 | 0.273 | 7.92 | 17.9 |
| 7 | 402-800 | 0.992 | 1.31 | 1.74 | 0.272 | 17.61 | 26.8 |
| 8 | 800-1601 | 0.538 | 1.79 | 1.78 | 0.083 | 0.15 | 31.6 |
| 9 | 1601-402 | 0.051 | 3.83 | 14.16 | 0.011 | 1.25 | 30.7 |
| 10 | 402-101 | 0.092 | 0.51 | 6.47 | 0.079 | 3.42 | 29.0 |

Remarks: Tested as Received



REP03102

Accredited for compliance with ISO/IES 17025.
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/National Standards.

Tested at Trilab Brisbane Laboratory.

Authorised Signatory

C. Channon

NATA
TECHNICAL COMPETENCE

Laboratory Number
9926