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***REPORT***
***on***
***GEOTECHNICAL INVESTIGATION***

***PROPOSED COMMERICAL DEVELOPMENT***
***12 STURT STREET, TOWNSVILLE***

***Prepared for***
***CAFALO PTY LTD***

***PROJECT 46330***
***December, 2006***

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## TABLE OF CONTENTS

Page

1. INTRODUCTION ........................................................ 1
2. SITE DESCRIPTION ........................................................ 2
3. REGIONAL GEOLOGY ........................................................ 2
4. FIELD WORK METHODS ........................................................ 2
5. FIELD WORK RESULTS ........................................................ 3
6. LABORATORY TESTING ........................................................ 4
7. COMMENTS ........................................................ 5
7.1 Proposed Development ........................................................ 5
7.2 Pertinent Geotechnical / Design Issues ........................................................ 5
7.3 Excavation Conditions ........................................................ 6
7.4 Re-Use of Excavated Material ........................................................ 6
7.5 Groundwater ........................................................ 7
7.6 Excavation Support ........................................................ 8
7.6.1 Design Parameters ........................................................ 8
7.6.2 Wall Options ........................................................ 9
7.6.3 Anchoring ........................................................ 11
7.7 Foundations ........................................................ 11
7.7.1 Overview ........................................................ 11
7.7.2 Pad Footings ........................................................ 12
7.7.3 Raft Footing ........................................................ 12
7.7.4 Piled Footings ........................................................ 13

Notes Relating to This Report
DRAWING No 1 – Location of Test Pits/Bores
APPENDIX A Results of Field Work

TDH:DSK:BDS
Project 46330
December 2006

**REPORT ON GEOTECHNICAL INVESTIGATION**
**PROPOSED COMMERICAL DEVELOPMENT**
**12 STURT STREET, TOWNSVILLE CITY**

## 1. INTRODUCTION

This report details the results of a geotechnical investigation carried out for a proposed commercial development at 12 Sturt Street, Townsville. The work was carried out at the request of Mr Bill Spee of Cafalo Pty Ltd, site owners and developers, and carried out in consultation with Mr Bruce Barrett of Barrett Architects and Associates Pty Ltd.

It is understood that the development includes a six storey commercial building with basement level car parking, the depth and extent of which was unknown at the time of investigation. Site investigation was carried out to provide the following information:

- subsurface and groundwater conditions;
- excavation conditions and suitable excavation support systems; and
- geotechnical parameters for the design of foundations and retaining wall structures.

The investigation included the drilling of test bores with in-situ testing, followed by laboratory testing of selected samples and engineering analysis. Details of the field and laboratory work are given in the report, together with comments relating to design and construction practice.

A layout plan showing existing boundaries of the site was provided by Barrett Architects and Associates Pty Ltd, to assist with the investigation.

## 2. SITE DESCRIPTION

The site is a rectangular shaped area of approximately 500 $m^2$ on the eastern side of Sturt Street, approximately 20 m south of its intersection with Denham Street (refer attached Drawing 1). The site is bounded by one and two storey commercial buildings to the north and east and by a multi-storey Telstra phone exchange building to the south. All these buildings appeared to be directly adjacent to the site boundaries.

At the time of the investigation the site was essentially vacant and being utilised as a car park. The surface level of the site was approximately 2.0 m below street level (i.e. at an approximate elevation of RL +6.5 m to +7.0 m) with an access ramp off Sturt Street. Street level (Sturt Street) was estimated to be at an approximate level of RL +8.5 m. The site sloped slightly down from the eastern end to the western end at angles of less than approximately 5º. It had been generally cleared of vegetation, with either gravel filling or bituminous chip seal surfacing.

## 3. REGIONAL GEOLOGY

Reference to the Queensland Department of Mines' 1:100 000 Geological Series Sheet 8259 for Townsville indicates the site is underlain by Pleistocene aged alluvium described as typically comprising "clay, silt, sand and gravel" and lies close to the boundary of early Permian aged Castle Hill Granite described as typically comprising "biotite leucogranite, microgranite; minor granophyre and granodiorite".

Alluvial sand was encountered in the easternmost of the two site bores, at shallow depth, and granite and residual soil were encountered at both bore locations, consistent with the above geological description.

## 4. FIELD WORK METHODS

The field work was undertaken on 11 and 12 September 2006 and included two test bores (Bores 1 and 2) using an EVH3300 truck-mounted auger/rotary soil sampling and drilling rig. The bores were initially advanced using 100 mm diameter spiral flight augers then rotary methods with mud stabilisation. Upon reaching the underlying bedrock, NMLC diamond core drilling techniques were used to extend the bores to depths of 14.7 m and 15.0 m, although drilling reverted to rotary methods with mud stabilisation where excessive core loss was experienced. Standard penetration tests (SPTs) were carried out at approximately 1.5 m intervals prior to commencement of core drilling.

The subsurface conditions encountered in the bores were logged by a geo-environmental scientist, who also collected samples for laboratory testing and identification purposes. Core samples from the bores were collected and stored in boxes, for detailed logging and photography. Point load strength testing was also undertaken on core samples from selected intervals.

Hand-slotted PVC standpipes were installed in the bores at the completion of drilling. The annulus between the standpipe and the bore was backfilled with granular cuttings and sealed at the surface with bentonite and concrete.

The approximate locations of the test bores are shown on Drawing 1 attached.

## 5. FIELD WORK RESULTS

The subsurface conditions encountered in the bores are shown in the attached borehole logs in Appendix A. These should be read in conjunction with the general notes preceding them, which explain the descriptive terms and classification methods.

A brief summary of the subsurface conditions encountered in the bores is presented below.

| | |
|---|---|
| FILLING | Filling generally comprising grey sandy gravel and brown sand was encountered to depths of 0.1 m and 1.0 m (RL +6.4 m and +6.1 m) in Bores 1 and 2 respectively. |
| SAND | Loose light brown sand was encountered in Bore 2 below the filling to 2.2 m depth (RL +4.9 m). It is considered that this stratum is probably alluvium. |
| CLAYEY SAND | Medium dense light brown clayey sand was encountered between 2.2 m and 5.4 m in Bore 2. It is considered that this stratum is decomposed granite (residual soil). |

| | |
|---|---|
| SANDY CLAY | Hard sandy clay was encountered below the filling in Bore 1 and below the clayey sand in Bore 2, and continued to depths of 5.9 m and 9.1 m respectively. It is considered that this stratum is also decomposed granite (residual soil). |
| GRANITE | Granite of variable strength was encountered in both Bores 1 and 2 below the sandy clay, below respective elevations of RL +0.6 m and -2.0 m, and continued to termination depths of the bores. The strength of the granite generally increased with depth from extremely low to very high strength and was generally fractured. Both strength and fracture spacing, however, was variable. |

No free groundwater was observed while augering the bores and the introduction of drilling fluids precluded any groundwater measurements during drilling. Water was measured in both standpipes a few days after installation, and the levels recorded are summarised in Table 1 below.

**Table 1 – Groundwater Observations**

| Bore | Surface Level (m AHD) | Date | Groundwater | |
|---|---|---|---|---|
| | | | Depth (m BGL[1]) | Reduced Level (m AHD) |
| 1 | 6.5 | 15 Sept 2006 | 1.7 | 4.8 |
| 2 | 7.1 | 15 Sept 2006 | 3.0 | 4.1 |

Notes: 1. BGL – Below Ground Level

It should be noted that groundwater levels are affected by climatic conditions and soil permeability and will therefore vary with time.

## 6. LABORATORY TESTING

Laboratory testing comprised point load testing on samples of the rock core. The results of the point load testing are shown on the borehole logs. The results of the point load testing indicated that the samples of granite tested were generally of very high strength.

# 7. COMMENTS

## 7.1 Proposed Development

It is understood that the proposed development includes the construction of a six storey building with basement carparking. Basement depths had not been finalised at the time of reporting, however it is understood that the excavation for basement levels will be as deep as practicable and that the footprint will extend to be immediately adjacent to surrounding structures, in order to maximise the car parking area.

No detailed design loadings were available at the time of reporting. However, preliminary compressive column loadings in the order of 5,000 kN (working) have been estimated for the purposes of this report.

## 7.2 Pertinent Geotechnical / Design Issues

The investigation indicated the site to be generally underlain by filling overlying alluvial soils, then residual soils (derived by the weathering of granite) to depths of 5.9 m (closest to the street in Bore 1) and 9.2 m (at the rear of the site in Bore 2). The top of underlying granite bedrock corresponded to respective depths of approximately 7.9 m and 10.5 m below street level.

The underlying granite is variable in strength, being up to high or very high strength with depth but with extremely low to very low strength bands. The granite was proved to a depth of 11.0 m (RL -4.0 m) in Bore 1 and to 15.0 m (RL -7.9 m) in Bore 2.

Previous experience suggests that very heavy ripping and/or rock breaking plant or blasting will be necessary for excavation into the underlying granite. Therefore, it is suggested that excavation for the basement be limited to the base of soils/top of granite interface to minimise excavation induced vibration associated with rock breaking plant and blasting. Excavation and construction of a basement within the alluvial sand and residual soil will require perimeter retaining wall support.

Groundwater was encountered in the standpipes installed in both Bores 1 and 2, several days after installation, at depths of 1.7 m and 3.0 m (elevations of RL +4.8 m and +4.1 m) respectively. Therefore, shallow groundwater at this site will be pertinent to retaining support, basement design and construction methods.

It should be noted that the design of basement wall retention systems and options for control of groundwater will be dependant upon the nature of the existing footings supporting the adjacent buildings (i.e. whether these buildings are supported on upper level footings or piles).

Further comments relating to design and construction practices associated with the above are given in the following sections.

### 7.3 Excavation Conditions

Excavation for basement levels will initially encounter filling then predominantly clayey sand and sandy clay. Removal of these soils should be readily carried out using conventional earthmoving equipment, such as a hydraulic excavator or backhoe, although productivity may decrease considerably as the density of the sandy clay (weathered granite) increases due to a lesser degree of weathering at depth.

The bores indicate bedrock depths are variable at the site, with granite encountered in Bores 1 and 2 at respective depths of 5.9 m and 9.1 m (i.e. at elevations of RL +0.6 m and RL -2.0 m). Point load strength index test results on core samples of the more intact and relatively higher strength granite were in the range of 2.4 MPa to 10.8 MPa, which suggests that the unconfined compressive strength (UCS) for the granite may be as high as 60 MPa to more than 200 MPa. As noted above bulk excavation of fractured rock of this nature may require blasting or at least very heavy ripping and the use of rock breaking tools. Given the depth of this material and the presence of the surrounding adjacent properties it will probably be more practicable and considerably more economical to limit basement excavation depths to the top of the granite.

Anticipated groundwater inflow and support for structures adjoining the basement excavation will require the construction of in-situ retaining wall support prior to excavation.

### 7.4 Re-Use of Excavated Material

Most of the spoil derived from excavation at the site should be suitable for re-use as engineered filling from a geotechnical perspective, given that it will comprise a mixture of sand, clayey sand and sandy clay.

### 7.5 Groundwater

Groundwater inflows into the excavation are expected given that groundwater was encountered during monitoring of the standpipes installed in the bores. Monitoring indicates that groundwater levels are RL 4.1 m to RL 4.8 mAHD, and hence within the residual soils (decomposed granite) profile some 4 m to 6 m above the top of granite bedrock.

The presence of groundwater will necessitate some form of groundwater control at the site. The options to control groundwater during construction and during service would appear to be as follows:

- restrict the basement floor level above the groundwater level;
- construct a ‘cut-off’ wall around the basement to restrict water flow; or
- lower the groundwater during construction by pumping from deep wells, ‘spear points’, or sumps.

It is suggested that ongoing monitoring of standpipe levels be undertaken to establish fluctuation of groundwater levels with seasonal or climatic influences.

The restriction of basement floor level will greatly reduce parking space, in effect reducing the potential basement depth by half. This may not be a viable option.

A continuous watertight ‘cut-off’ wall will probably be required for reasons of retention of the loose alluvial sand and filling and options are discussed in Section 7.6 below.

It should be recognised that long-term pumping to lower groundwater levels may result in settlement being induced under adjacent buildings founded on upper level footings.

The adoption of a fully ‘tanked’ or watertight basement construction is preferred, rather than the adoption of a drained basement floor slab for the relief of groundwater accumulating below the slab. Watertight permanent basement construction would preclude the need for long term pumping and maintenance of pumps and fittings, which may be susceptible to clogging if the groundwater has elevated total dissolved solids in this area.

Final design for the development will have to take into account hydrostatic uplift loadings on the basement floor and retaining walls. Given the relatively low lying nature of the site and low surface relief in the surrounding area, allowance should be made for groundwater rising to close to existing street level (approximately RL +8 m), which may occur after prolonged or heavy rain periods. These groundwater fluctuations should only be relatively short term.

## 7.6 Excavation Support

### 7.6.1 Design Parameters

As basement excavation is proposed close to the boundaries of adjoining properties, in-situ retaining support constructed prior to excavation will be necessary, to support the adjoining properties both during construction and as part of the final structure. Reference is made to the results of Bore 2, which encountered sand to RL +4.9 m. In addition, much of the filling appeared to be granular. Hence, if full retaining support is not provided, collapse of these soils during excavation may undermine footings of neighbouring structures and subsequently cause damage to these structures.

It is suggested that design of permanent retaining structures be based on an average bulk unit weight for the retained material of 20 kN/m$^3$ and on a triangular earth pressure distribution. In order to maximise rigidity of these walls, 'at rest' ($K_o$) earth pressure conditions should be applied. Where multi-level anchoring of the walls is undertaken (or internal propping) then pressure distribution will be rectangular beneath the first row of anchors (rather than triangular).

Earth pressure coefficients for retaining wall design are presented in Table 2 below.

Surcharge loads from adjacent development (upper level building footings) should be included in the wall design by multiplying vertical loads by the appropriate coefficient given in Table 2.

**Table 2 – Earth Pressure Coefficients (non-sloping crest surface)**

| Material | Unit Weight ($kN/m^3$) | Earth Pressure Coefficient | |
|---|---|---|---|
| | | $K_0$ (Rigid wall) | Passive Resistance at base of wall |
| Filling (typically above RL +6 m) | 20 | 0.60 | - |
| Alluvial loose sand | 18 | 0.50 | - |
| Residual medium dense clayey sand | 20 | 0.40 | $K_P$ = 4.0 |
| Residual hard sandy clay | 20 | 0.40 | 300 kPa |
| Granite of very low strength or better | 22 | 0.30 | 400 kPa |

It should be noted that passive pressures and coefficients are ultimate values and should be factored by no less than 2 (working stress approach) in order to restrict lateral movement.

The permanent basement walls should be designed for full hydrostatic pressure, where the basement is designed as fully 'tanked' without pressure relief. Uplift forces due to buoyancy should also be considered for a fully 'tanked' basement design.

### 7.6.2 Wall Options

A number of alternative methods can be considered for excavation support, as detailed below:

**(i) Secant Pile or Diaphragm Wall**

It is possible (dependant upon actual basement depths) that a secant pile wall or a diaphragm wall may be stiff enough not to require props or anchors. This, however, requires verifying by calculations once adjacent building footings and load layout have been researched. The construction of an unanchored wall, however, would also be dependent upon good penetration into granite bedrock. A reasonably watertight wall may be achieved by either method.

**(ii) Contiguous Pile Wall**

A contiguous pile wall, where piles are installed slightly apart or just touching is unlikely to be suitable at this site due to the presence of uncontrolled granular fill and loose sand. Furthermore, such a wall would not be watertight.

### (iii) Soldier Pile Wall

A soldier pile wall, constructed with piles at 2 to 3 diameter spacings and pre-cast concrete rails or shotcrete panels in-between is not likely to be suitable for the same reasons as for the contiguous pile wall. Furthermore, this soldier pile method relies upon the excavated face remaining near vertical during installation.

### (iv) Sheet Pile Wall

It is considered unlikely that sheet piles may be stiff enough to adequately support adjacent surcharge loadings and requisite depths of cutting without penetration into granite and exterior anchoring. Driving of sheet piles into hard soils and weak rock often results in opening of the 'clutches' between piles, reducing watertightness. Furthermore the relative deepness of the granite means that anchors would require to intrude over large portions of the neighbouring properties as they are most effectively installed at about $15^{o}$ to $20^{o}$ below horizontal. Furthermore, sheet piles may not be suitable as a permanent wall, unlike say secant and diaphragm walls, and would require the construction of an additional adjoining permanent wall on the inside. Apart from the risk of movement to adjacent structures, an additional drawback with sheet piling is induced ground vibration which could lead to damage of rigid adjacent building materials.

It is anticipated that a secant pile wall would provide the most suitable construction system at the site. The wall would have to be keyed into the underlying granite as far as practicable in order to prevent the inflow of groundwater below the toe of the wall. Such penetration, however, may not be sufficient to totally prevent anchoring and will be severely restricted by the presence of the high to very high strength granite.

It would be prudent to undertake dilapidation surveys on adjoining buildings prior to construction to document their condition prior to commencement of work in order to respond to any spurious claims arising from construction activities. It may also be beneficial to carry out monitoring to confirm vibration levels on adjoining properties during relevant site works.

### 7.6.3 Anchoring

The use of inclined post-tensioned tie-back anchors is suggested as one method of anchoring support with minimal deflection. Soil anchors, rather than rock anchors, may require to be considered due to the relative depth of the bedrock. Such anchors require specialist contractors and equipment and will be relatively expensive to install. If anchors are not feasible due to cost or the pressure of piles beneath adjacent structures, then internal propping may be required in conjunction with 'top-down' construction. Anchors need only be of temporary construction, since it is assumed that permanent propping support will be provided by the building and basement floors. They should be designed to have a free (non-bonded) length equal to at least their height above the base of the excavation, and after installation they should be check stressed to 125% of the nominal working load.

For preliminary design of anchors, bond lengths may be based on maximum allowable bond stresses as set out below, dependant upon which materials the anchors derive their resistance.

- 60 kPa in hard residual sandy clays;
- 200 kPa in very low strength granite rock; and
- 1500 kPa in high strength granite rock.

It may be more appropriate for estimation of maximum allowable bond stress to be made by the contractor installing anchors, at the time of construction. Anchors should have a minimum bond length of 3 m.

## 7.7 Foundations

### 7.7.1 Overview

It has been assumed that the plan area of the building above will be the same or slightly larger than the plan area of the basement beneath the building. Allowing for groundwater rising close to street level (RL +8.5 m), the groundwater head associated with basement excavation to the underlying high to very high strength granite (approx RL +1 m) equates with a uniform buoyant uplift pressure of approximately 75 kPa for a fully tanked basement. The compressive loading from the six storey structure is estimated to be approximately 60 kPa, which suggests that additional uplift resistance may be required such as by vertical rock anchors through the lower basement floor.

Provided that basement excavation is to be taken to the top of granite bedrock (nominally close to RL +1 m) or within no greater than 1 m to 2 m above granite bedrock, then building column loads may most effectively be carried via pad footings constructed through the floor of the excavation. Further comments are presented below.

Piles may also be considered where pads require excessive excavation. Comments are provided below on design of piles and also on an alternative raft slab option.

Where pad or pile footings are to found on granite rock, additional confirmatory drill holes will be required beneath at least 50% of these footings, accompanied by spoon testing to confirm that such foundation rock does not contain overly high thicknesses of clay seams and fracturing.

### 7.7.2 Pad Footings

Due to the anticipated relatively high column loadings (up to an estimated 5000 kN) it is suggested that pad footings, where preferred/selected, be founded on granite rock to minimise footing sizes and settlements. For initial design comparison and footing sizings, allowable bearing pressures for typical soils encountered in the test bores are as follows:

- 300 kPa in hard residual sandy clays or extremely low strength granite;
- 1000 kPa in fractured very low strength or better granite (subject to additional investigation);
- 2500 kPa in fractured high strength or better granite (subject to additional investigation).

It follows from the above that a pad footing loaded to 5000 kN, bearing on granite of 1000 kPa allowable being pressure, will require to be approximately 2.2 m square.

### 7.7.3 Raft Footing

Site preparation for a reinforced concrete basement raft foundation would require over excavation followed by proof rolling of the subgrade, then placement of a well compacted gravel working platform.

Design of the raft would require detailed assessment of the shears, displacements and moments resulting from what is likely to be a small number of concentrated loads from columns.

Consideration will need to be given to the likely settlements due to the effects of the loading from the building, less the excavated materials on the underlying soils.

Provided that there is no net increase in stress (i.e. the building loads are less than the weight of soil removed, which is about 20 kPa per m depth), then settlements will be essentially negligible and will take place relatively quickly (eg. during construction). However, care must be taken in design of the raft to ensure that there are no local areas where foundation stresses are excessive. Reference should be made to Douglas Partners when full details of loadings are available, to assist with the calculation of settlements and probable moments.

#### 7.7.4 Piled Footings

Driven enlarged base “Franki” piles (or similar pile types) or precast concrete piles are the most common pile types in North Queensland. However, the relatively high column loading and the need to found in granite bedrock will result in driven piles inducing potentially damaging ground vibration and noise, thus precluding their use at this site.

Alternative preferred piling options are bored piles or concrete screw pile systems such as “Atlas” or similar. Bored piles may require temporary steel casing to prevent hole collapse in clayey sands beneath the groundwater. Suggested design values for bored piles are presented below. Ultimate end bearing values are as follows:

- 2500 kPa in fractured very low strength or better granite;
- 7500 kPa in fractured high strength granite.

Ultimate skin friction values are as follows:

- 100 kPa in hard residual sandy clays and extremely low strength granite;
- 250 kPa in fractured low strength or better granite;
- 750 kPa in fractured high strength granite.

It is recommended that a factor of safety of 2.5 be applied to the above unfactored ultimate end bearing and skin friction values for worked stress analysis, or a geotechnical strength reduction factor ($Ø_g$) of 0.4 for limit state design. Successful dynamic load testing of a sufficient number of piles will allow the use of a higher $Ø_g$ value.

The above end bearing and skin friction values are dependant upon all loosened or disturbed spoil material being removed from the base of the pile and any smear from the side walls, prior to casting of concrete.

**DOUGLAS PARTNERS PTY LTD** Reviewed by:

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# NOTES RELATING TO THIS REPORT

## Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring 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.

## Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

| Soil Classification | Particle Size |
|---|---|
| Clay | less than 0.002 mm |
| Silt | 0.002 to 0.06 mm |
| Sand | 0.06 to 2.00 mm |
| Gravel | 2.00 to 60.00 mm |

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

| Classification | Undrained Shear Strength kPa |
|---|---|
| Very soft | less than 12 |
| Soft | 12—25 |
| Firm | 25—50 |
| Stiff | 50—100 |
| Very stiff | 100—200 |
| Hard | Greater than 200 |

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

| Relative Density | SPT "N" Value (blows/300 mm) | CPT Cone Value ($q_c$ — MPa) |
|---|---|---|
| Very loose | less than 5 | less than 2 |
| Loose | 5—10 | 2—5 |
| Medium dense | 10—30 | 5—15 |
| Dense | 30—50 | 15—25 |
| Very dense | greater than 50 | greater than 25 |

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

## Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

## Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

**Test Pits —** these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

**Large Diameter Auger (eg. Pengo) —** the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

**Continuous Sample Drilling —** the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

**Continuous Spiral Flight Augers —** the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

**Non-core Rotary Drilling —** the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

**Rotary Mud Drilling —** similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

**Continuous Core Drilling —** a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

## Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

## Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6)\ N \text{ (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

## Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

## Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

## Bore Logs

The bore logs presented herein 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. 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 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, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

## Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it 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.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations 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.

## Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company 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 condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

## Site Anomalies

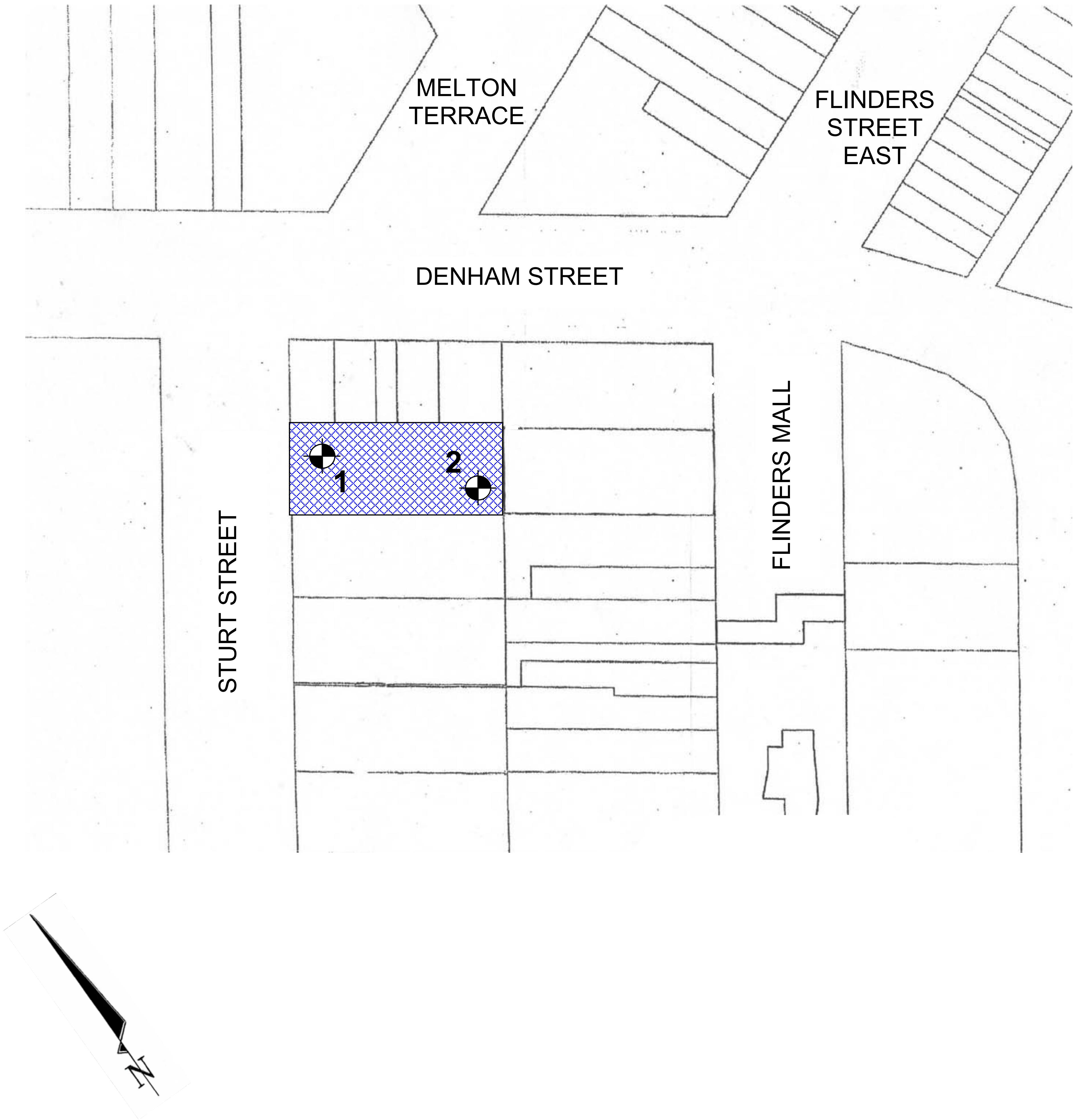
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

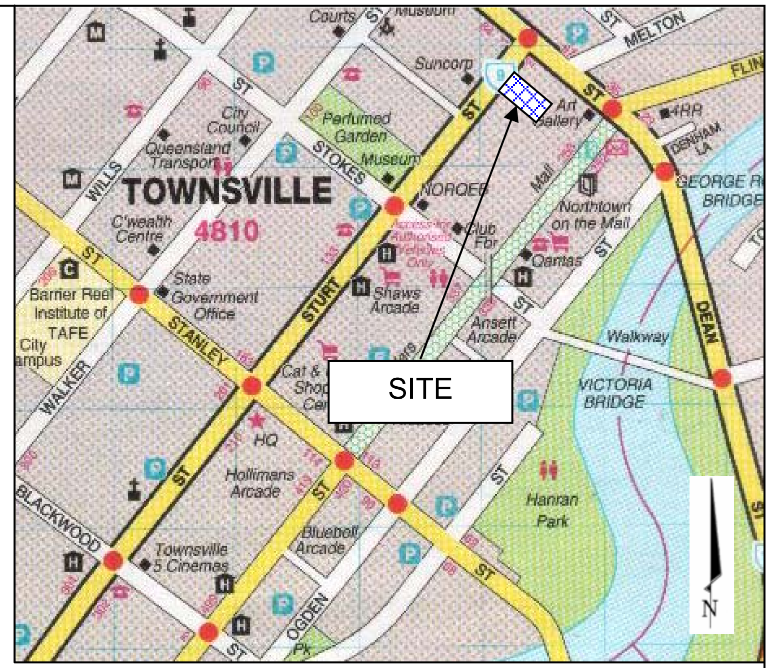
## Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation 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. The Company 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 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.




**LOCALITY**

Legend

Test Bore Location and Number

Notes:

1. Test locations are approximate only and are shown with reference to existing site features.
2. Drawing adapted from layout plan provided by client.

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*Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Singleton, Campbelltown, Townsville, Cairns, Darwin*

TITLE:

TEST LOCATION PLAN
PROPOSED COMMERCIAL DEVELOPMENT
12 STURT STREET, TOWNSVILLE CITY

| | | | |
|---|---|---|---|
| CLIENT: Cafalo Pty Ltd | | | |
| DRAWN BY: DSK | SCALE: NTS | PROJECT No: 46330 | OFFICE: Townsville |
| APPROVED BY: | | DATE: November 2006 | DRAWING No: 1 |

# APPENDIX A

# RESULTS OF FIELD WORK

# BOREHOLE LOG

**CLIENT:** Cafalo Pty Ltd
**PROJECT:** Proposed Commercial Development
**LOCATION:** 12 Sturt Street, Townsville

**SURFACE LEVEL:** +6.5m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/–

**BORE No:** 1
**PROJECT No:** 46330
**DATE:** 11 Sep 06
**SHEET** 1 OF 2

| RL | Depth (m) | Description of Strata | Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break) | Type | Core Rec. % | RQD % | Test Results & Comments |
|---|---|---|---|---|---|---|---|
| | 0.1 | FILLING - Grey fine to coarse gravel filling with some sand; humid | | | | | |
| | | SANDY CLAY - Hard orange brown sandy clay with a trace of fine to medium gravel; sand fraction fine to coarse; M<Wp (probable decomposed granite) | | D | | | |
| | 1 | | | | | | |
| | | | | S | | | 12,15,17 N = 32 |
| | 2 | | | | | | |
| | 3 | | | D | | | |
| | | - becoming grey without gravel from 3.2m depth | | S | | | 5,9,23 N = 32 |
| | 4 | | | S | | | 25,31,30/80mm refusal |
| | 5 | | | S | | | 30/80mm refusal |
| | 5.9 / 6 | GRANITE - High then extremely low to very low strength moderately to highly weathered then extremely to highly weathered pink granite | 5.9m: Unless otherwise noted defects are rough undulated joints at 30mm to 300mm spacing | C | 80 | 13 | |
| | 6.55 | | 6.55m: CORE LOSS: 100mm | | | | |
| | 7 | | | C | 100 | 0 | |
| | | - becoming medium to high strength moderately to slightly weathered from 7.5m to 8.1m depth | 7.7m: J, tight, 10° | C | 100 | 38 | |

**RIG:** EVH 3300 **DRILLER:** Probin **LOGGED:** TDH **CASING:** NW to 6.0m depth

**TYPE OF BORING:** Auger to 1.5m depth; rotary mud flush to 5.9m detph; NMLC coring to 14.7m depth

**WATER OBSERVATIONS:** Addition of drilling fluids precluded groundwater observations

**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | |
|---|---|---|---|
| A | Auger sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | PID | Photo ionisation detector |
| B | Bulk sample | S | Standard penetration test |
| U, | Tube sample (x mm dia.) | PL | Point load strength Is(50) MPa |
| W | Water sample | V | Shear Vane (kPa) |
| C | Core drilling | ▷ | Water seep ₹ Water level |

CHECKED
Initials: OSK
Date: 19/12/06

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# BOREHOLE LOG

**CLIENT:** Cafalo Pty Ltd
**PROJECT:** Proposed Commercial Development
**LOCATION:** 12 Sturt Street, Townsville

**SURFACE LEVEL:** +6.5m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 1
**PROJECT No:** 46330
**DATE:** 11 Sep 06
**SHEET** 2 **OF** 2

Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

| Depth (m) | Description of Strata | Discontinuities | Type | Core Rec. % | RQD % | Test Results & Comments |
|---|---|---|---|---|---|---|
| | | | C | 66 | 0 | |
| 8.2 | | 8.2m: CORE LOSS: 100mm | | | | |
| | - becoming extremely low strength extremely to highly weathered below 8.3m depth | | | | | |
| | - becoming high to very high strength highly to slightly weathered with some extremely low strength bands below 8.8m depth | | C | 60 | 0 | |
| 9 9.03 | | 9.03m: CORE LOSS: 420mm | | | | |
| | | | C | 80 | 0 | PL(A) = 8.22MPa<br>PL(D) = 10.82MPa |
| 9.81 | | 9.81m: CORE LOSS: 90mm | C | 100 | 0 | |
| 10 10.05 | | 9.93m: J, stepped, clay lined, 60° | | | | |
| | | 10.05m: CORE LOSS: 850mm | C | 11 | 0 | |
| 11 | - medium to high strength slightly weathered to fresh then high to very high strength moderately to slightly weathered | 11.2m: J, ti, 50° | C | 100 | 72 | PL(A) = 8.68MPa<br>PL(D) = 10.82MPa |
| 12 | | 11.95m: J, r, u, stepped, 45° | C | 100 | 0 | |
| | | 12.7m: J, r, stepped, 50° | C | 100 | 62 | PL(A) = 3.13MPa<br>PL(D) = 4.36MPa<br>PL(A) = 4.47MPa<br>PL(D) = 4.36MPa |
| 13 | | 13.17m: J, ti, 30° | | | | |
| | | 13.4m: J, ti, clay lined, u, 50° | | | | |
| 14 | | | C | 100 | 66 | |
| 14.6 | | 14.6m: CORE LOSS: 100mm | C | 0 | 0 | |
| 14.7 | Bore discontinued at 14.7m depth | | | | | |
| 15 | | | | | | |

**RIG:** EVH 3300 **DRILLER:** Probin **LOGGED:** TDH **CASING:** NW to 6.0m depth
**TYPE OF BORING:** Auger to 1.5m depth; rotary mud flush to 5.9m detph; NMLC coring to 14.7m depth
**WATER OBSERVATIONS:** Addition of drilling fluids precluded groundwater observations
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | |
|---|---|---|---|
| A | Auger sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | PID | Photo ionisation detector |
| B | Bulk sample | S | Standard penetration test |
| U, | Tube sample (x mm dia.) | PL | Point load strength Is(50) MPa |
| W | Water sample | V | Shear Vane (kPa) |
| C | Core drilling | ▷ | Water seep ▼ Water level |

CHECKED
Initials: DSK
Date: 19/12/06

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# BOREHOLE LOG

**CLIENT:** Cafalo Pty Ltd
**PROJECT:** Proposed Commercial Development
**LOCATION:** 12 Sturt Street, Townsville

**SURFACE LEVEL:** +7.1m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/–

**BORE No:** 2
**PROJECT No:** 46330
**DATE:** 12 Sep 06
**SHEET** 1 OF 2

| RL | Depth (m) | Description of Strata | Graphic Log | Sampling & In Situ Testing: Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| | 0.1 | FILLING - Grey fine to coarse gravel filling with some fine to coarse grained sand; humid | | | | | | | |
| | | FILLING - Generally comprising brown fine to coarse grained sand with some gravel; damp | | D | 0.2 | | | | |
| | 1.0 | SAND - Loose light brown fine to coarse grained sand with a trace of clay; damp (probable alluvial) | | | | | | | |
| | | - trace of manganese nodules below 1.5m depth | | D | 1.5 | | | | |
| | | | | S | 1.5–1.95 | | 2,3,4 N = 7 | | |
| | 2.2 | CLAYEY SAND - Medium dense light brown clayey fine to coarse grained sand; damp to moist (probable decomposed granite) | | D | 2.5 | | | | |
| | | - becoming grey and orange brown below approximately 3.0m depth | | S | 3.0–3.4 | | 7,8,10 N = 18 | | |
| | | | | S | 4.5–4.95 | | 7,7,11 N = 18 | | |
| | 5.4 | SANDY CLAY - Hard grey sandy clay; sand fraction fine to coarse grained; M<Wp (probable decomposed granite) | | | | | | | |
| | | | | S | 6.0–6.26 | | 20,30/110mm refusal | | |
| | | | | S | 7.5–7.57 | | 30/75mm refusal | | |

**RIG:** EVH 3300 **DRILLER:** Probin **LOGGED:** TDH **CASING:** NW to 6.0m depth
**TYPE OF BORING:** Auger to 1.5m depth; rotary mud flush to 9.1m depth; NMLC coring to 15.0m depth
**WATER OBSERVATIONS:** Addition of drilling fluids precluded groundwater observations
**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | |
|---|---|---|---|
| A | Auger sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | PID | Photo ionisation detector |
| B | Bulk sample | S | Standard penetration test |
| U, | Tube sample (x mm dia.) | PL | Point load strength Is(50) MPa |
| W | Water sample | V | Shear Vane (kPa) |
| C | Core drilling | ▷ | Water seep ▼ Water level |

CHECKED
Initials: DSK
Date: 19/12/06

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# BOREHOLE LOG

**CLIENT:** Cafalo Pty Ltd
**PROJECT:** Proposed Commercial Development
**LOCATION:** 12 Sturt Street, Townsville

**SURFACE LEVEL:** +7.1m AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 2
**PROJECT No:** 46330
**DATE:** 12 Sep 06
**SHEET** 2 **OF** 2

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water | Well Construction Details |
|---|---|---|---|---|---|---|---|---|---|
| | | SANDY CLAY - Hard grey sandy clay; sand fraction fine to coarse grained; M<Wp (probable decomposed granite) *(continued)* | | | | | | | |
| | | - trace of fine gravel below 8.5m depth | | S | 8.5<br>8.61 | | 30/110mm refusal | | |
| | 9 | | | | | | | | 9 |
| | 9.1 | GRANITE - High to very high strength slightly weathered to fresh pink granite with some extremely low to low strength extremely to highly weathered bands | | C | 9.1 | | | | |
| | 10 | | | C | 10.0 | | | | 10 |
| | 10.38 | | | | | | | | |
| | | | | C | 10.5 | | | | |
| | 11 | | | | | | | | 11 |
| | | | | C | 11.15 | | | | |
| | | | | | 11.35 | | PL(A) = 4.55MPa<br>PL(D) = 6.51MPa | | |
| | 12 12.0 | | | | | | | | 12 |
| | | | | C | 12.25 | | | | |
| | 12.83 | | | | | | | | |
| | 13 | | | C | 13.25 | | | | 13 |
| | 13.85 | | | | 13.85 | | | | |
| | 14 | - becoming medium to high to very high strength moderately to fresh weathered granite below 14m depth | | C | 14.45 | | PL(A) = 2.4MPa<br>PL(D) = 4.36MPa | | 14 |
| | 15 15.0 | Bore discontinued at 15.0m depth | | | 15.0 | | | | 15 |

**RIG:** EVH 3300 **DRILLER:** Probin **LOGGED:** TDH **CASING:** NW to 6.0m depth

**TYPE OF BORING:** Auger to 1.5m depth; rotary mud flush to 9.1m depth; NMLC coring to 15.0m depth

**WATER OBSERVATIONS:** Addition of drilling fluids precluded groundwater observations

**REMARKS:**

**SAMPLING & IN SITU TESTING LEGEND**

| | | | |
|---|---|---|---|
| A | Auger sample | pp | Pocket penetrometer (kPa) |
| D | Disturbed sample | PID | Photo ionisation detector |
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CHECKED
Initials: DSK
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