Douglas Partners

Geotechnics / Environment / Groundwater

Integrated Practical Solutions

# Report on Geotechnical Investigation

Proposed Collaroy Vet Project
1121 Pittwater Road, Collaroy

Prepared for
BigCity Design Pty Ltd

DRAFT

Project 99918.00
December 2020

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# Report on Geotechnical Investigation
# Proposed Collaroy Vet Project
# 1121 Pittwater Road, Collaroy

## 1. Introduction

This report presents the results of a geotechnical investigation undertaken for the proposed Collaroy vet project at 1121 Pittwater Road, Collaroy. The investigation was commissioned by Dimitra Lomis of BigCity Design Pty Ltd and was undertaken in accordance with Douglas Partners' proposal SYD201340 dated 19/11/2020.

It is understood that the proposed development includes demolition of the rear single storey part of the existing building and construction of a two-storey extension. A lift is also proposed as a part of the extension. It is understood the underside of the lift core will be about 1m below the existing soil level. An upgrade to the rear carpark is also understood to be proposed. The carpark is to be re-surfaced, at a minimum, and may need to be re-graded as well.

Investigation was carried out to provide information on surface and subsurface conditions, including groundwater, and results of in-situ testing and laboratory results. This report provides the following:

- Existing pavement thickness/profile Information;
- Existing Subgrade strength to aid assessment of pavement design;
- Information on existing footings encountered at client selected location (TP1); and
- Design parameters and footing options for temp/permanent support for the lift core.

## 2. Site Description

The site is located on the north-western corner of Pittwater Road and Collaroy Street, Collaroy. The site is bounded by a multi-storey unit development to the north, Collaroy Street to the south, Pittwater Road to the east and laneway and multi-storey unit development to the west.

The site is irregular shaped with an area of 670 $m^2$ which extends about 13 m along Collaroy Street and about 53 m along Pittwater Road.

Topographically, the site sits at the foot of the Collaroy Plateau. The site is generally flat and gradually rises to the west beyond the property boundary.

The existing building is a brick building which is located in the eastern half of the site. The house is of double brick construction suspended on brick concrete footings. The existing building was observed to be in relatively good condition for its age.

The rear of the site is accessible via a concrete driveway and comprises a bitumen sealed carpark and concrete pathway which surrounds the existing building. The existing condition of the carpark was in generally poor condition with signs of fatigue and pavement failure. Signs of pavement fatigue/failure was presented by

## 3. Regional Geology

Reference to Sydney 1:100,000 Geology Sheet the site is underlain by Quaternary Sediments – Foredune over beach ridge system (Qhf/Qhbr) which typically comprises medium to fine marine sand over quartz sand with minor shell content.

A previous investigation in Jenkins Street, to the north, was carried out by DP within the same stratigraphy. CPT data showed loose to loose/medium dense soils to 10m depth. Groundwater was measured to be about 4.5m to 5m below the existing grade.

Reference to the NSW Acid Sulphate Soils Risk maps indicates the area consists mainly of aeolian sands where this a low probability of ASS generation greater than 3m below the existing ground surface.

## 4. Field Work Methods

The field work for the current investigation was carried out on 3 December 2020 and 7 December 2020 in the presence of a Senior Geotechnical Engineer and Geotechnical Engineer from DP.

Prior to excavation of CPTs and test pits, service locating was undertaken near the proposed test locations. A shovel was used prior digging test pits as a precaution to avoid clashing with any in-ground services. CBR1, CBR2 and TP1 was cut using a diamond tip saw cutter prior to excavation.

Fieldwork carried out on 3 December included one piezocone penetration tests (CPT01) to cone tip refusal depth of 7.96 m. In a cone penetration test (CPT) a ballasted truck-mounted test rig is used to push a 35 mm diameter instrumented cone tipped probe into the soil with a hydraulic ram system. Continuous measurements are made of the end-bearing pressure on the cone tip and the friction on a 135 mm long sleeve located immediately behind the cone. The cone tip resistance and friction readings are displayed during the test and stored for subsequent plotting of results and interpretation.

Groundwater measurement were made after completion of the CPT and withdrawal of the rods.

Fieldwork carried out on 7 December included three test pits (TP1, CBR1 and CBR2) excavated to 1.3m and 1.2m depth using hand tools. Tests Pits CBR1 and CBR2 were carried out within the existing carpark in order to assess the existing pavement depth and underlying subgrade. Test Pit TP1 was carried out on the footpath adjacent to the southern side of the existing building to ascertain to the depth and type of the existing footings at the edge of the house.

The test locations are shown on Drawing 1 in Appendix B.

## 5. Field Work Results

CPT plots and test pit logs are presented in Appendix C, together with notes explaining descriptive terms and classification methods used. The sequence of subsurface materials encountered across the site, in increasing depth order, may be summarised as follows:

### 5.1 Lift Well Cone Penetration Test (CPT)

| | |
|---|---|
| **Fill** | Interpreted from the CPTs to extent to depth of 0.75 m. The fill was generally interpreted as sand and clayey sand material, however the borehole previously drilled within the site indicated the fill typically includes gravel. The fill was generally consistent with very loose to medium dense sand. |
| **Sand** | Loose to medium dense sand to depths of 3.72 m over loose to medium dense Silty SAND / Sandy SILT to depths of 5.17 m; over dense to very dense sand to depths of 7 m to 10 m; then very dense sand to the CPT termination depths of between 12.6 m to 15 m. CPTs 401 to 404 encountered practical refusal in the very dense sand. |
| **Clay** | Very Stiff to hard to practical refusal at 7.96 m depth in over-consolidated clay or weathered rock |

### 5.2 Carpark (CBR Test Pits)

| | |
|---|---|
| **Pavement** | • Wearing Course: Asphaltic Concrete varying in depths of 70mm and 20mm thickness; over,<br>• Fill / Gravelly SAND roadbase, fine to medium angular gravel, fine to medium sand, apparently well compacted, to depths of 0.14 m and 0.15 m; underlain by,<br>• Fill / Sandy GRAVEL, fine to medium subangular gravel, fine to medium sand to depths of 0.35 m and 0.26 m |
| **Fill** | • Sandy CLAY / Clayey Sand, low plasticity, fine to medium grained sand. Consistency of sandy clay was firm to stiff. Relative density of sand were considered medium dense. Termination at 1.3m and 1.2m depth. |

### 5.3 Existing Footing (TP1)

The cross section of the existing footing is shown in the Appendix C.

### 5.4 Groundwater

The measured depths to groundwater on completion of the CPTs indicate a groundwater level at the time of the investigation of approximately 5m below the existing ground level. The groundwater level can fluctuate with climatic conditions and are likely to increase following periods of extended wet weather and tidal flows.

## 6. Laboratory Testing

Laboratory samples collected during the field investigation were subjected to the following tests:

- Two 4-day soak California bearing ratio (CBR) tests (AS 1289.5.1.1 & 2.1.1); and,
- Two Moisture Content Tests;

The tests were undertaken within a NATA accredited laboratory in accordance to Australian Test Methods referenced above. Detailed results of these tests are presented in Appendix D. Result summaries of the laboratory results are presented below in Table 3.

**Table 3: CBR, Atterberg Limits, Linear Shrinkage and Moisture Content Test Results**

| BH ID | Depth (m) | Material | CBR (%) | SMDD (t/m$^3$) | OMC (%) | FMC (%) | Swell (%) |
|---|---|---|---|---|---|---|---|
| CBR 1 | 0.4-1.0 | F/ Sandy CLAY | **3.5** | 1.73 | 19.0 | 22.3 | 1.0 |
| CBR2 | 0.35-0.6 | F/ Clayey SAND | **30.0** | 1.72 | 16.0 | 19.7 | 0.0 |

Notes:
SMDD = Standard Maximum Dry Density
OMC = Optimum Moisture Content
FMC = Field Moisture Content
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
GS = Group Symbol as per AS-1726-2017
F/ = FILL/

## 7. Geotechnical Model

A geotechnical model of the site has been prepared based on the results of the current investigation. This section shows interpreted geotechnical divisions of underlying soil. The descriptions shown in Table 4 are generalised due to the variability in strength and should be used as a guide only. Reference should be made to the CPT results for more detailed information and descriptions of the soil profile.

**Table 4: Interpreted Geotechnical Model**

| Depth Range (m) | Layer Description |
|---|---|
| 0.8 | FILL: sand, generally poorly compacted |
| 0.8 – 5.2 | SAND: very loose to medium dense |
| 5.2 – 7.96 | CLAY / Silty CLAY: very stiff to hard |

The groundwater level was about 5 m below the existing ground level. Groundwater levels will fluctuate with weather and may temporarily rise by at least 1 m following periods of prolonged rainfall. Published literature by Merrick indicates that fluctuations of up to 2 m can occur, based on historical data dating from the 1940s. On-going monitoring of groundwater levels, particularly after heavy rainfall, should continue in order to obtain more information on fluctuations in groundwater levels.

# 8. Proposed Development

The proposed development includes construction of the Collaroy vet project. The proposed structure includes a two-storey building addition to the rear of the site with single lift. It is understood the lift core will require excavation to approximately 1 m below existing surface level.

# 9. Comments

## 9.1 Excavation Support

### 9.1.1 General

It is understood that no basement levels are proposed for this development, hence only minor excavations for site levelling, and detailed excavation for foundations, lift pits and services will be required.

Excavations through sandy fill and sand should be readily achieved using conventional earthmoving equipment such as tracked excavators. Some allowance for removal of potential obstructions such as buried pavements and concrete slabs and footings in the fill should be made.

Groundwater is expected at a depth of about 4-5 m or more and is not expected to be encountered during shallow excavation on site.

All excavated materials to be removed from the site will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA, 2014).

### 9.1.2 Batters

Steep or vertical excavations in uncontrolled filling and natural sand are not expected to be stable for any period of time. Therefore, both temporary and permanent batters may be required for excavations and earthworks.

Where there is sufficient space, maximum temporary and permanent batters of 1.5H:1V and 2H:1V, respectively, are suggested for excavations less than 3 m high in filling and/or natural sand, above the water table, and where not subjected to surcharge loads. It is anticipated that groundwater will note be encountered during excavation of the lift core, foundations, or services.

Care should be taken when excavating near existing structures so as not to undermine existing footings or buildings. If the above conditions are encountered shoring or underpinning may be required prior to excavation.

### 9.1.3 Earth Pressures

Any retaining walls, for example the walls of the lift pit, will be subjected to earth pressures from the ground surface down to the base of the excavation. Table 5 below outlines material and strength parameters that could be used for the design of excavation support structures.

**Table 5: Material and Strength Parameters for Wall Design Purposes**

| Material | Unit Weight (kN/m³) | Buoyant Unit Weight (kN/m³) | Coefficient of Active Earth Pressure ($K_a$) | Coefficient of Earth Pressure at Rest ($K_o$) | Passive Earth Pressure* |
|---|---|---|---|---|---|
| Filling | 20 | 10 | 0.40 | 0.60 | N/A |
| Loose to Medium Dense Sand | 20 | 10 | 0.35 | 0.53 | $K_p$ = 2.5 |

Notes: *Ultimate values and only below bulk excavation level

A triangular lateral earth pressure distribution could be assumed for a cantilevered wall or a wall with a single row of anchors/props. A trapezoidal lateral earth pressure distribution, where the maximum pressure acts over the central 60% of the wall, could be assumed for walls propped at the top and base.

Surcharge pressures from adjacent structures, construction machinery and traffic should also be incorporated into the design of the walls as necessary.

## 9.2 Subgrade Preparation

The design subgrade level for pavements and slabs is relatively shallow, hence predominantly uncontrolled filling and natural sand is likely to be exposed. The existing filling is assumed to be uncontrolled in the absence of compaction records and should be removed and replaced as engineered filling to a depth that is appropriate for the pavement or structure to be supported.

From a geotechnical perspective, the predominantly sand/gravel filling is considered to be suitable for re-use as engineered filling, provided that it is free of oversize particles (>100 mm) and deleterious material. The suitability of re-using site-won filling and natural soil should also be considered from a contamination perspective (refer to DP's contamination report).

Subgrade preparation measures are recommended up to subgrade level as follows:

- Remove filling to at least 0.6 m below the design subgrade level, or to the top of natural sand, whichever is shallower.
- Compact the exposed material, then proof roll the exposed surface using a minimum 10-tonne roller in non-vibration mode. The proof roll should be witnessed by an experienced geotechnical engineer to detect any 'soft' spots;
- Any loose/soft areas identified during proof rolling should be removed/rectified as directed by the geotechnical engineer;
- Replacement filling should be free of oversize particles (>100 mm) and deleterious material, and should be placed in loose layer thicknesses not greater than 200 mm (dependent upon the size of compaction machinery) and compacted to a dry density ratio of at least 98% relative to Standard compaction, with moisture contents maintained within 2% of Standard optimum moisture content,

increasing to a dry density ratio of 100% standard for the upper layer of the subgrade. If the replacement filling used is sand, a density index of 75% should be targeted;

- Some moisture conditioning (i.e. drying or wetting) may be required for compaction of filling; and
- Density testing in accordance with AS 3798 - 2007 *(Guidelines on earthworks for commercial and residential developments)* should be undertaken to verify that the required compaction/moisture criteria are achieved.

## 9.3 Foundations

### 9.3.1 Shallow Footings

For lightly loaded structures, shallow strip or pad footings bearing in (natural) loose or loose to medium dense sand, below the uncontrolled filling, may be feasible.

By way of example, a 0.5 m by 0.5 m pad footing or a 0.5 m wide strip footing, embedded 0.5 m deep in the natural sand, with a water table close to the founding depth, may be designed for a maximum allowable bearing pressure of 100 kPa and 80 kPa, respectively.

The amount of settlement for shallow footings founded in sand depends upon the load conditions, footing size and foundation material, but should be less than 1% of the footing width if proportioned on the basis of the above parameters.

### 9.3.1 CFA Piles

Continuous flight auger (CFA) piles founded in the natural sands or clays, or in bedrock, could be used to support the proposed structure. This type of piles is associated with relatively low levels of noise and vibration.

It is expected that noise and vibration constraints at this site will preclude the use of driven pile types. Open bored piles will not be appropriate due to the potential for soil collapse and groundwater inflow, however bored piles drilled under bentonite could be considered.

CFA piles founded in the natural sands, that are founded at least 4 pile diameters below the ground surface and 5 pile diameters above any weaker layers, or in the bedrock could be designed using the parameters provided in Table 6.

**Table 6: CFA Pile Allowable Design Parameters**

| Material Description | Allowable Shaft Adhesion (kPa) | Allowable End Bearing (kPa) |
|---|---|---|
| Filling and soft to firm (or softer) alluvium soils | - | - |
| Loose silty sand/sandy and firm silty clay | 5 | - |
| loose to medium dense silty sand and firm to stiff silty clay | 7 | - |

| | | |
|---|---|---|
| Pleistocene soils anticipated to comprise interbedded stiff (or stronger) silty clay and medium dense silty sand/sandy silt | 15 | - |
| Weathered "weak" (assumed low strength) rock (1) | 90(2) | 1500(2) |

Notes: (1) = based on "weak" rock parameters provided in SSE report

(2) = values limited as inspection of rock sockets will be limited due to piling method.

Shaft adhesion values should be reduced by 70% for the case of uplift (tension) loads and cone pull-out criteria should also be satisfied.

An appropriate geotechnical strength reduction factor should be applied when using the limit-state approach as outlined in AS 2159 – 2009 *Piling – Design and installation*. The determination of the geotechnical strength reduction factor uses a risk based approach; for preliminary design purposes a factor of 0.5 could be assumed. The serviceability limit state should also be assessed in the design of the piles.

Soil decompression can occur during CFA piling when a strong stratum is encountered. This occurs when the augers continue to rotate but the rate of auger progression decreases, displacing soil from around the auger upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and should be avoided by monitoring auger speed and progression closely.

Settlement of a pile is dependent on the loads applied to the pile and the foundation conditions in the socket zone and below the pile toe. Settlement analysis should be undertaken during the detailed design phase to provide settlement estimates to refine pile spacing and founding levels.

### 9.3.2 Steel Screw Piles

The use of steel screw piles (raked for lateral support) with a pile cap could be adopted for lightly loaded structures requiring minimal lateral resistance. Screw piles could be designed using the allowable values indicated in Table 7 below.

**Table 7 : Allowable Screw Pile Design Parameters**

| Founding Strata | Allowable End Bearing (kPa) |
|---|---|
| Medium dense (or denser) silty sand and very stiff (or stronger) silty and sandy silty clay | 200 |
| Extremely low strength (or stronger) rock | 400 |

It is important that the installation of steel screw piles be carefully controlled in the field to ensure the pile does not meet refusal prior to meeting its termination depth. In this scenario, advancement of the pile will cease, causing over rotation and disturbance of the overburden soils above the helix. This phenomenon is often encountered where steel screw piles encounter an underlying harder stratum (such as weathered rock) and the toe penetration is considerably reduced in comparison to the string rotation. Where over-rotation occurs, the bearing capacity for the helix would be substantially reduced and/or pile movements incurred.

The actual capacity of steel screw piles depends not only on the soil conditions but also on structural considerations of the piles such as the strength of the helix and the helix/shaft joint. It is considered that the structural section capacity as well as geotechnical capacity will need to be considered where the required load carrying capacity of individual steel screw piles is greater than (say) 600 kN. Measurement of installation torque should not be relied upon to indicate pile capacity, as it has been documented that significantly misleading results can be obtained. For this reason, piling contractors would be responsible for assessment of actual pile capacities for their piles.

Structural capacity of the steel screw pile should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Lateral capacity of steel screw piles could be increased by constructing concrete pile caps or by using proprietary head attachments which are dragged into the soil providing additional lateral resistance at the pile head. The lateral support is generally limited and is generally suited to non-critical structures that can accommodate some lateral movement such as light poles, signs and small towers.

## 9.4 Groundwater

Recent groundwater measurements within open CPT holes has indicated groundwater depths of about 5m below the existing ground level. In the absence of long-term monitoring of groundwater levels, it is suggested that a potential groundwater level 3m below the existing level may be considered for design and construction of below ground structures (e.g. lift pits). It is anticipated that excavation for the proposed development will be well above the water table.

## 9.5 Pavements

Following subgrade preparation as outlined in Section 9.2 and allowing for some variability in the subgrade soils, a CBR value of 3% is also considered appropriate for pavement design purposes. It may be possible to achieve a higher value if the existing fill is reworked and tested.

The above pavement thicknesses are based on the assumption that adequate surface and subsurface drainage is provided to the pavement and adjacent areas. Subsoil drains are recommended at a minimum depth of 0.6 m below subgrade levels.

Experience has shown that most of the natural or fill subgrade soils will experience at least some swelling if subjected to surface/subsurface water. If this potential for absorbing water and swelling occurs during the life of the pavement, then pavement damage could occur through the early on-set of fatigue cracking, due to increased deflections under traffic load. Subsoil drainage is therefore necessary throughout the full length of the intersection. Drainage should consist of:

- Appropriate table drains and pipe culverts to collect and discharge all surface waters within the vicinity of the pavement.
- Toe drains and crest drains at all cut batters.
- Subsoil drains at the base of all longitudinal and transverse pavement joints, positioned not less than 300 mm below subgrade level.

## 10. Limitations

Douglas Partners (DP) has prepared this report for the proposed Collaroy vet project development at Collaroy in accordance with DP's proposal SYD201340.P.001.Rev0 and acceptance received from BigCity Design Pty Ltd. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of BigCity Design Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. 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 scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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 (geotechnical / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

**Douglas Partners Pty Ltd**

# 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



## 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.

# Cone Penetration Tests

Douglas Partners

## Introduction

The Cone Penetration Test (CPT) is a sophisticated soil profiling test carried out in-situ. A special cone shaped probe is used which is connected to a digital data acquisition system. The cone and adjoining sleeve section contain a series of strain gauges and other transducers which continuously monitor and record various soil parameters as the cone penetrates the soils.

The soil parameters measured depend on the type of cone being used, however they always include the following basic measurements

- Cone tip resistance $q_c$
- Sleeve friction $f_s$
- Inclination (from vertical) i
- Depth below ground z

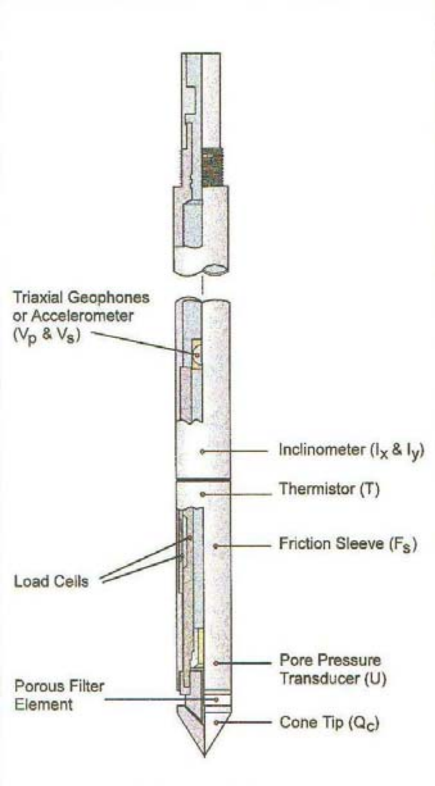



Figure 1: Cone Diagram

The inclinometer in the cone enables the verticality of the test to be confirmed and, if required, the vertical depth can be corrected.

The cone is thrust into the ground at a steady rate of about 20 mm/sec, usually using the hydraulic rams of a purpose built CPT rig, or a drilling rig. The testing is carried out in accordance with the Australian Standard AS1289 Test 6.5.1.

Figure 2: Purpose built CPT rig

The CPT can penetrate most soil types and is particularly suited to alluvial soils, being able to detect fine layering and strength variations. With sufficient thrust the cone can often penetrate a short distance into weathered rock. The cone will usually reach refusal in coarse filling, medium to coarse gravel and on very low strength or better rock. Tests have been successfully completed to more than 60 m.

## Types of CPTs

Douglas Partners (and its subsidiary GroundTest) owns and operates the following types of CPT cones:

| Type | Measures |
|---|---|
| Standard | Basic parameters ($q_c$, $f_s$, i & z) |
| Piezocone | Dynamic pore pressure (u) plus basic parameters. Dissipation tests estimate consolidation parameters |
| Conductivity | Bulk soil electrical conductivity ($\sigma$) plus basic parameters |
| Seismic | Shear wave velocity ($V_s$), compression wave velocity ($V_p$), plus basic parameters |

## Strata Interpretation

The CPT parameters can be used to infer the Soil Behaviour Type (SBT), based on normalised values of cone resistance (Qt) and friction ratio (Fr). These are used in conjunction with soil classification charts, such as the one below (after Robertson 1990)

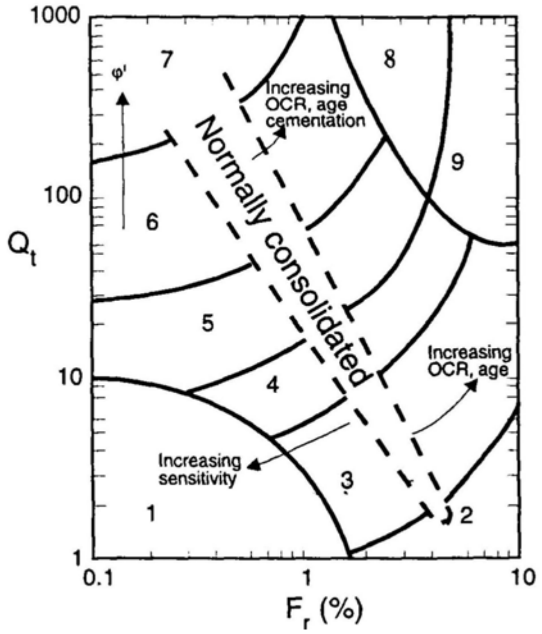



Figure 3: Soil Classification Chart

DP's in-house CPT software provides computer aided interpretation of soil strata, generating soil descriptions and strengths for each layer. The software can also produce plots of estimated soil parameters, including modulus, friction angle, relative density, shear strength and over consolidation ratio.

DP's CPT software helps our engineers quickly evaluate the critical soil layers and then focus on developing practical solutions for the client's project.

## Engineering Applications

There are many uses for CPT data. The main applications are briefly introduced below:

**Settlement**

CPT provides a continuous profile of soil type and strength, providing an excellent basis for settlement analysis. Soil compressibility can be estimated from cone derived moduli, or known consolidation parameters for the critical layers (eg. from laboratory testing). Further, if pore pressure dissipation tests are undertaken using a piezocone, in-situ consolidation coefficients can be estimated to aid analysis.

**Pile Capacity**

The cone is, in effect, a small scale pile and, therefore, ideal for direct estimation of pile capacity. DP's in-house program ConePile can analyse most pile types and produces pile capacity versus depth plots. The analysis methods are based on proven static theory and empirical studies, taking account of scale effects, pile materials and method of installation. The results are expressed in limit state format, consistent with the Piling Code AS2159.

**Dynamic or Earthquake Analysis**

CPT and, in particular, Seismic CPT are suitable for dynamic foundation studies and earthquake response analyses, by profiling the low strain shear modulus $G_0$. Techniques have also been developed relating CPT results to the risk of soil liquefaction.

**Other Applications**

Other applications of CPT include ground improvement monitoring (testing before and after works), salinity and contaminant plume mapping (conductivity cone), preloading studies and verification of strength gain.

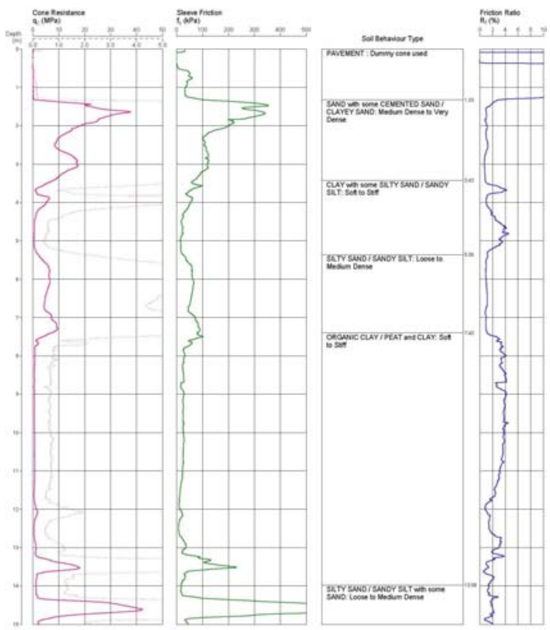


Figure 4: Sample Cone Plot

# Soil Descriptions

Douglas Partners

## Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

## Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

| Type | Particle size (mm) |
|---|---|
| Boulder | >200 |
| Cobble | 63 - 200 |
| Gravel | 2.36 - 63 |
| Sand | 0.075 - 2.36 |
| Silt | 0.002 - 0.075 |
| Clay | <0.002 |

The sand and gravel sizes can be further subdivided as follows:

| Type | Particle size (mm) |
|---|---|
| Coarse gravel | 19 - 63 |
| Medium gravel | 6.7 - 19 |
| Fine gravel | 2.36 – 6.7 |
| Coarse sand | 0.6 - 2.36 |
| Medium sand | 0.21 - 0.6 |
| Fine sand | 0.075 - 0.21 |

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

| Term | Proportion of sand or gravel | Example |
|---|---|---|
| And | Specify | Clay (60%) and Sand (40%) |
| Adjective | >30% | Sandy Clay |
| With | 15 – 30% | Clay with sand |
| Trace | 0 - 15% | Clay with trace sand |

In coarse grained soils (>65% coarse)
- with clays or silts

| Term | Proportion of fines | Example |
|---|---|---|
| And | Specify | Sand (70%) and Clay (30%) |
| Adjective | >12% | Clayey Sand |
| With | 5 - 12% | Sand with clay |
| Trace | 0 - 5% | Sand with trace clay |

In coarse grained soils (>65% coarse)
- with coarser fraction

| Term | Proportion of coarser fraction | Example |
|---|---|---|
| And | Specify | Sand (60%) and Gravel (40%) |
| Adjective | >30% | Gravelly Sand |
| With | 15 - 30% | Sand with gravel |
| Trace | 0 - 15% | Sand with trace gravel |

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

# *Soil Descriptions*

## Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

| Description | Abbreviation | Undrained shear strength (kPa) |
|---|---|---|
| Very soft | VS | <12 |
| Soft | S | 12 - 25 |
| Firm | F | 25 - 50 |
| Stiff | St | 50 - 100 |
| Very stiff | VSt | 100 - 200 |
| Hard | H | >200 |
| Friable | Fr | - |

## Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

| Relative Density | Abbreviation | Density Index (%) |
|---|---|---|
| Very loose | VL | <15 |
| Loose | L | 15-35 |
| Medium dense | MD | 35-65 |
| Dense | D | 65-85 |
| Very dense | VD | >85 |

## Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

## Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

  Soil tends to stick together.

  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.

  Soil tends to stick together, free water forms when handling.

## Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

# Symbols & Abbreviations

Douglas Partners

## Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

## Drilling or Excavation Methods

| | |
|---|---|
| C | Core drilling |
| R | Rotary drilling |
| SFA | Spiral flight augers |
| NMLC | Diamond core - 52 mm dia |
| NQ | Diamond core - 47 mm dia |
| HQ | Diamond core - 63 mm dia |
| PQ | Diamond core - 81 mm dia |

## Water

| | |
|---|---|
| ▷ | Water seep |
| ▽ | Water level |

## Sampling and Testing

| | |
|---|---|
| A | Auger sample |
| B | Bulk sample |
| D | Disturbed sample |
| E | Environmental sample |
| $U_{50}$ | Undisturbed tube sample (50mm) |
| W | Water sample |
| pp | Pocket penetrometer (kPa) |
| PID | Photo ionisation detector |
| PL | Point load strength Is(50) MPa |
| S | Standard Penetration Test |
| V | Shear vane (kPa) |

## Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

### Defect Type

| | |
|---|---|
| B | Bedding plane |
| Cs | Clay seam |
| Cv | Cleavage |
| Cz | Crushed zone |
| Ds | Decomposed seam |
| F | Fault |
| J | Joint |
| Lam | Lamination |
| Pt | Parting |
| Sz | Sheared Zone |
| V | Vein |

### Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

| | |
|---|---|
| h | horizontal |
| v | vertical |
| sh | sub-horizontal |
| sv | sub-vertical |

### Coating or Infilling Term

| | |
|---|---|
| cln | clean |
| co | coating |
| he | healed |
| inf | infilled |
| stn | stained |
| ti | tight |
| vn | veneer |

### Coating Descriptor

| | |
|---|---|
| ca | calcite |
| cbs | carbonaceous |
| cly | clay |
| fe | iron oxide |
| mn | manganese |
| slt | silty |

### Shape

| | |
|---|---|
| cu | curved |
| ir | irregular |
| pl | planar |
| st | stepped |
| un | undulating |

### Roughness

| | |
|---|---|
| po | polished |
| ro | rough |
| sl | slickensided |
| sm | smooth |
| vr | very rough |

### Other

| | |
|---|---|
| fg | fragmented |
| bnd | band |
| qtz | quartz |

# *Symbols & Abbreviations*

## Graphic Symbols for Soil and Rock

### General

- Asphalt
- Road base
- Concrete
- Filling

### Soils

- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

### Sedimentary Rocks

- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

### Metamorphic Rocks

- Slate, phyllite, schist
- Gneiss
- Quartzite

### Igneous Rocks

- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

# Appendix B

Drawings


| | | | |
|---|---|---|---|
| Douglas Partners<br>Geotechnics / Environment / Groundwater | CLIENT: BigCity Projects Pty Ltd | **Site Plan** | PROJECT No: 99918.00 |
| | OFFICE: Sydney | **Proposed Collaroy Vet Project** | FIGURE No: 1 |
| | DATE: 22 December 2020 | **1121 Pittwater Road, Collaroy** | REVISION: 0 |

# Appendix C

Results of Field Work

# CONE PENETRATION TEST

**CLIENT:** Big City Projects

**PROJECT:** Collaroy Vet Project

**LOCATION:** 1121 Pittwater Road, Collaroy

**REDUCED LEVEL:** 0

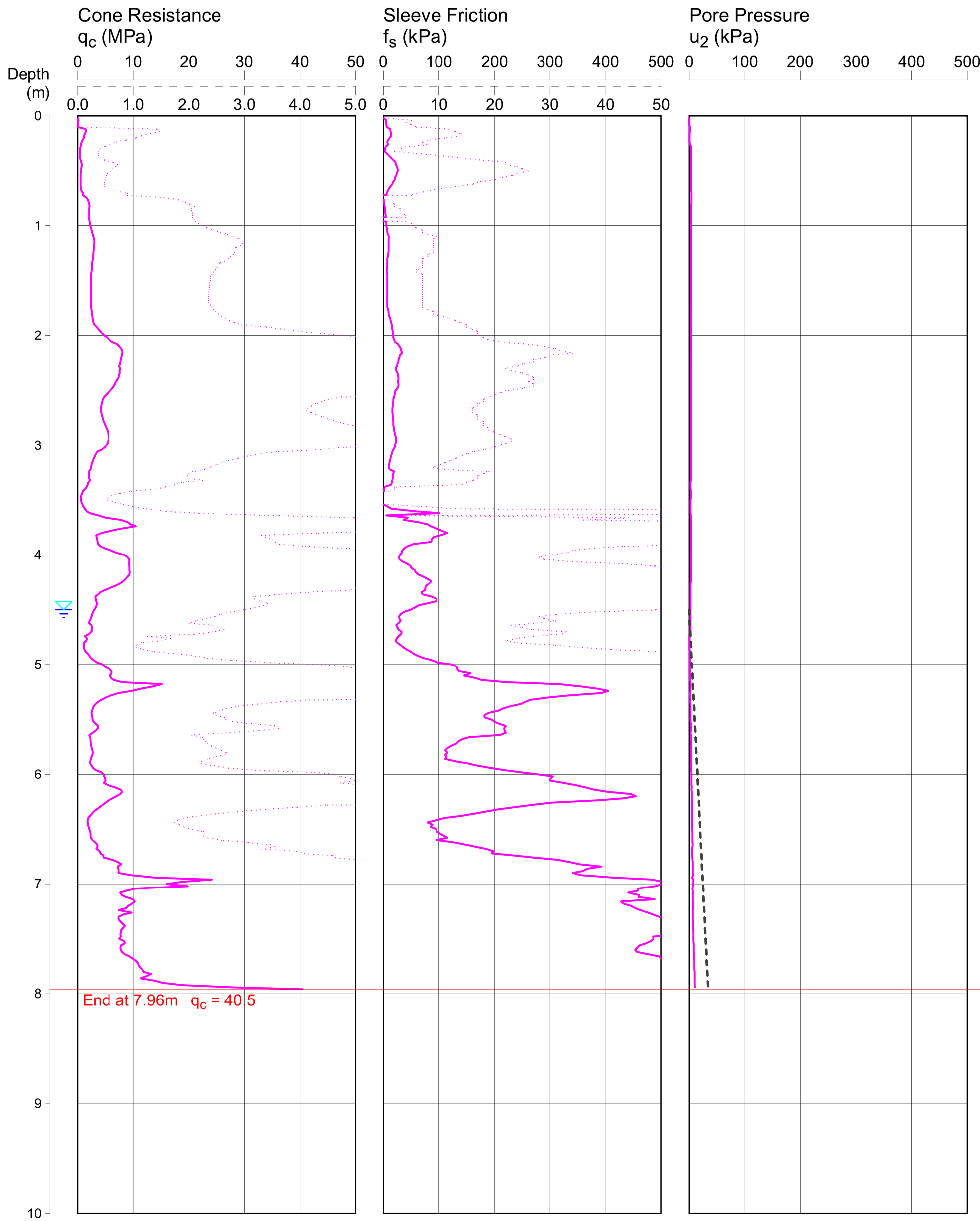
**COORDINATES:** AHD

## CPT1

**Page 1 of 1**

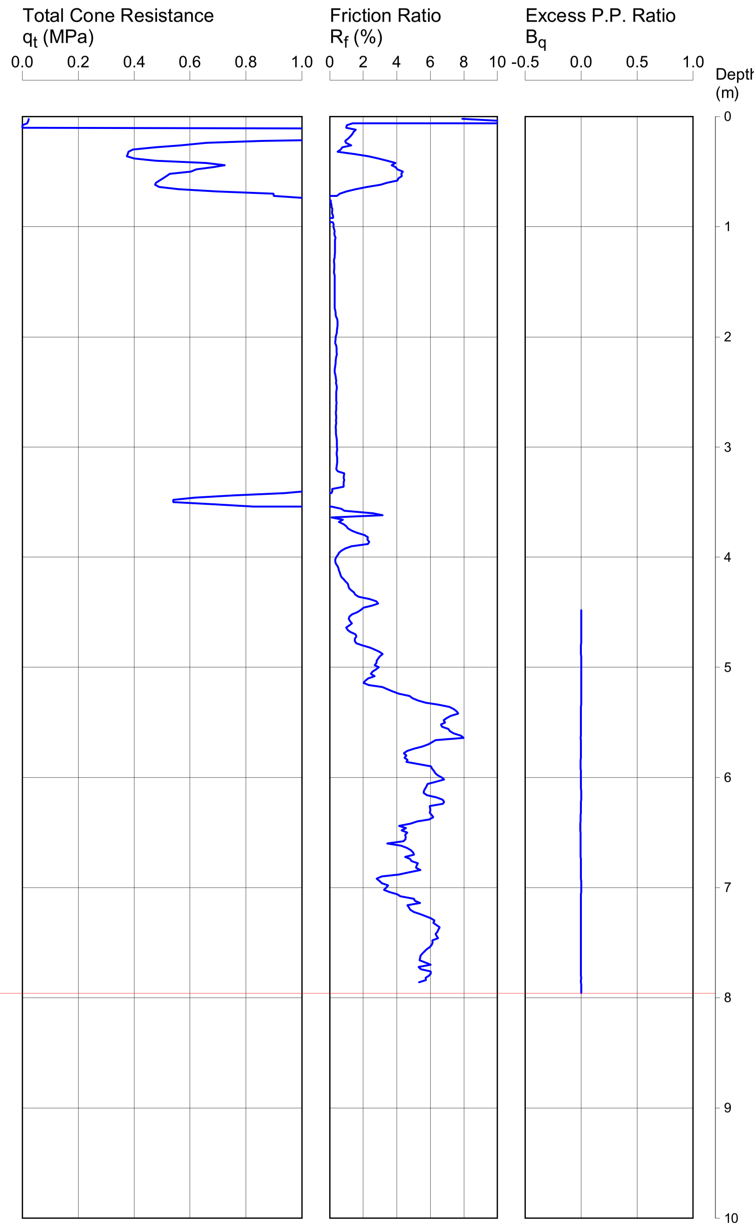
**DATE** 3/12/2020

**PROJECT No:** 99918.00




Soil Behaviour Type

| Depth (m) | Soil Behaviour Type |
|---|---|
| 0 – 0.75 | FILL / SAND and SILTY CLAY / CLAYEY SILT: Loose to Medium Dense |
| 0.75 – 2.11 | SAND: Loose to Medium Dense |
| 2.11 – 3.72 | SAND with some SILTY SAND / SANDY SILT: Loose to Medium Dense |
| 3.72 – 5.17 | SILTY SAND / SANDY SILT with some SAND: Loose to Medium Dense |
| 5.17 – 6.75 | CLAY and SILTY CLAY / CLAYEY SILT: Very Stiff to Hard |
| 6.75 – 7.96 | OVERCONSOLIDATED CLAY with some SILTY CLAY / CLAYEY SILT: Hard |




**REMARKS:**

**Water depth after test: 4.50m depth (assumed)**

**File:** P:\99918.00 - COLLAROY, Collaroy Street, Geotech\4.0 Field Work\4.1 Logs\CPT\Collaroy CPT1 DRAFT.CP5
**Cone ID:** 160917 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2


Douglas Partners
Geotechnics / Environment / Groundwater

# BOREHOLE LOG

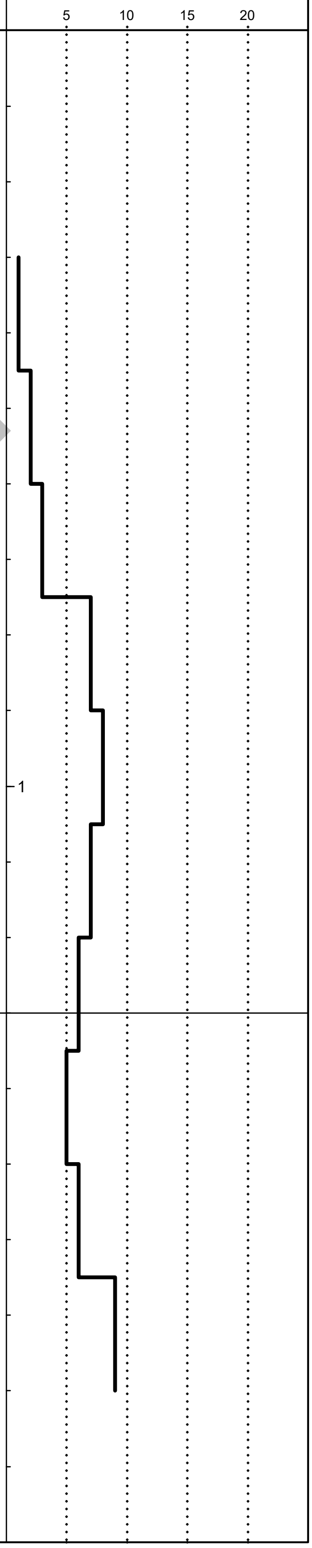
**CLIENT:** BigCity Design Pty Ltd
**PROJECT:** Proposed Collaroy Vet Project
**LOCATION:** 1121 Pittwater Road, Collaroy

**SURFACE LEVEL:** --
**EASTING:** 342563.1
**NORTHING:** 6266268.7
**DIP/AZIMUTH:** 90°/--

**BORE No:** BH01
**PROJECT No:** 99918.00
**DATE:** 7/12/2020
**SHEET** 1 OF 1

| RL | Depth (m) | Description of Strata | Graphic Log | Sampling & In Situ Testing: Type | Depth | Sample | Results & Comments | Water |
|---|---|---|---|---|---|---|---|---|
| | | ASPHALTIC CONCRETE | | | | | | |
| | 0.07 | FILL/ SAND: fine to medium sand, pale brown, with fine to medium subangular igneous gravel, dry, apparently well compacted, roadbase | | | | | | |
| | 0.14 | FILL/ Sandy GRAVEL: fine to medium subangular igneous gravel, dark grey, dry, apparently well compacted, roadbase | | | | | | |
| | 0.35 | FILL/ Sandy CLAY: low plasticity, dark brown-grey, fine to medium sand, w~PL, apparently in a firm condition<br>Below 0.9m: apparently in a very stiff condition | | B | 0.4<br>1.0 | | | |
| 1 | | | | | | | | |
| | 1.1 | FILL/ Sandy CLAY CL: low plasticity, brown, fine to medium sand, w<PL, apparently in a stiff to very stiff condition, potentially natural | | | | | | |
| | 1.3 | Bore discontinued at 1.3m<br>Target depth reached | | | | | | |

Dynamic Penetrometer Test (blows per 150mm)






**RIG:** Hand tools **DRILLER:** NR/ TM **LOGGED:** TM **CASING:** Uncased
**TYPE OF BORING:** Hand tools to 1.3m
**WATER OBSERVATIONS:** No free groundwater observed
**REMARKS:** Location coordinates are in MGA94 Zone 56.

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

**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

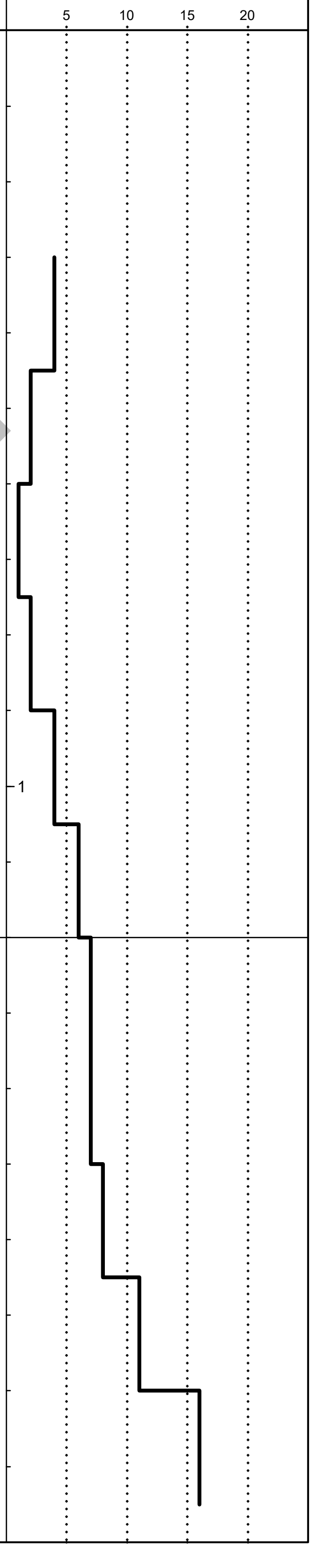
**CLIENT:** BigCity Design Pty Ltd
**PROJECT:** Proposed Collaroy Vet Project
**LOCATION:** 1121 Pittwater Road, Collaroy

**SURFACE LEVEL:** --
**EASTING:** 342550.5
**NORTHING:** 6266277.9
**DIP/AZIMUTH:** 90°/--

**BORE No:** BH02
**PROJECT No:** 99918.00
**DATE:** 7/12/2020
**SHEET** 1 OF 1

| RL | Depth (m) | Description of Strata | Graphic Log | Sampling & In Situ Testing: Type | Depth | Sample | Results & Comments | Water |
|---|---|---|---|---|---|---|---|---|
| | 0.02 | ASPHALTIC CONCRETE | | | | | | |
| | 0.15 | FILL/ SAND: fine to medium sand, pale brown, with fine to medium subangular igneous gravel, dry, apparently well compacted, roadbase | | | | | | |
| | 0.26 | FILL/ Sandy GRAVEL: fine to medium subangular igneous gravel, dark grey, dry, apparently well compacted, roadbase | | | | | | |
| | 0.65 | FILL/ Clayey SAND: fine to medium sand, yellow-brown, trace fine to medium sandstone gravel, moist, apparently in a medium dense condition | | B | 0.35–0.6 | | | |
| | 1 | FILL/ Sandy CLAY: low plasticity, dark brown-grey, fine to medium sand, w~PL, apparently in a firm condition<br>Below 0.9m: apparently in a very stiff condition | | | | | | |
| | 1.2 | Bore discontinued at 1.2m<br>Target depth reached | | | | | | |

Dynamic Penetrometer Test (blows per 150mm)




**RIG:** Hand tools **DRILLER:** NR/ TM **LOGGED:** TM **CASING:** Uncased
**TYPE OF BORING:** Hand tools to 1.3m
**WATER OBSERVATIONS:** No free groundwater observed
**REMARKS:** Location coordinates are in MGA94 Zone 56.

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

**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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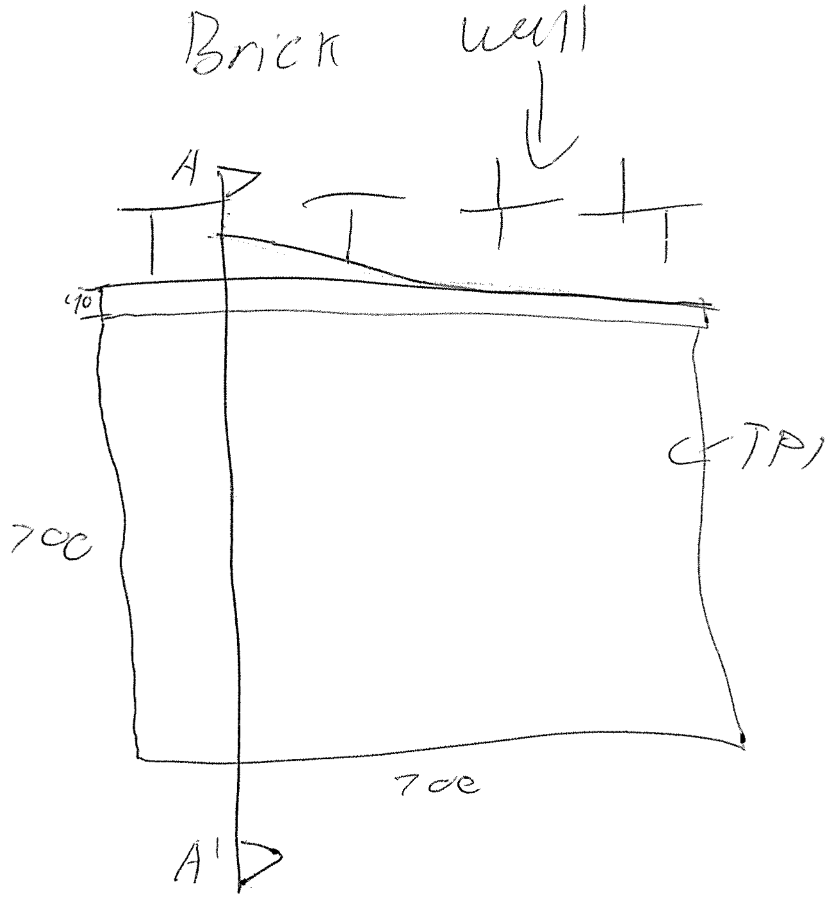
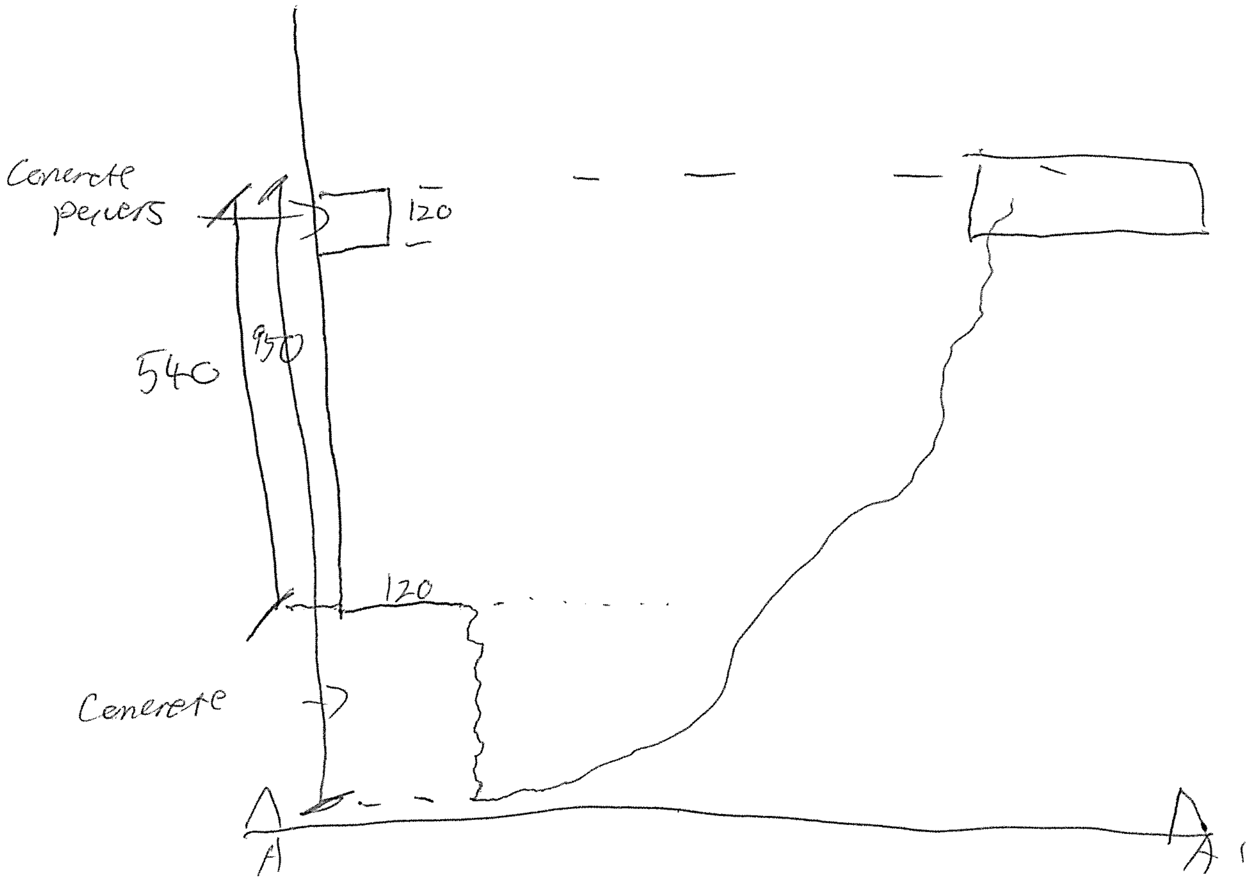
TP1


Brick wall
A
A'
90
700
700
TP1


Concrete pavers
A
120
540
950
120
Concrete
A
A'

# Appendix D

Laboratory Test Results

# Material Test Report

| | |
|---|---|
| **Report Number:** | **99918.00-1** |
| **Issue Number:** | 1 |
| **Date Issued:** | 16/12/2020 |
| **Client:** | BigCity Design Pty Ltd<br>PO Box 526, Turramurra NSW 2074 |
| **Contact:** | Dimitra Lomis |
| **Project Number:** | 99918.00 |
| **Project Name:** | Proposed Collaroy Vet Project |
| **Project Location:** | 1121 Pittwater Road, Collaroy |
| **Work Request:** | 7193 |
| **Sample Number:** | SY-7193A |
| **Date Sampled:** | 07/12/2020 |
| **Dates Tested:** | 08/12/2020 - 15/12/2020 |
| **Sampling Method:** | Sampled by Engineering Department<br>*The results apply to the sample as received* |
| **Sample Location:** | **BH1 (0.4-1m)** |
| **Material:** | Sandy CLAY: dark brown |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Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: andrew.hutchings@douglaspartners.com.au

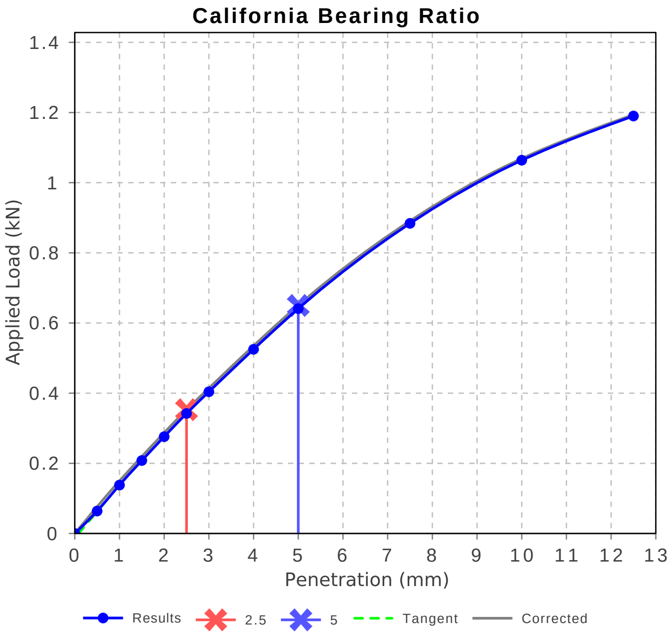

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Andrew Hutchings
Laboratory Manager

NATA Accredited Laboratory Number: 828

| California Bearing Ratio (AS 1289 6.1.1 & 2.1.1) | | Min | Max |
|---|---|---|---|
| CBR taken at | 5 mm | | |
| CBR % | **3.5** | | |
| Method of Compactive Effort | Standard | | |
| Method used to Determine MDD | AS 1289 5.1.1 & 2.1.1 | | |
| Method used to Determine Plasticity | Visual Assessment | | |
| Maximum Dry Density ($t/m^3$) | 1.73 | | |
| Optimum Moisture Content (%) | 19.0 | | |
| Laboratory Density Ratio (%) | 100.0 | | |
| Laboratory Moisture Ratio (%) | 99.0 | | |
| Dry Density after Soaking ($t/m^3$) | 1.72 | | |
| Field Moisture Content (%) | 22.3 | | |
| Moisture Content at Placement (%) | 18.6 | | |
| Moisture Content Top 30mm (%) | 19.7 | | |
| Moisture Content Rest of Sample (%) | 18.7 | | |
| Mass Surcharge (kg) | 4.5 | | |
| Soaking Period (days) | 4 | | |
| Curing Hours | 67.8 | | |
| Swell (%) | 1.0 | | |
| Oversize Material (mm) | 19 | | |
| Oversize Material Included | Excluded | | |
| Oversize Material (%) | 0 | | |



# Material Test Report

Douglas Partners
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Douglas Partners Pty Ltd
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NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Andrew Hutchings
Laboratory Manager

NATA Accredited Laboratory Number: 828

**Report Number:** **99918.00-1**
**Issue Number:** 1
**Date Issued:** 16/12/2020
**Client:** BigCity Design Pty Ltd
PO Box 526, Turramurra NSW 2074
**Contact:** Dimitra Lomis
**Project Number:** 99918.00
**Project Name:** Proposed Collaroy Vet Project
**Project Location:** 1121 Pittwater Road, Collaroy
**Work Request:** 7193
**Sample Number:** SY-7193B
**Date Sampled:** 07/12/2020
**Dates Tested:** 08/12/2020 - 15/12/2020
**Sampling Method:** Sampled by Engineering Department
*The results apply to the sample as received*
**Sample Location:** **BH2 (0.35-0.6m)**
**Material:** Clayey SAND: yellow brown

| California Bearing Ratio (AS 1289 6.1.1 & 2.1.1) | | Min | Max |
|---|---|---|---|
| CBR taken at | 5 mm | | |
| CBR % | **30** | | |
| Method of Compactive Effort | Standard | | |
| Method used to Determine MDD | AS 1289 5.1.1 & 2.1.1 | | |
| Method used to Determine Plasticity | Visual Assessment | | |
| Maximum Dry Density ($t/m^3$) | 1.72 | | |
| Optimum Moisture Content (%) | 16.0 | | |
| Laboratory Density Ratio (%) | 99.5 | | |
| Laboratory Moisture Ratio (%) | 102.0 | | |
| Dry Density after Soaking ($t/m^3$) | 1.70 | | |
| Field Moisture Content (%) | 19.7 | | |
| Moisture Content at Placement (%) | 16.4 | | |
| Moisture Content Top 30mm (%) | 18.1 | | |
| Moisture Content Rest of Sample (%) | 17.0 | | |
| Mass Surcharge (kg) | 4.5 | | |
| Soaking Period (days) | 4 | | |
| Curing Hours | 45.3 | | |
| Swell (%) | 0.0 | | |
| Oversize Material (mm) | 19 | | |
| Oversize Material Included | Excluded | | |
| Oversize Material (%) | 0 | | |