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Report on Geotechnical Investigation

Proposed Residential Development
4-18 Doncaster Avenue, Kensington

Prepared for
Blue Sky Commercial Asset Managers Pty Ltd

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1. Introduction

This report presents the results of a geotechnical investigation carried out for a proposed residential development at 4-12 Doncaster Avenue, Randwick. The work was originally commissioned by Built (NSW) Pty Ltd in 2015 and the report has been reproduced and amended, with permission, for Sky Blue Commercial Asset Managers Pty Ltd to cover the new development.

DP previously carried out a geotechnical investigation and prepared a geotechnical report for 4-12 Doncaster Avenue. Following the original investigation, 14-18 Doncaster Avenue was acquired and the supplementary geotechnical investigation included testing on 18 Doncaster Avenue. This report incorporates the previous and supplementary testing and covers the whole site (4-18 Doncaster Avenue).

The proposed development includes the construction of three storey residential buildings with a common single level basement carpark that will require excavation to depths of approximately 3 m below existing ground level. The existing semi-detached houses on the central part of the site (No. 10-12 Doncaster Avenue) will be retained and refurbished. Investigation was carried out to provide information on subsurface soil and groundwater conditions for planning, design and construction purposes.

The investigations included cone penetration tests and installation of small diameter groundwater monitoring wells to allow measurement of water levels. Details of the field work are provided in this report together with comments relating to design and construction issues.

2. Site Description

The site is a rectangular-shaped area of about 4,200 m^2 with a western frontage to Doncaster Avenue. At the time of the previous investigation the site was occupied by a single storey brick house on the northern part of the site (No. 4, to be demolished), one to two storey brick semi-detached houses on the central part of the site (No. 10-12, to be retained), one storey brick semi-detached houses (No. 14-16, to be demolished) and an asphaltic concrete paved access road (No.18).

Ground surface levels on the site fall slightly to the south from approximately RL 28.7 m to RL 27.9 m, relative to Australian Height Datum (AHD).

On the properties to the south and east of the site, there were single storey brick houses and a tram yard with acoustic walls.

The property to the north of the site was vacant and covered with grass and a bitumen paved access road and car parking area. A substation (kiosk) was located near the north-western corner of the site and plans obtained from "Dial Before You Dig" indicate that electrical services run parallel to the northern boundary.

3. Geology and Hydrogeology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Quaternary sediments comprising medium to fine grained marine sands. Bedrock comprising Hawkesbury Sandstone would be expected at significant depth. The field work confirmed the presence of sands to the investigation depth of 20 m.

The site is located over the Botany Sand Beds which contain a shallow unconfined to semi-confined groundwater system, known as the Botany Sand Aquifer, within the unconsolidated sediments. The average saturated thickness of the Botany Sands Aquifer is 15 – 20 m. Hydraulic conductivity within the sand beds is highly variable and is typically around 20 m/day in clean sand. This value decreases to 5 – 10 m/day in silty or peaty sands and to less than 4 m/day in sandy peat or clay. Groundwater flow directions are typically towards the main surface water systems (Botany Bay and Alexandra Canal, being the closest to the site) with gradients variable but in the order of 1 in 120 (Ref. 1).

The area of the Botany Sand Aquifer, extending from Botany Bay to Surry Hills and Centennial Park, contains over 30 monitoring bores operated by DWLC and numerous licensed bores. Extracted groundwater is used for industrial, domestic and irrigation purposes. Groundwater is also used for irrigation at Randwick Racecourse, Centennial Park and the University of New South Wales.

4. Previous Investigations

DP has previously undertaken investigations on the northern and central part of the subject site and also on the neighbouring sites close to the subject site as summarised below.

Project 73965.02 (December 2015)

Investigation was carried out for 4-18 Doncaster Avenue, Kensington. The field work included two CPTs to depths of approximately 20 m and installation of a small diameter groundwater monitoring wells. The approximate locations of the previous CPT tests are included on Drawing 1 in Appendix B and the CPT plots are provided in Appendix C. The sequence of subsurface materials encountered within the CPTs is described below in increasing depth order:

- Filling comprising sand and gravel to depths of 0.5 m to 1 m; over,
- Very loose to loose sand to a depth of 3.1 m then very soft to firm organic clay/peat to a depth of 5.3 m in CPT101, and very loose sand over medium dense sand to a depth of 3.7 m in CPT102; over,
- Mostly dense to very dense sand to depths to the investigation depth of 20 m. A band of very stiff to hard silty clay and medium dense silty sand about 0.5 m to 1 m thick was encountered at 15.7 m depth.

The results of the groundwater measurements from the previous and current investigations are summarised below.

Table 1: Summary of Measured Groundwater Levels in CPTs

Date	Depth (RL) to Groundwater (m)					
	CPT101 (well)	CPT102	CPT1 (well)	CPT2	CPT3 (well)	CPT4 (well)
13 May 2014	-	-	2.2 (RL26.3)	2.1 (RL26.4)	2.1 (RL26.5)	2.0 (RL26.6)
6 June 2014 (wet weather)	-	-	2.0 (RL26.5)	2.0 (RL26.5)	2.0 (RL26.6)	1.9 (RL26.7)
9 December 2015	2.8 (RL25.1)	Collapsed to 2.8 (RL25.6)	-	-	-	-

Project 73965.00 (June 2014)

Investigation was carried out for 4-12 Doncaster Avenue, Kensington. The field work included four CPTs to depths of 20 m and installation of three small diameter groundwater monitoring wells. The approximate locations of the previous CPT tests are included on Drawing 1 in Appendix B and the CPT plots are provided in Appendix C. The sequence of subsurface materials encountered within the CPTs is described below in increasing depth order:

- Possibly shallow filling near the surface over very loose to loose sand and silty sand to depths of 2 m to 3 m; over,
- Mostly medium dense to very dense sand to depths of 15 m to 16 m, with the exception of some very loose/stiff silty bands from 7.8 m to 9.6 m depth in CPT1; over,
- Mostly medium dense to very dense sands with loose/stiff silty bands to depths of 17 m to 19 m; over,
- Very dense sand to the termination depths of 19.7 m to 20 m.

The groundwater was measured at depths of between 1.9 m and 2.1 m (RL 26.3 m to RL 26.7 m).

Project 29615 (May 2001)

Investigation was carried out for a stormwater upgrade in Doncaster Avenue and included several boreholes and groundwater monitoring wells. One of the boreholes (BH1) was located to the north-west of the subject site at the approximate location shown on Drawing 1 in Appendix B. Some of the relevant findings from the investigation include:

- Groundwater was measured in the monitoring well (BH1) at a depth of 2.7 m (RL 26.1 m) in May 2001;
- Rising head permeability tests were carried in the monitoring wells and indicated hydraulic conductivity (permeability) values of 2×10^{-5} to 9×10^{-5} m/sec;
- Groundwater samples were collected from the wells and indicated pH values of between 5.5 to 6.3; and

- Soil samples were collected from the boreholes to depths of 5 m and tested to assess the potential for acid sulphate soils (ASS). The testing gave no indication of the presence of actual or potential ASS within the 5 m depth.

Project 44542 (2006 to 2012)

Investigation was carried out for a proposed development to the east, between the subject site and Randwick Racecourse. The investigation included measurement of groundwater levels within monitoring wells on several occasions between 2006 and 2012. The measured groundwater level in the monitoring well closest to the subject site varied from depths of 1.8 m to 2.2 m (RL 26.5 m to RL 26.2 m).

5. Geotechnical Model

A geotechnical cross section (Section A-A') showing the interpreted subsurface profile between the current and previous test locations is shown on Drawing 2 in Appendix B. The section shows interpreted geotechnical divisions of underlying soil together with the extent of the proposed basement. The descriptions shown on the cross sections are generalised due to the variability in both material type and strength and should be used as an approximate guide only. Reference should be made to the CPT results for more detailed information and descriptions of the soil profile.

CPT101 encountered very soft to firm organic clay/peat to a depth of about 5 m which will have implications for shoring, footings and subgrade preparation. Further investigation using CPTs will be required to assess the extent of the soft clayey soil on the site for detailed design. Further investigation of the soft soils for acid sulphate soils should also be carried out.

The monitoring indicates the groundwater table at the time of the investigations ranged from 2.0 m to 2.8 m (RL 25.0 m to RL 26.7 m) with the groundwater surface generally falling towards the south at an average gradient of approximately 1.5%. The groundwater table was generally shallower on the northern part of the site, which is consistent with our experience in the area. However, the groundwater levels may also be slightly lower than previously measured in 2014.

Experience with groundwater levels on sites underlain by sand indicate that short term fluctuations in groundwater levels of at least 1 m can occur during periods of prolonged and heavy rainfall. Published literature by Merrick (Reference 2) 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. It is understood that the flood level in this area is close to the existing surface level.

6. Comments

6.1 Proposed Development

Based on architectural drawings by Hayball Architecture (Project 2309, dated November 2018) it is understood that the proposed development includes the construction of three storey residential buildings partly on grade and partly over a common single level basement carpark. The basement floor level (RL 25.5 m) will require excavation to depths of approximately to 3 m below existing ground level. The basement footprint will be set back from the eastern and southern site boundaries by about 2 m, and will extend up to western boundary near the southern end of the site.

It is anticipated that excavation to at least 0.5 m below the floor slab level will be required for services and to allow for construction of the floor slabs and footings, although this will depend on the thickness of the floor slab and footing system adopted.

6.2 Excavation Conditions and Batter Slopes for the Basement

Excavations for the basement will be carried out through filling and natural sands which should be readily achieved using conventional earthmoving equipment such as tracked hydraulic excavators.

Based on the measured groundwater levels at the time of the investigations, it is anticipated that bulk excavation to RL 25.0 m (i.e. 0.5 m below the proposed FFL) will be about 1.5 m below the groundwater level on the northern part of the site and close to the groundwater level on the southern part of the site (based on groundwater levels measured at the time of the investigations). Temporary dewatering will be required, particularly following periods of prolonged and heavy rainfall when the groundwater table may temporarily rise as discussed in Section 6.4.

Trafficability on the sandy soils during bulk earthworks will generally require the use of tracked plant and machinery. Trafficability after bulk excavation could be improved by placement of a layer of compacted crushed concrete or similar, which may subsequently be used as sub-base.

During the bulk excavation phase, it is recommended that temporary batter slopes above the groundwater table do not exceed 1.5:1 (H:V) in both filling and sand soils. Where there is potential for groundwater to rise into the excavation, batter slopes should not exceed 2.5H:1V. Batter slopes may be possible on some parts of the western boundary and northern boundary where structures and boundaries are set back more than about 5 m from the excavation. Batter slopes are not recommended adjacent to the terrace houses to be retained or any other movement sensitive structures or services.

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA, 2014). This includes filling and natural materials that may be removed from the site. Accordingly, environmental testing will need to be carried out to classify spoil prior to transport from the site.

6.3 Dilapidation Surveys

Dilapidation reports should be undertaken on surrounding structures and pavements prior to commencing work on the site to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed. The appropriate extent of dilapidation surveys may be better assessed once details of the proposed development and construction methods have been confirmed.

6.4 Dewatering and Tanking

It is assumed that bulk excavation for the basement will be carried out to approximate RL 25.0 m (allowing for 0.5 m below the proposed floor level) although this will depend on the design of the floor slab and footings. Generally the groundwater level should be lowered to at least 1 m below the bulk excavation to allow machinery to operate and traverse the site. On this basis, the normal groundwater level (measured at the time of the investigation) may need to be temporarily lowered by approximately 2.5 m on the northern part of the site, with the depth and extent of dewatering gradually reducing towards the southern part of the site (to be confirmed with further monitoring of groundwater levels).

It is expected that the groundwater may temporarily rise above the basement level during periods of prolonged wet weather. Therefore it is recommended that the basement should be tanked and designed for hydrostatic uplift to allow for potential groundwater levels. Based on experience in the area, it is anticipated that water levels could temporarily rise by at least 1 m following periods of intense rainfall, although published hydrogeological information by Merrick (Reference 2) indicates that groundwater levels can vary by up to 2 m at the northern end of the Botany Basin. It is understood that the 1 in 100 year flood level is close to the existing surface. It is suggested that typical loads due to a groundwater table rising to the ground surface during flood events should be considered in the basement design.

In the long term, the downward force to resist uplift is typically provided by the weight of the building itself, and the detailing of the slab and foundations should be designed accordingly. It is anticipated that these pressures may be counteracted by the dead load of the building once it is at say 2 to 3 levels above ground (subject to confirmation by the structural engineer).

6.4.1 Method of Dewatering

Dewatering on sites underlain by sandy soils is usually undertaken with spears installed at regular spacing within the confines of the excavation which is usually surrounded by ‘cut-off’ walls. Spears (slotted PVC pipes) are installed below the groundwater table and generally spaced at about 1 m to 2 m centres around the perimeter of the excavation. The spears are connected by a series of pumps and hoses which collect groundwater, usually in a sedimentation tank, prior to discharge off-site. Additional spears may be required within the site to effectively lower the water table across the entire site. Sump and pump dewatering methods may be considered however they are unlikely to be practical or effective for the high permeability sandy soils.

Dewatering of the site should be carried out by a contractor with demonstrated experience in similar conditions. The use of recharge wells and shoring walls may be considered to limit drawdown of groundwater levels outside the site subject to approval from relevant authorities.

6.4.2 Drawdown and Settlement

It is anticipated that the dewatering system will require lowering of the normal groundwater table by about 2.5 m on the northern part of the site with depth of lowering reducing towards the south. The drawdown within the permeable sands should reduce rapidly away from the dewatering system. It is expected that a drawdown of less than 1 m would be within the range of historic low groundwater levels and therefore settlements due to drawdown should be relatively minor (less than 5 mm).

It is recommended that drawdown outside the excavation in the vicinity of the adjacent properties should be monitored and kept to less than 1 m below normal groundwater levels. The following general procedure is recommended to monitor groundwater drawdown levels:

- Install standpipes in accessible areas on adjacent properties to monitor groundwater drawdown levels during dewatering;
- Measure groundwater levels on a weekly basis for three weeks prior to operation of the dewatering system to establish pre-developed levels;
- Measure groundwater levels twice per day during the first two days of dewatering, and then daily during the first week of dewatering and weekly until decommissioning of the dewatering pumps, or until a lesser frequency is advised by the geotechnical engineer;
- The measured values are to be provided to the geotechnical engineer on the day of measurement for review; and
- Where drawdown levels exceed 1 m (trigger level) below pre-developed groundwater levels, the reason for the change in groundwater level should be investigated and measures put in place to rectify the exceedance. These measures could include reduction of pumping rates or suspension of dewatering.

6.4.3 Groundwater Impacts

It is considered that the temporary dewatering and construction of the floor slabs and tanked basement would not have any significant impact on groundwater flows or licensed groundwater users. To ensure the protection of any groundwater dependant ecosystems, the groundwater drawdown should be reduced and groundwater discharge should be regularly monitored.

6.4.4 Groundwater Disposal

The groundwater removed from the site will require disposal. Generally, water resulting from dewatering operations should be suitable for disposal by pumping to stormwater drains subject to confirmation testing and approval from Council.

Typically groundwater being disposed to the stormwater should comply with the following criteria, although this should be confirmed with the approving authority/council at the time:

- Suspended Solids <50 mg/L;
- Turbidity <50 NTU (this is equivalent to a clear glass of water); and
- Oil and Grease <10 mg/L (essentially the visual detection limit for oil in water).

Regular measurement of the suspended solids and turbidity at the point of discharge will be required during disposal to the stormwater system to assess the levels and any requirement for additional settling of solids prior to discharge.

It will also be appropriate to establish whether or not general contaminant levels including heavy metals, hydrocarbons and pesticides in the groundwater are within acceptable levels.

If the water at any stage does not meet the criteria nominated by the Council/approval authority then it must be stored on site for treatment prior to disposal or re-injection into the aquifer. This will require a location where the water can be stored for sedimentation, flocculation, or pH correction. Alternatively, if required, consideration may be given to disposal to the sewer system, which allows higher levels of contaminants, and requires a trade waste agreement with Sydney Water.

6.5 Retaining Walls

Vertical excavations within the sand will require retaining structures both during construction and as part of the final structure. It is anticipated that shoring will be required around the perimeter of the basement. Deeper shoring walls may be needed to reduce groundwater inflow.

Design of cantilevered shoring systems or shoring with a single row of anchors may be based on an average unit weight of 20 kN/m^3 for the retained soil, with a triangular earth pressure distribution calculated using an active earth pressure coefficient (K_a) value of 0.35 where some wall movement is acceptable, or an at rest earth pressure coefficient (K_o) value of 0.5 where wall movement is to be reduced. A coefficient of passive earth pressure (K_p) equal to 3.0 within loose to medium dense sands and 4.0 within dense to very dense sands may be assumed below the bulk excavation level. These K_p values represent ultimate values which are mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety to reduce wall movement.

In design of the retaining walls due allowance should be made for surcharge loads including plant operating above the excavation during construction and also hydrostatic pressures due to the groundwater table.

The most suitable type of shoring system will depend on the actual depth of excavation and proximity to nearby structures. Various options for shoring systems are described below:

- A secant pile wall would be suitable for the site, comprising interlocking Continuous Flight Auger (CFA) piles or CFA piles with jet grouted columns between the piles. This shoring system can generally provide an effective seal to minimise sand loss and water inflow from behind the wall, and if adequately supported, minimise lateral deflections. The ‘hard’ (steel reinforced) concrete piles can be incorporated into the vertical load carrying footing system and can generally form part of the basement structure;
- Soil mixed wall systems have been used as an alternative to the more conventional secant pile wall. These walls are constructed using specialised equipment to blend cement with the in-situ soils to create a soil-cement mix. There are several different systems available and further advice should be obtained from the specialist piling contractor regarding the suitability of the wall system to this site. In particular, confirmation should be sought in relation to the consistency/strength of the soil mixed wall, the long term durability, permeability, potential issues with blending cement and joining the soil mixed wall with the tanked basement slab;
- Sheet piles are generally suitable for shallower excavations above the water table and where there are no movement sensitive structures adjacent to the excavation. The use of sheet pile walls is subject to noise and vibration controls and achieving adequate penetration depth. Consideration should take into account appropriate sheet thickness and vibrations associated with installation. Pre-drilling may be required to reduce vibrations and reduce the possibility of buckling when driving through medium dense or dense sand. Due to the proximity of adjacent buildings and presence of dense sands, sheet piles are probably not suitable for the site although there are some systems that can push the sheets without vibrations; and

- A contiguous pile wall comprising closely spaced/touching CFA piles is also not recommended for this site due to risks associated with seepage and sand loss in between the piles, particularly below the groundwater table.

Design of temporary anchors within loose to medium dense sand may be based on a friction angle (ϕ) of 30 to 33 degrees. The bond stress between the sand and anchors could be increased if a pressure-grouted anchor system is adopted. Design bond stress values should be confirmed by specialist contractors for their method of anchor installation. It is recommended that trial anchors be installed to prove that the required capacities can be achieved. The anchors should be bonded behind an imaginary line drawn up at 45 degrees from the base of the excavation.

6.6 Nearby Structures

In the case of the retained building on 10-12 Doncaster Avenue which is sensitive to movement, it is suggested that the basement retaining walls near the building are designed using the ‘at rest’ earth pressure coefficient to reduce any lateral movement of the walls and thus reduce the risk of potential damage to the building during excavation and construction works.

The survey and architecture drawings show that the acoustic walls on the tram shed site are 2 m from the common boundary and the basement on 4-18 Doncaster Avenue is also 2 m from the common boundary. This places the acoustic wall outside an imaginary 45^{o} line from the base of the bulk excavation and therefore outside the major zone of influence of a basement excavation.

6.7 Subgrade Preparation

It is suggested that site preparation and engineered filling for lightly loaded pavements and slabs on ground should incorporate the following:

- Following excavation to achieve design subgrade levels, the exposed soil surface should be thoroughly rolled with a minimum of eight passes using an appropriately sized smooth drum roller (say 8 tonne static weight). The final pass (proof roll) should be inspected by a geotechnical engineer to help identify any soft or heaving areas. Any “soft spots” detected during proof rolling should be stripped to a stiff base and replaced with engineered filling. CPT101 encountered very soft to firm organic clay/peat to about 5 m depth which will not be suitable to support floor slabs and footings. This soft clayey soil will need to be removed and replaced with engineered filling unless the structure is suspended on piles. Further investigation will be required to assess the extent of the soft clayey soils on the site;
- Engineered filling should be placed in layers and compacted to a minimum dry density ratio of 98% relative to Standard compaction (or density index of 75% for clean sand soils) and within 2% of the optimum moisture content (OMC). The density ratio should be increased to 100% relative to standard compaction (or density index of 80%) within 0.3 m of the design surface level. From a geotechnical point of view, the existing natural sand on site should be suitable for reuse as engineered filling provided it is free of organic and obvious deleterious material. Reuse of soil on site will need to consider the contamination status and potential acid sulphate soils which would require further investigation. For imported material, if required, preference should be given to the use of good quality granular material such as ripped medium to high strength sandstone; and

- Density testing of each layer of filling should be undertaken in accordance with AS 3798-2007 “Guidelines for Earthworks for Commercial and Residential Developments” to verify that the specified compaction has been achieved.

Piling rigs will presumably be required to operate on the site and may require the construction of a working platform to allow these rigs to operate and traverse on the exposed sandy soils. The platform, where required, may be constructed from good quality granular material such as recycled concrete, crushed rock, or high strength sandstone. The thickness of the platform will need to be assessed once specific details of the piling rig loads are known.

6.8 Foundations

6.8.1 Shallow Foundations

Based on the CPT results, the foundations below the proposed on grade and basement level will include very loose to loose sands approximately 1 m to 3 m thick over medium dense sand grading to dense to very dense sand. However, CPT101 encountered very soft to firm organic clay/peat to a depth of 5 m (to about 3.5 m below the basement level).

The very loose to loose sands will offer relatively limited bearing capacity for pad or strip footings with excessive total and differential settlements for a three storey building. The soft clay/peat is unsuitable to support structural loads and therefore this material will need to be removed unless structures are suspended on piles, where soft clay is encountered. Further investigation is required to delineate and assess the extent of soft clay on the site.

Excavation to expose medium dense sands for higher bearing capacities will involve excavation below the groundwater table and associated dewatering and may not be a practical or economical solution.

The allowable end bearing pressure in sands will depend on the density/strength of the foundations, depth of embedment and size of the footing and depth to groundwater. As a guide, allowable end bearing pressures and elastic modulus values for the typical soil strata are provided in Table 2. The allowable end bearing pressures shown in Table 2 are based on a pad footing with a plan area of 2 m by 2 m, embedment of 0.5 m, groundwater at the surface and a factor of safety equal to 2.5.

Table 2: Summary of Typical Design Parameters for Shallow Foundations

Foundation Material	Allowable End Bearing (kPa)	Elastic Modulus (MPa)
Sand: Loose*	100	20
Sand: Medium Dense to Dense	200	80
Sand: Dense to Very Dense	300	100

Note: * footings on very loose sand and organic clay/peat are not recommended and excessive settlements may occur for relatively high loads.

6.8.2 Raft Slabs

Consideration may be given to the use of a raft slab foundation. This will however be subject to detailed review and analysis of bearing pressures and settlements once more specific details of the founding level, column layout and slab loadings have been confirmed. The presence of the very loose to loose sand and soft organic clay/peat below the raft slab should be considered in the design, particularly for the concentrated column loadings. Consideration could be given to removing some of the very loose to loose sand to improve the foundations however this would be problematic due to the groundwater table. As discussed above, the soft organic clay/peat is not suitable to support structural loads and therefore some form of bridging layer or possibly a piled raft may be required where soft clays are encountered and cannot be removed.

Details of structural loads were not available at the time of preparing this report. Based on similar sized projects it is anticipated that a distributed slab load in the order of 30 kPa to 40 kPa may be applicable for the three storey building with a basement carpark. As a guide, for raft slab foundations, preliminary settlement analyses has been carried out assuming a distributed slab load of 40 kPa, with a loaded area of 20 m by 20 m. Based on the results of the analyses, preliminary design of raft slabs to support column and floor loadings may be based on a modulus of subgrade reaction (k) value of the order of 3 kPa/mm to 5 kPa/mm for the broad loaded area. Settlements of between 5 mm to 15 mm could therefore be expected under the assumed loads. It is noted that the k value is the expression of the settlement under a specific loading, and as such the k value (which is not strictly a soil parameter) is heavily dependent on the size of the loaded area and the rigidity of the raft system.

Construction of the raft slabs should incorporate subgrade preparation as outlined in Section 8.6. It is also suggested that a 150 mm thick layer of good quality granular material such as recycled concrete or crushed rock should be placed and compacted over the prepared surface, particularly at the more heavily loaded areas. The granular layer will help to confine the sandy soils and improve the compaction and density of the surface soils.

A piled raft foundation may also be considered to reduce differential settlements, if required.

Further geotechnical analysis and advice will be required in relation to the design and construction of both raft slabs and piled raft slabs, if these are to be considered.

6.8.3 Piled Foundations

The alternative to shallow foundations is to support the structural loads on piles founded within at least dense sand which is typically at depths of approximately 3.8 m to 5.3 m below the existing surface level. The pile design will need to consider the presence of underlying weaker layers, particularly at CPT2 where very loose/silty bands were encountered between depths of 7.8 m to 9.6 m. Piles may be required in areas where soft clays are encountered (i.e. CPT101).

It is expected that noise and vibration constraints at this site will preclude the use of driven pile types. Similarly, the adoption of open conventional bored piles will not be appropriate due to the potential for soil collapse and groundwater inflow.

Continuous Flight Auger (CFA), concrete injected piles could be considered for this site, as could cast-in-situ screwed pile types such as Atlas or Omega piles. These types of piles are all associated with relatively low levels of noise and vibration. Screwed cast in-situ piles leave a reinforced concrete screw shaped pile and involve lateral displacement of the soil during installation, allowing use of the in-situ capacity of the soil.

Steel screw piles may be considered subject to confirmation of their load carrying capacity and durability. Steel screw piles are a proprietary product, and as such information on their installation and load carrying capacity must be obtained from the specialist contractor. Based on previous experience with steel screw piles, a maximum working capacity (vertical load) of about 500 kN to 600 kN is usually achievable. Higher capacities may be possible, however it would be prudent to carry out a load testing programme to prove the load capacities of heavily loaded piles and ensure that excessive settlements do not occur under load.

As a guide for design of piles in soil, preliminary estimates of the geotechnical capacity of grout or concrete-injected piles (0.6 m diameter) are provided in Appendix D. The pile capacity estimates are calculated using ConePile which is an in-house DP pile analysis and design program. The pile capacity estimates indicate the assessed ultimate end bearing and shaft friction values with depth together with an ultimate geotechnical ($R_{d,ug}$) and design strength ($_{Rd,g}$) for the piles at varying depths. The design geotechnical strength is based on an assumed geotechnical strength reduction factor ($Ø_g$) of 0.45. This $Ø_g$ value, however, should be determined by the designer in accordance with the AS2159 (November 2009). The selection of $Ø_g$ is based on a series of individual risk ratings (IRR) which are weighted to give an average risk rating (ARR). The IRR values depend on factors such as the type and quality of testing, design method and parameter selection, pile installation control and monitoring, pile testing regime, and the redundancy in the foundation system.

7. References

1. Department of Land and Water Conservation (GWMA018, March 2000) “Botany Sand Beds, Botany Basin, NSW Northern, Southern and Western Zones Status Report No.2” Merrick (1998)
2. “Evolution of Groundwater Levels and Usage in the Botany Basin” Environmental Geology of the Botany Basin, Ed McNally and Jankowski (pp 230-239).

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 4-18 Doncaster Avenue, Kensington in accordance with DP’s proposal dated 5 December 2018 and acceptance received from Matthew Hill of Sky Blue Commercial Asset Managers Pty Ltd dated 5 December 2018. The work was carried out under DP’s Conditions of Engagement. This report is provided for the exclusive use of Sky Blue Commercial Asset Managers 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 sampling and/or 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 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

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

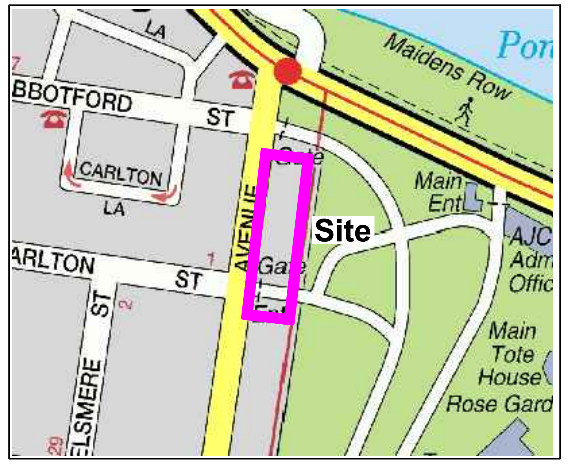
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

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

Appendix B

Drawings



Locality Plan

LEGEND

- Previous CPT (1,2,3,4)
- Previous borehole and monitoring well (project 29615)
- Current CPT (101, 102)
- A — A' Interpreted Geotechnical Cross Section A-A'

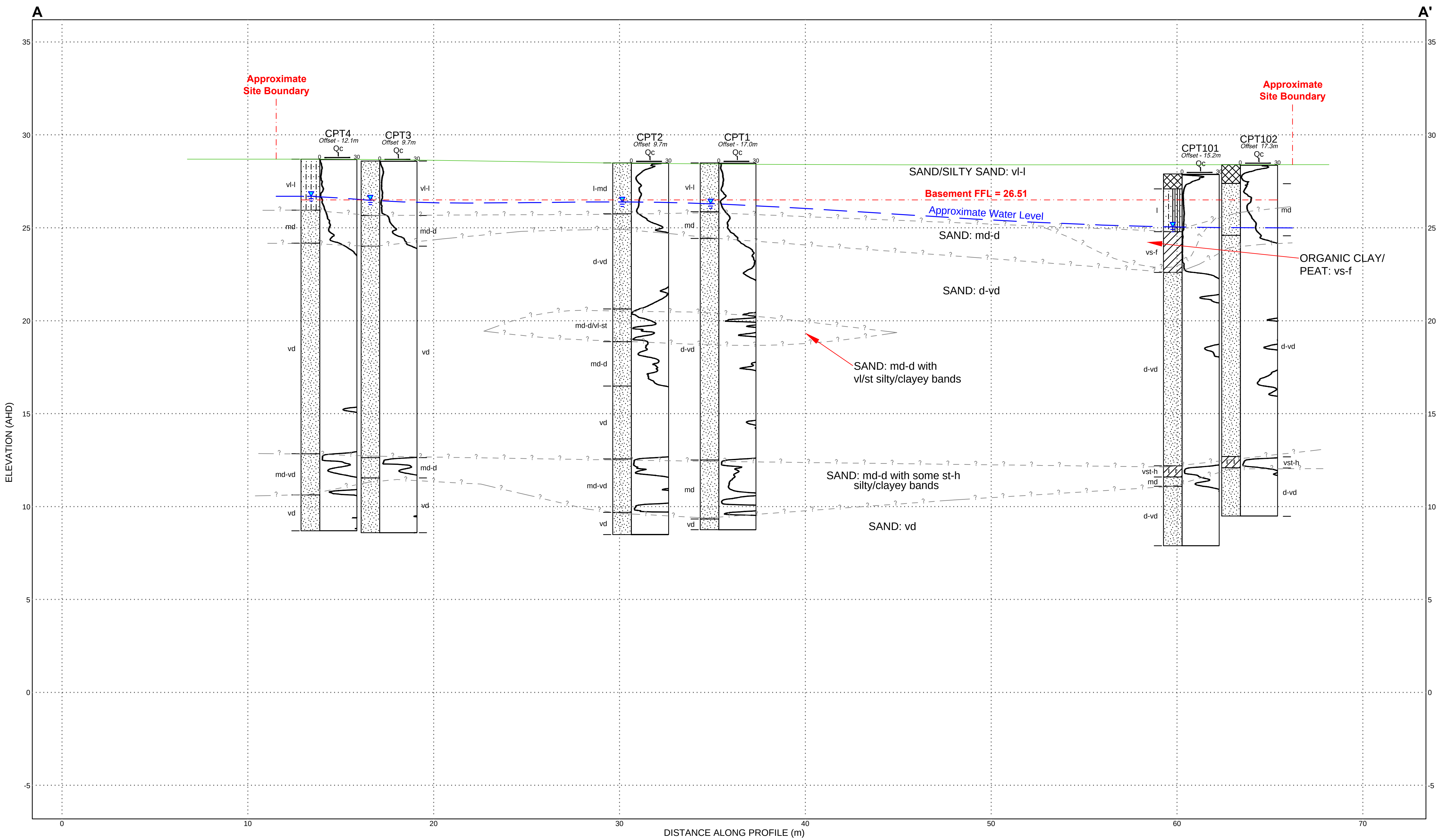
NOTE:

1: Base image from Nearmap
2: Test locations are approximate only and are shown with reference to existing featues.

0 5 10 15 20 30 40 60m

1:600 @ A3

Douglas Partners — Geotechnics / Environment / Groundwater	CLIENT: Built (NSW) Pty Ltd		TITLE: Test Locations	PROJECT No: 73965.02
	OFFICE: Sydney	DRAWN BY: PSCH	Proposed Residential Development	DRAWING No: 1
	SCALE: 1:600 @ A3 approx.	DATE: 17.12.2015	4-18 Doncaster Avenue, KENSINGTON	REVISION: 0



LEGEND

- Sand
- Filling
- Clay
- Silty Sand
- Silty Clay

NOTE:

1. Subsurface conditions are accurate at test locations. Variations in subsurface conditions may occur between test locations. Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only. Should be read in conjunction with detailed CPT logs.

SOIL CONSISTENCY

vs - very soft	vl - very loose
s - soft	l - loose
f - firm	md - medium dense
st - stiff	d - dense
vst - very stiff	vd - very dense
h - hard	

TESTS / OTHER

- Water level

0 8
Horizontal Scale (metres)

Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Built (NSW) Pty Ltd		TITLE: **Interpreted Geotechnical Cross-Section A-A'** **Proposed Residential Development** **4-18 Doncaster Avenue, KENSINGTON**	PROJECT No: 73965.02
OFFICE: Sydney	DRAWN BY: LJH / STE		DRAWING No: 2
SCALE: 1:400 (H) 1:200 (V) @ A3	DATE: 18.12.2015		REVISION: 0

Appendix C

CPT Results

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

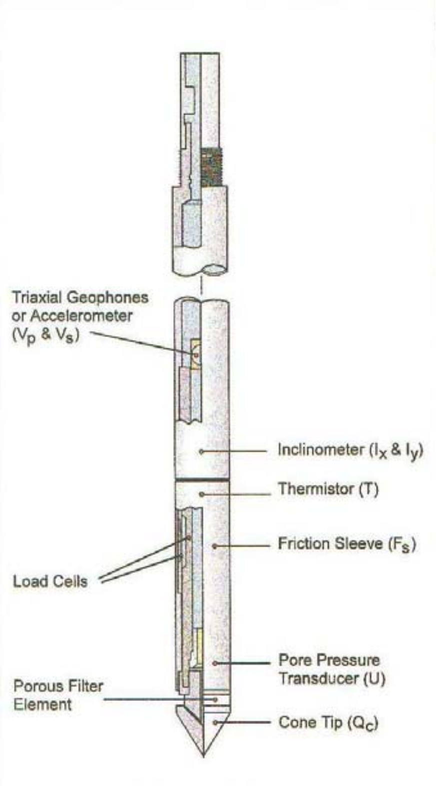


Figure 1: Cone Diagram

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 (σ) 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)

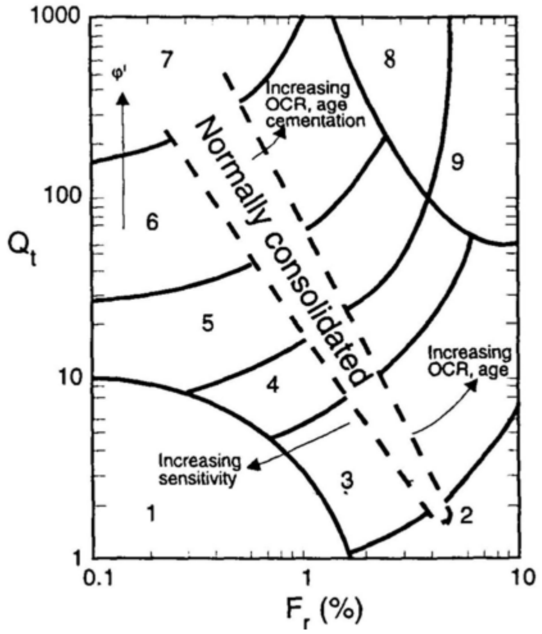


Figure 3: Soil Classification Chart

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.

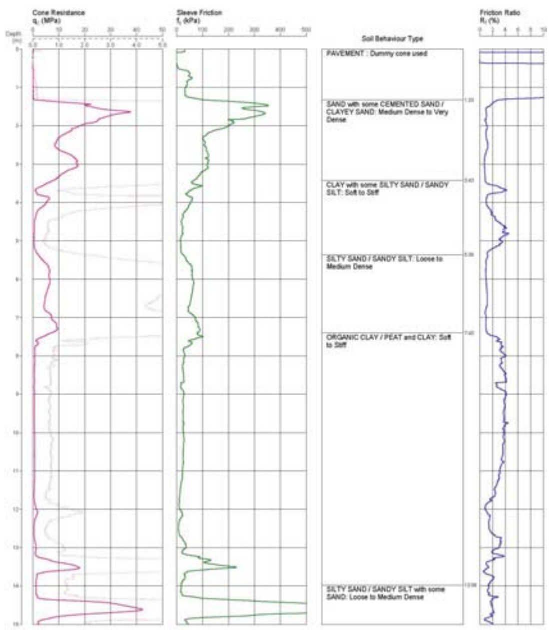


Figure 4: Sample Cone Plot

CONE PENETRATION TEST

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-12 DONCASTER AVENUE, RANDWICK

REDUCED LEVEL: 28.5

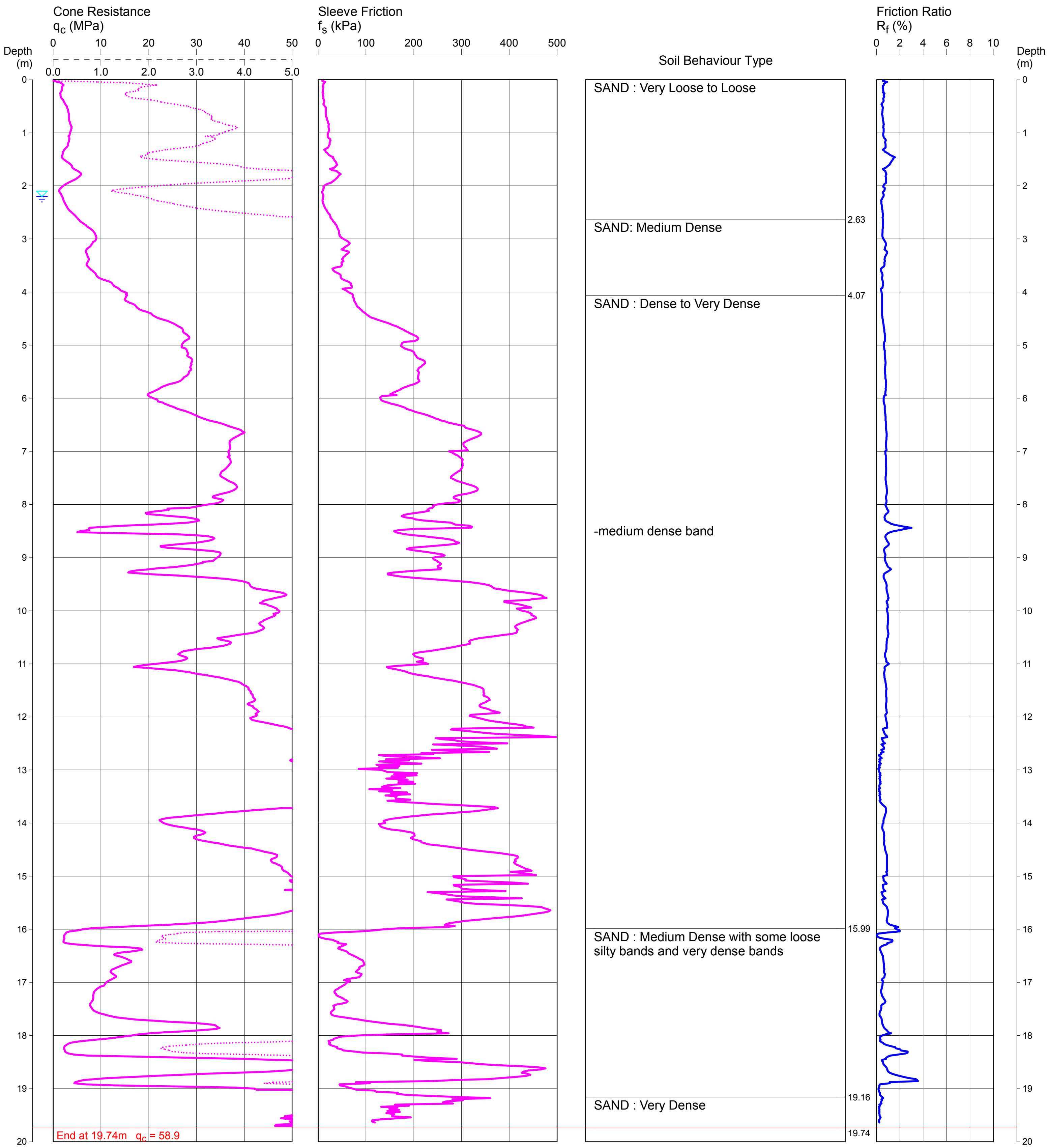
COORDINATES:

CPT1

Page 1 of 1

DATE 13/5/2014

PROJECT No: 73965



REMARKS: HOLE DISCONTINUED DUE TO EXCESSIVE ROD BOWING
GROUNDWATER OBSERVED AT 2.2 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.20m depth (assumed)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT1.CP5

Cone ID: 120619 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: BLUE SKY COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-12 DONCASTER AVENUE, RANDWICK

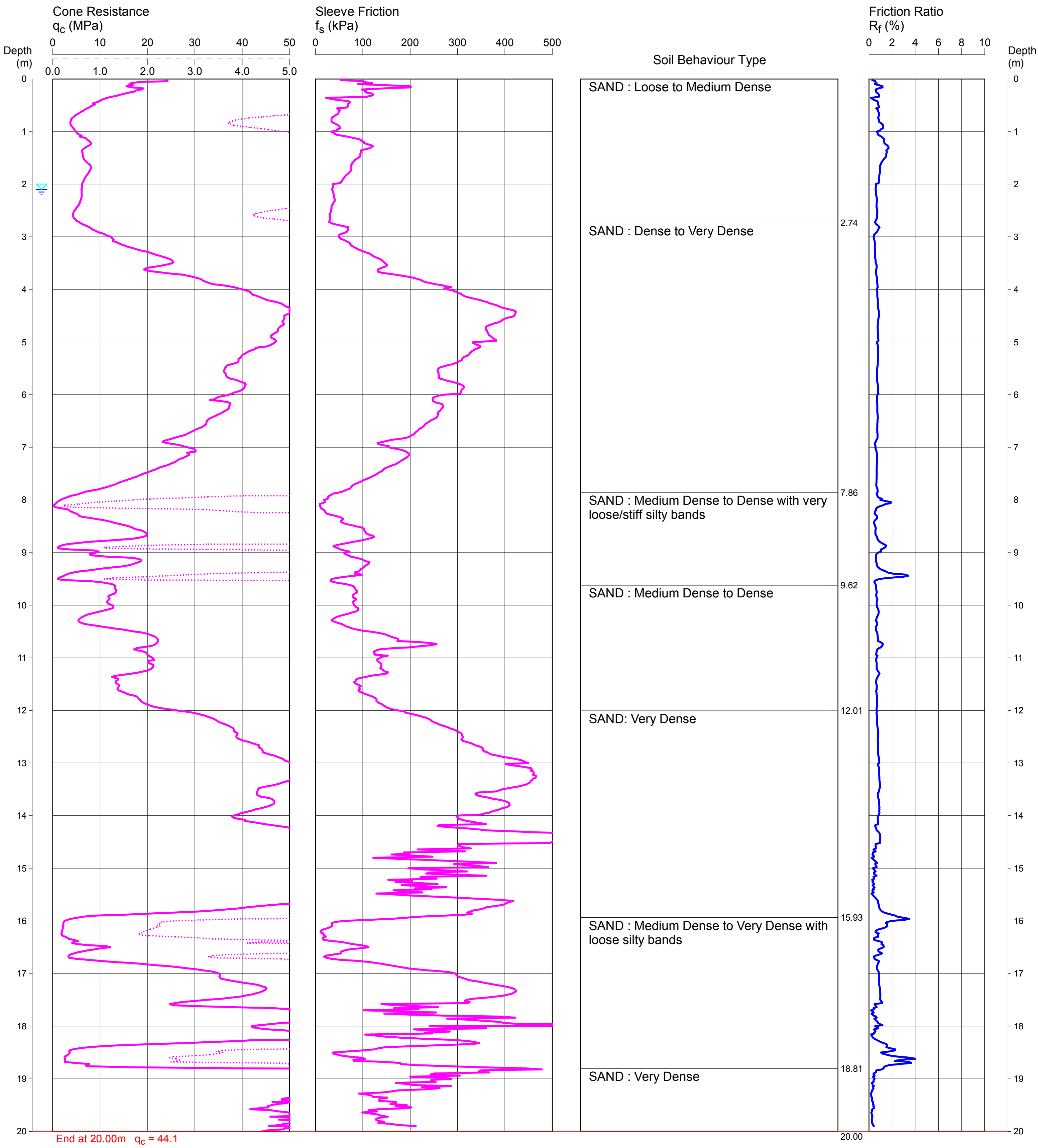
REDUCED LEVEL: 28.5

COORDINATES:

CPT2

DATE 13/5/2014

PROJECT No: 73965



REMARKS: GROUNDWATER OBSERVED AT 2.1 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.10m depth (assumed)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT2.CP5
Cone ID: 120619 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-12 DONCASTER AVENUE, RANDWICK

REDUCED LEVEL: 28.6

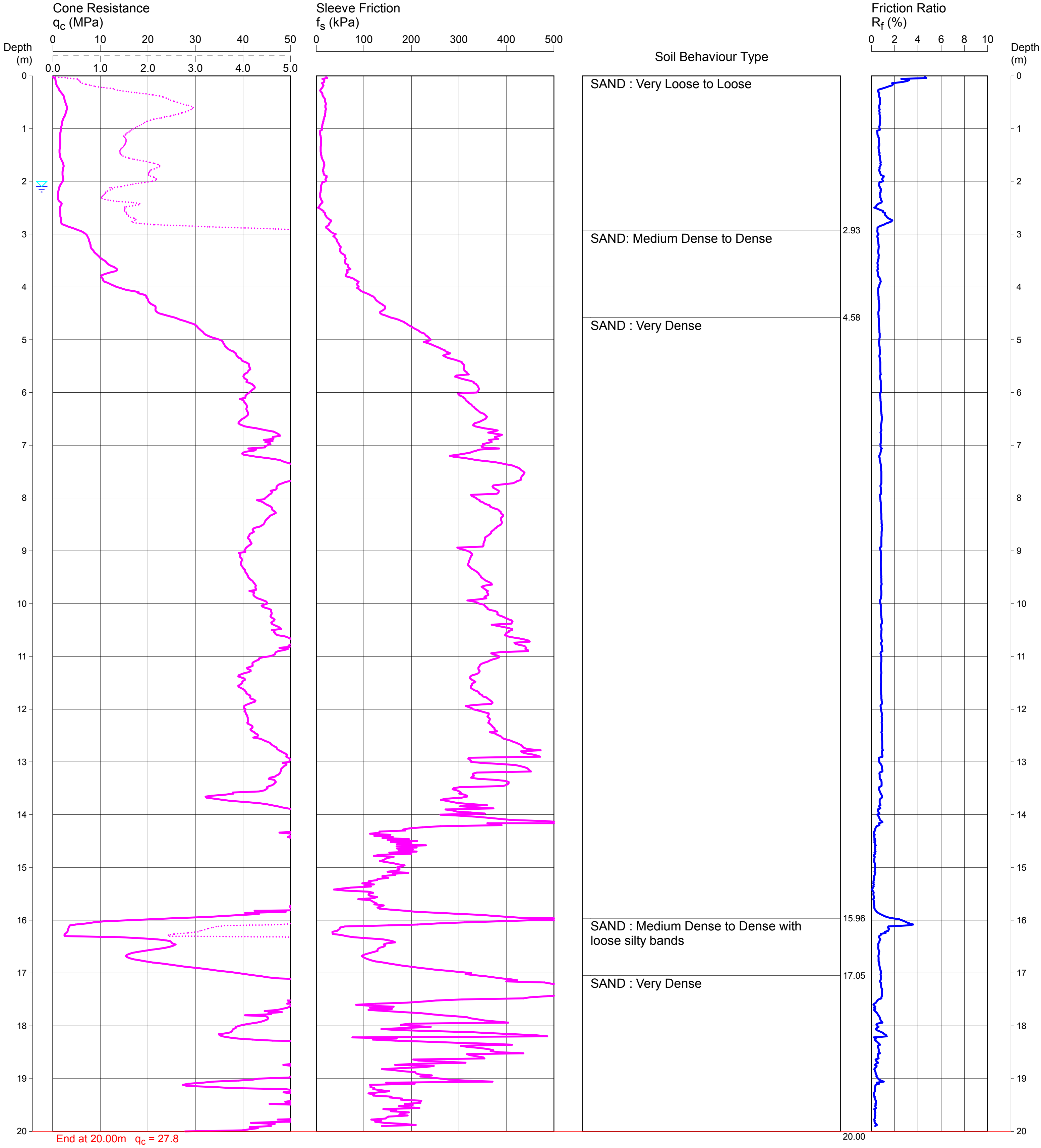
COORDINATES:

CPT3

Page 1 of 1

DATE 13/5/2014

PROJECT No: 73965



REMARKS: GROUNDWATER OBSERVED AT 2.1 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.10m depth (assumed)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT3.CP5

Cone ID: 120619 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-12 DONCASTER AVENUE, RANDWICK

REDUCED LEVEL: 28.6

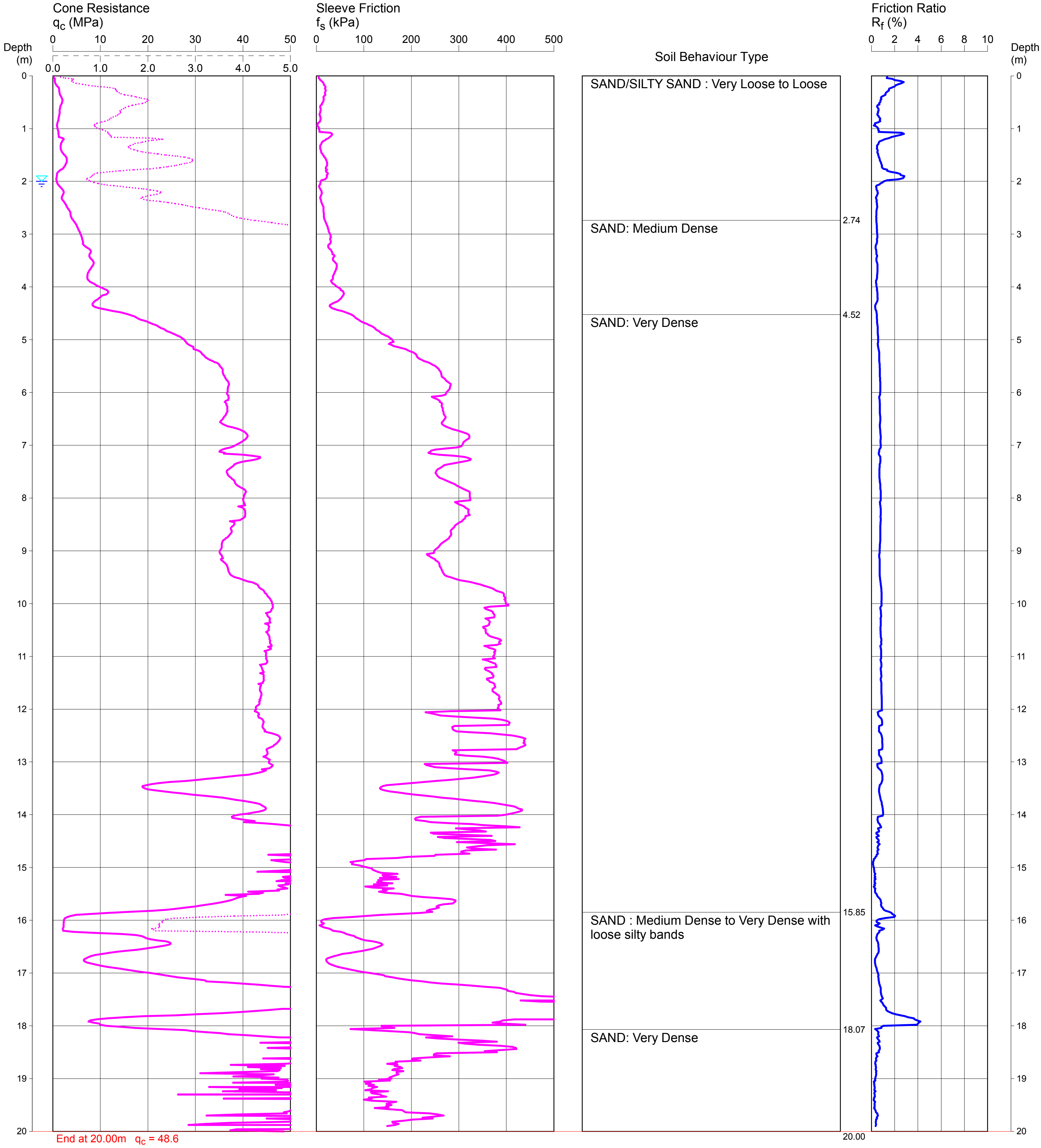
COORDINATES:

CPT4

Page 1 of 1

DATE 13/5/2014

PROJECT No: 73965



REMARKS: GROUNDWATER OBSERVED AT 2.0 m DEPTH AFTER WITHDRAWAL OF RODS

Water depth after test: 2.00m depth (assumed)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT4.CP5

Cone ID: 120619 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners

Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-18 DONCASTER AVENUE, KENSINGTON

REDUCED LEVEL: 27.9

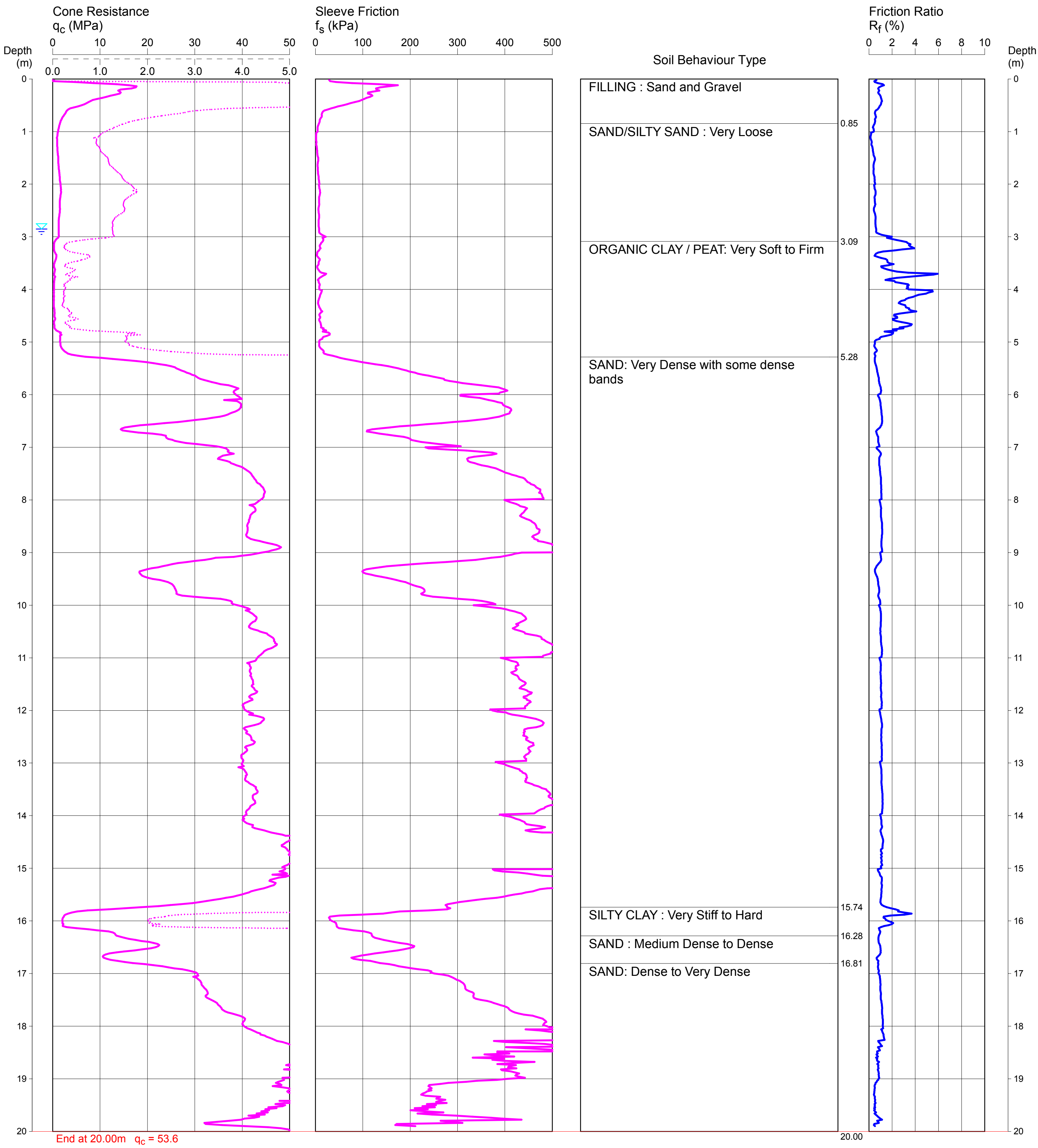
COORDINATES: 336013E 6246850N AHD

CPT101

Page 1 of 1

DATE 9/12/2015

PROJECT No: 73965.02



REMARKS: STANDPIPE INSTALLED TO 4.9 m DEPTH AFTER WITHDRAWAL OF RODS.
GROUNDWATER OBSERVED AT 2.85 m DEPTH IN INSTALLED STANDPIPE.

Water depth after test: 2.85m depth (measured)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT101.CP5
Cone ID: 120620 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-18 DONCASTER AVENUE, KENSINGTON

REDUCED LEVEL: 28.4

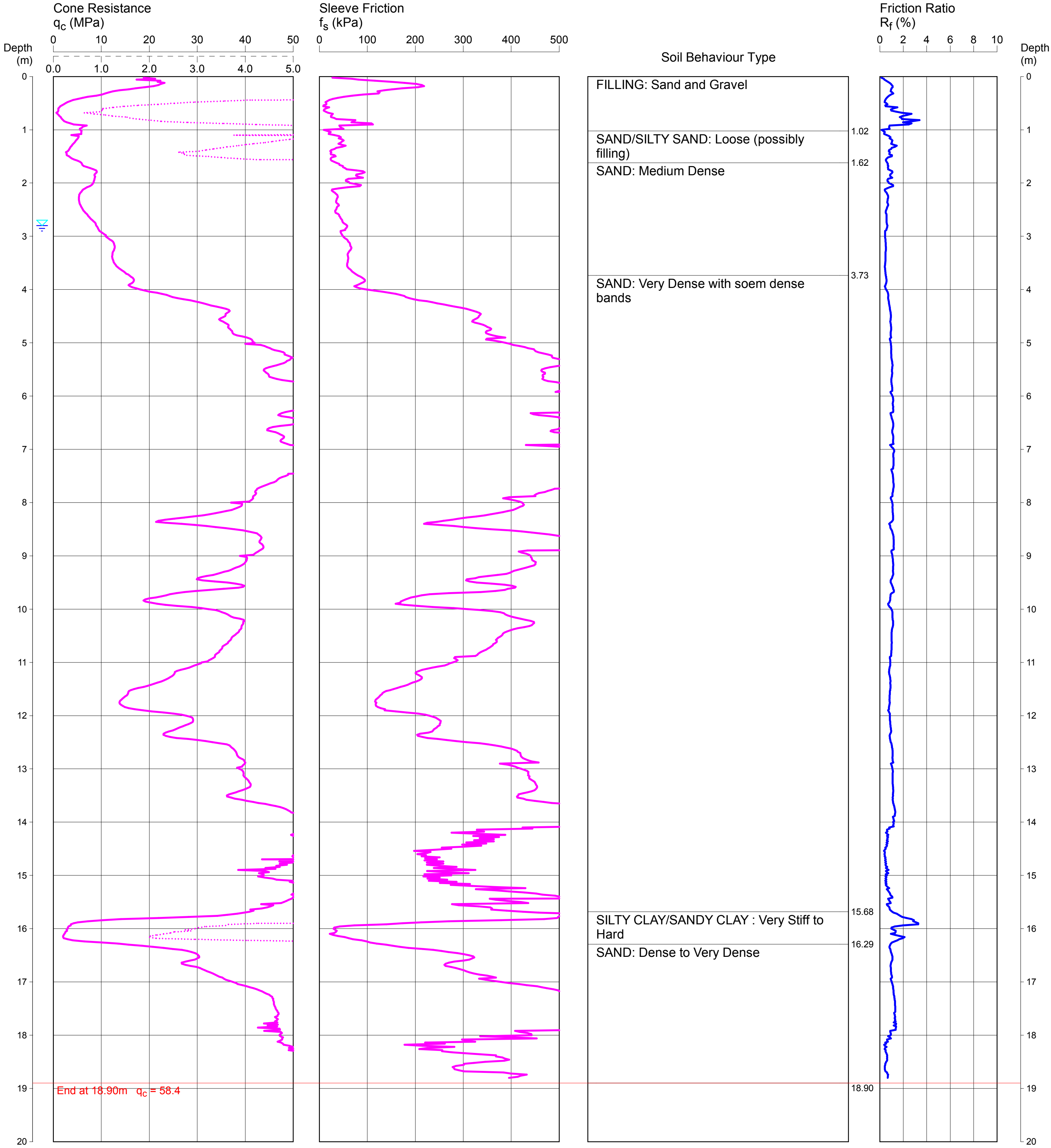
COORDINATES: 336049E 6246847N AHD

CPT102

Page 1 of 1

DATE 9/12/2015

PROJECT No: 73965.02



REMARKS: HOLE DISCONTINUED DUE TO LIMIT OF THRUST.
HOLE COLLAPSE AT 2.8 m DEPTH AFTER WITHDRAWAL OF RODS.

Water depth after test: 2.80m depth (assumed)

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT102.CP5
Cone ID: 120620 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix D

Pile Capacity Estimates

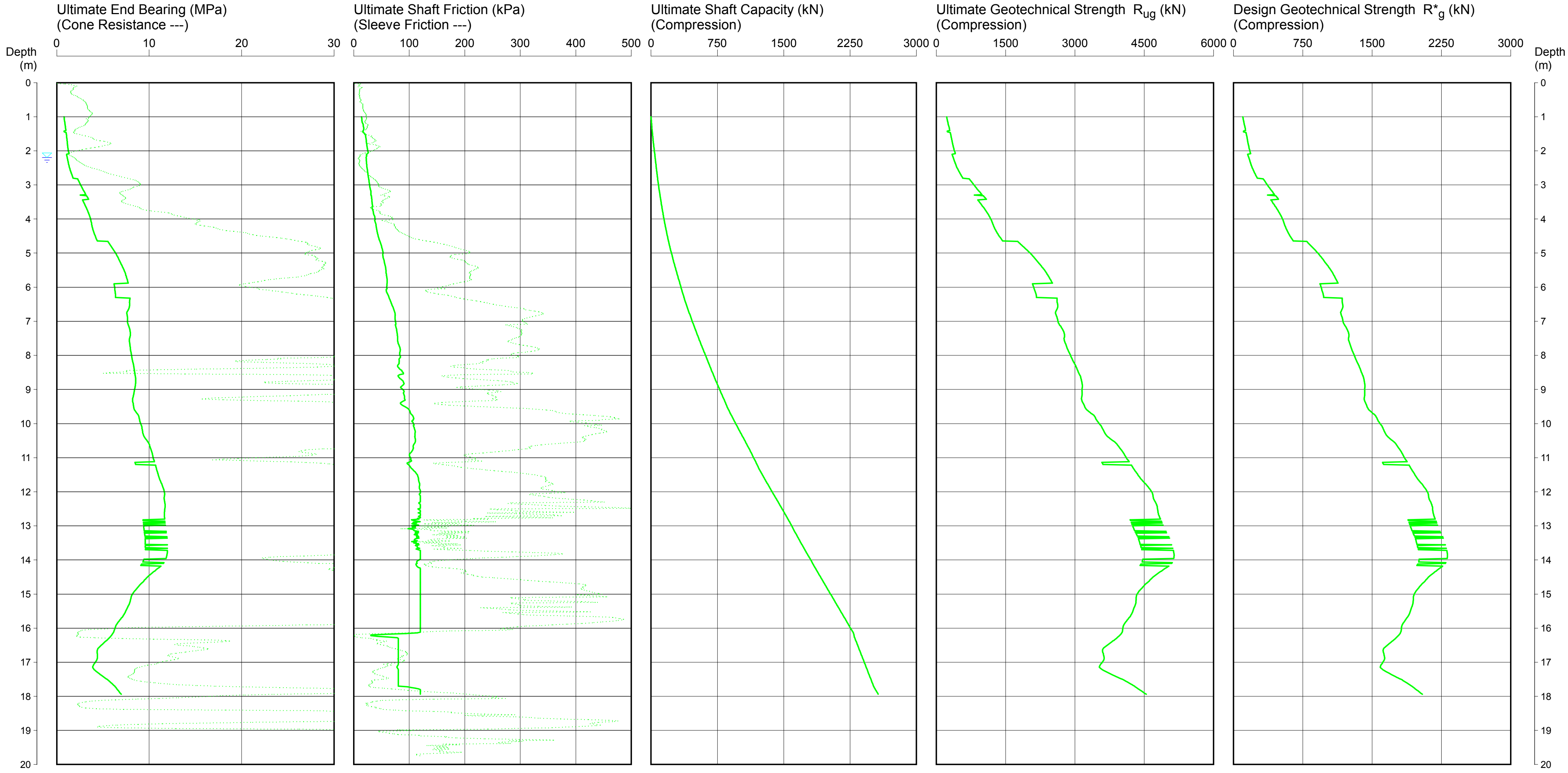
PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT
LOCATION: 4-12 DONCASTER AVENUE, RANDWICK
CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

CPT1

DATE 13/05/2014
PROJECT No: 73965
SURFACE RL: 28.5



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.20m depth

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT1.CP5
Cone ID: 120619 **Type:** I-CFXY-10

ConePile Version 5.9.1

Douglas Partners
Geotechnics / Environment / Groundwater

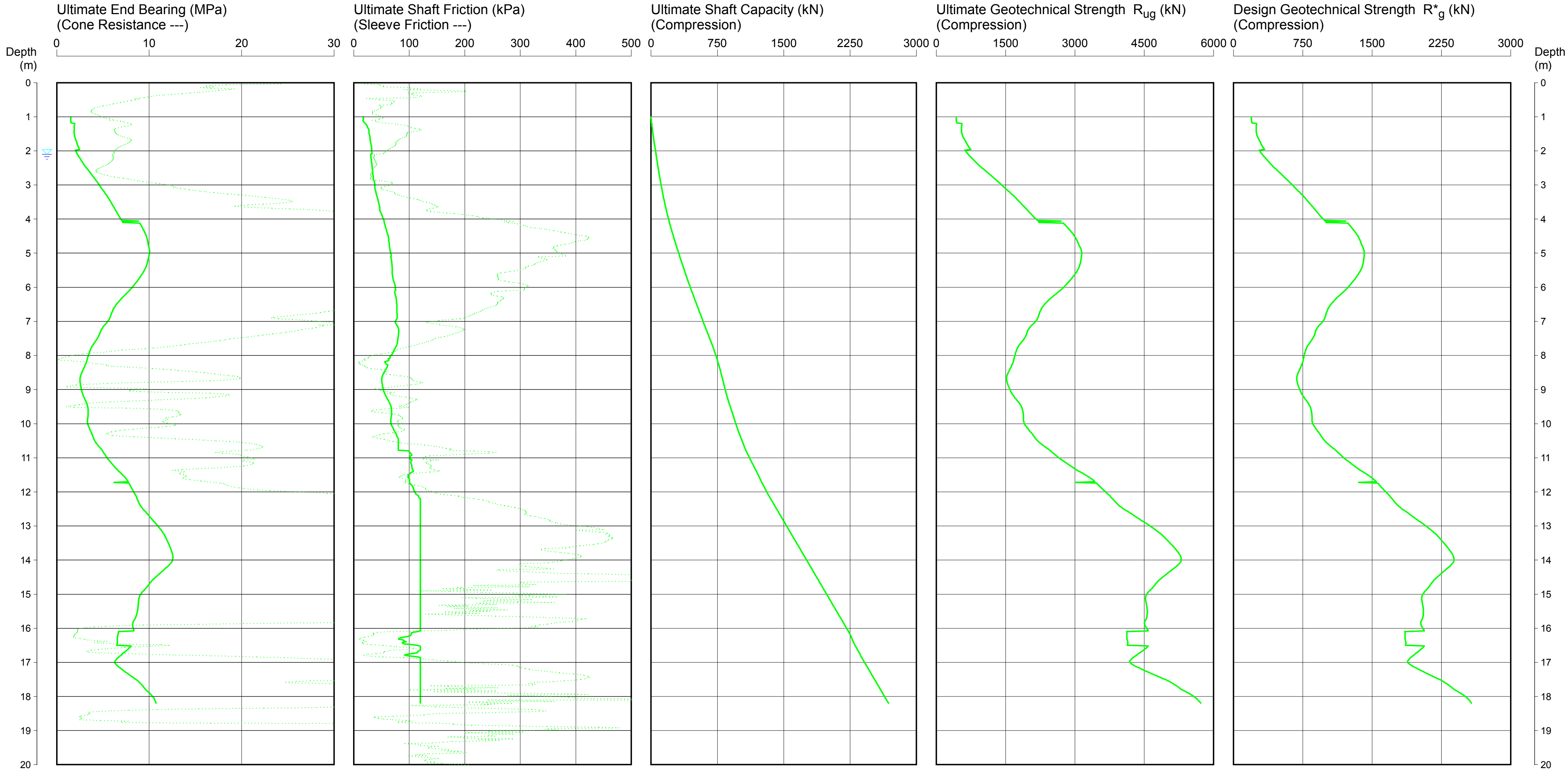
PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT
LOCATION: 4-12 DONCASTER AVENUE, RANDWICK
CLIENT: BLUE SKY COMMERCIAL ASSET MANAGERS PTY LTD

CPT2

DATE 13/05/2014
PROJECT No: 73965
SURFACE RL: 28.5



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.10m depth

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT2.CP5
Cone ID: 120619 **Type:** I-CFXY-10

ConePile Version 5.9.1

Douglas Partners
Geotechnics / Environment / Groundwater

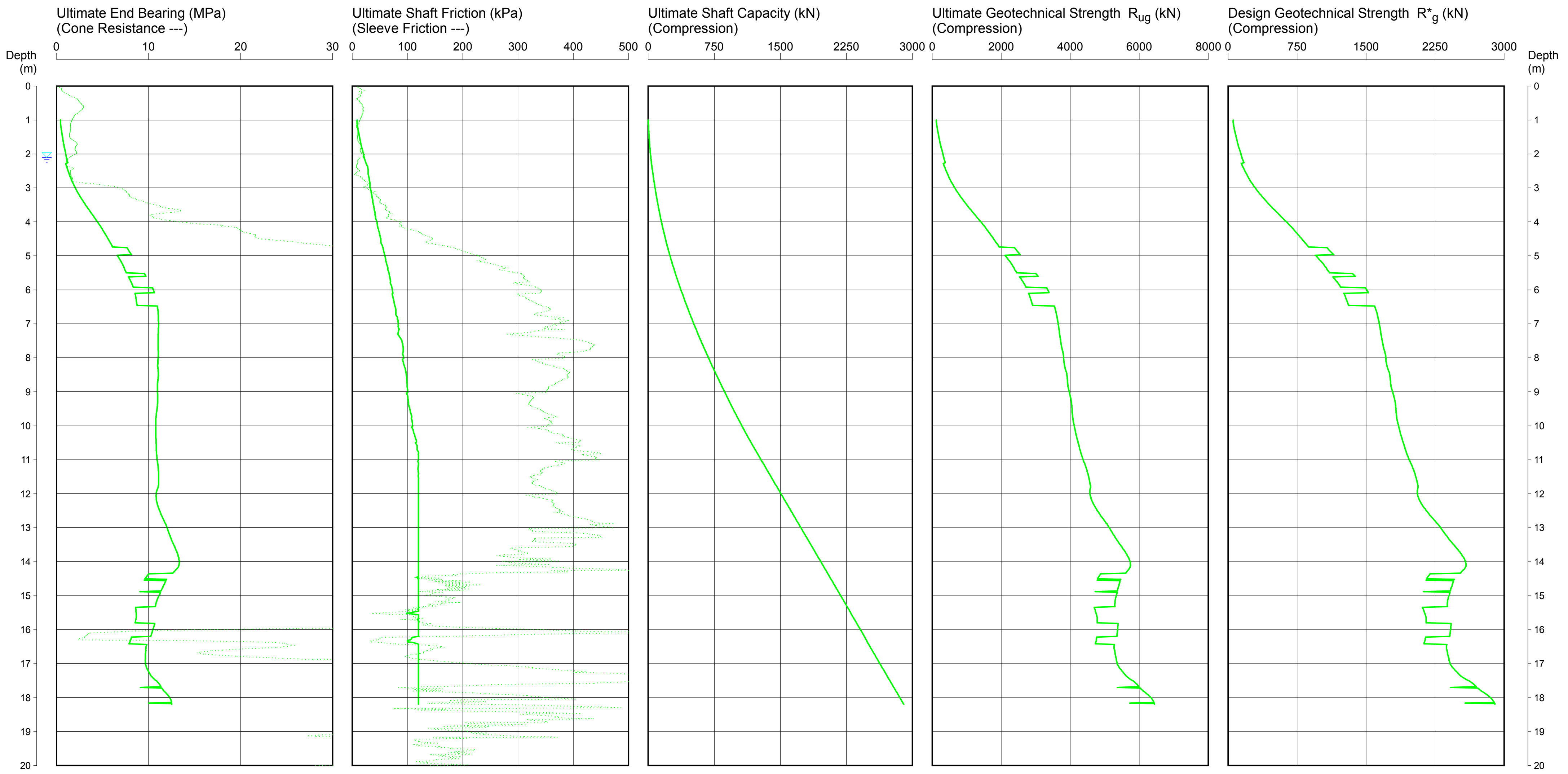
PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT
LOCATION: 4-12 DONCASTER AVENUE, RANDWICK
CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

CPT3

DATE 13/05/2014
PROJECT No: 73965
SURFACE RL: 28.6



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.10m depth

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT3.CP5
Cone ID: 120619 **Type:** I-CFXY-10

ConePile Version 5.9.1

Douglas Partners
Geotechnics / Environment / Groundwater

PILE CAPACITY ESTIMATE

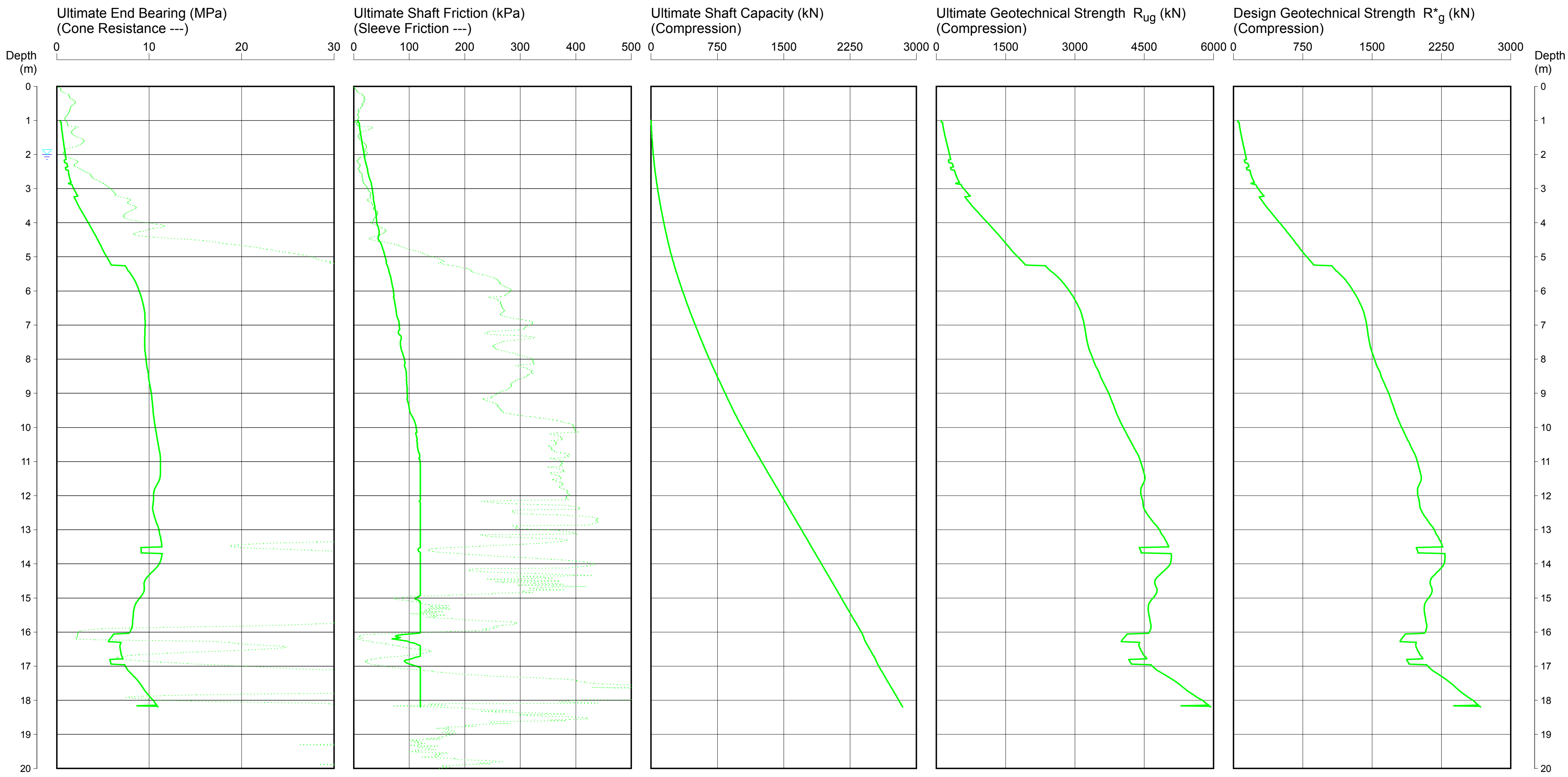
PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT
LOCATION: 4-12 DONCASTER AVENUE, RANDWICK
CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

CPT4

Page 1 of 1

DATE 13/05/2014
PROJECT No: 73965
SURFACE RL: 28.6



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.00m depth

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT4.CP5
Cone ID: 120619 **Type:** I-CFXY-10

ConePile Version 5.9.1

Douglas Partners
Geotechnics / Environment / Groundwater

PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

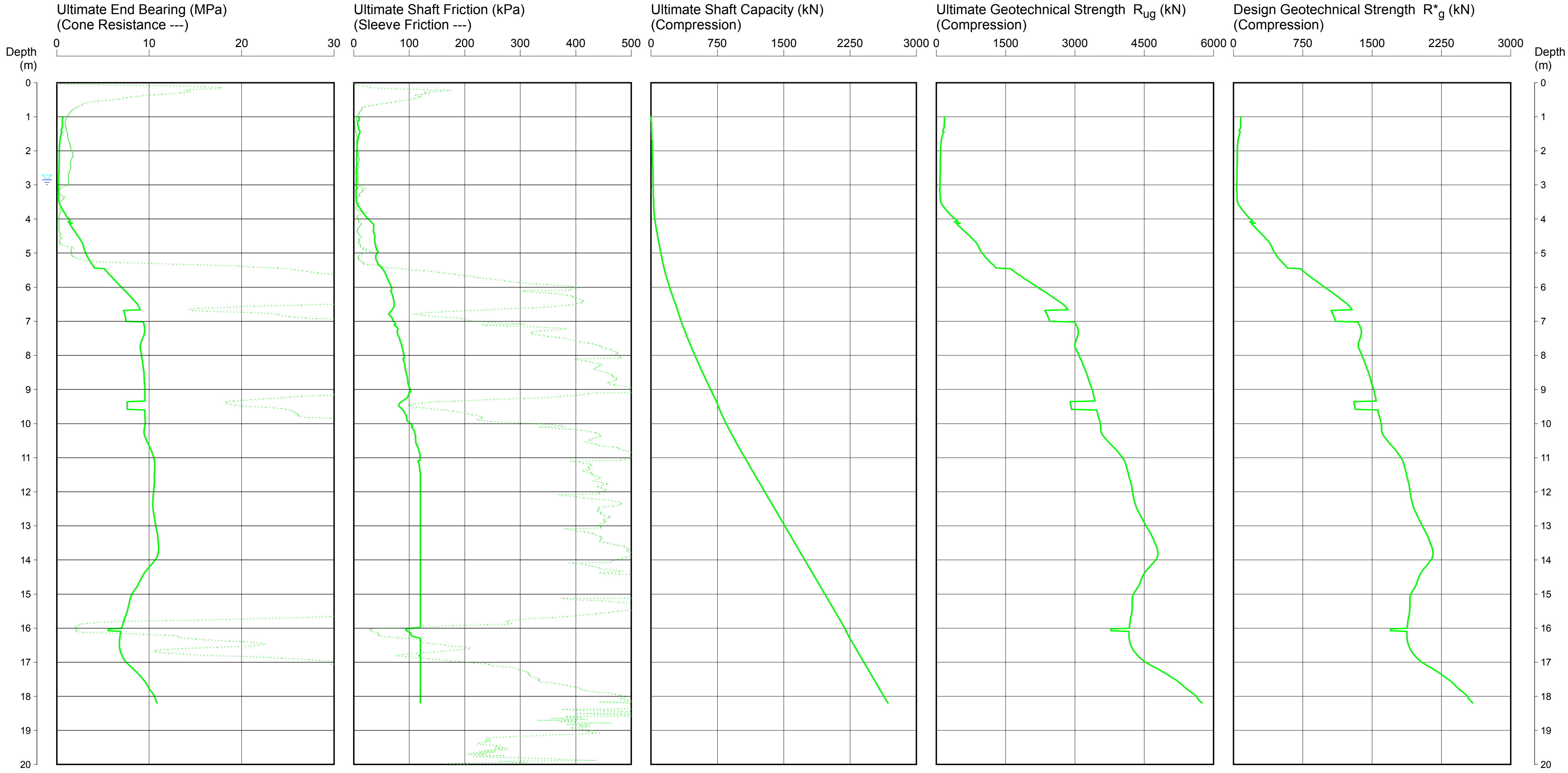
LOCATION: 4-18 DONCASTER AVENUE, KENSINGTON

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

CPT101

Page 1 of 1

DATE 9/12/2015
PROJECT No: 73965.02
SURFACE RL: 27.9



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.85m depth

Coordinates: 336013 6246850

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT101.CP5
Cone ID: 120620 **Type:** I-CFXY-10

ConePile Version 5.9.1

Douglas Partners
Geotechnics / Environment / Groundwater

PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.60
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

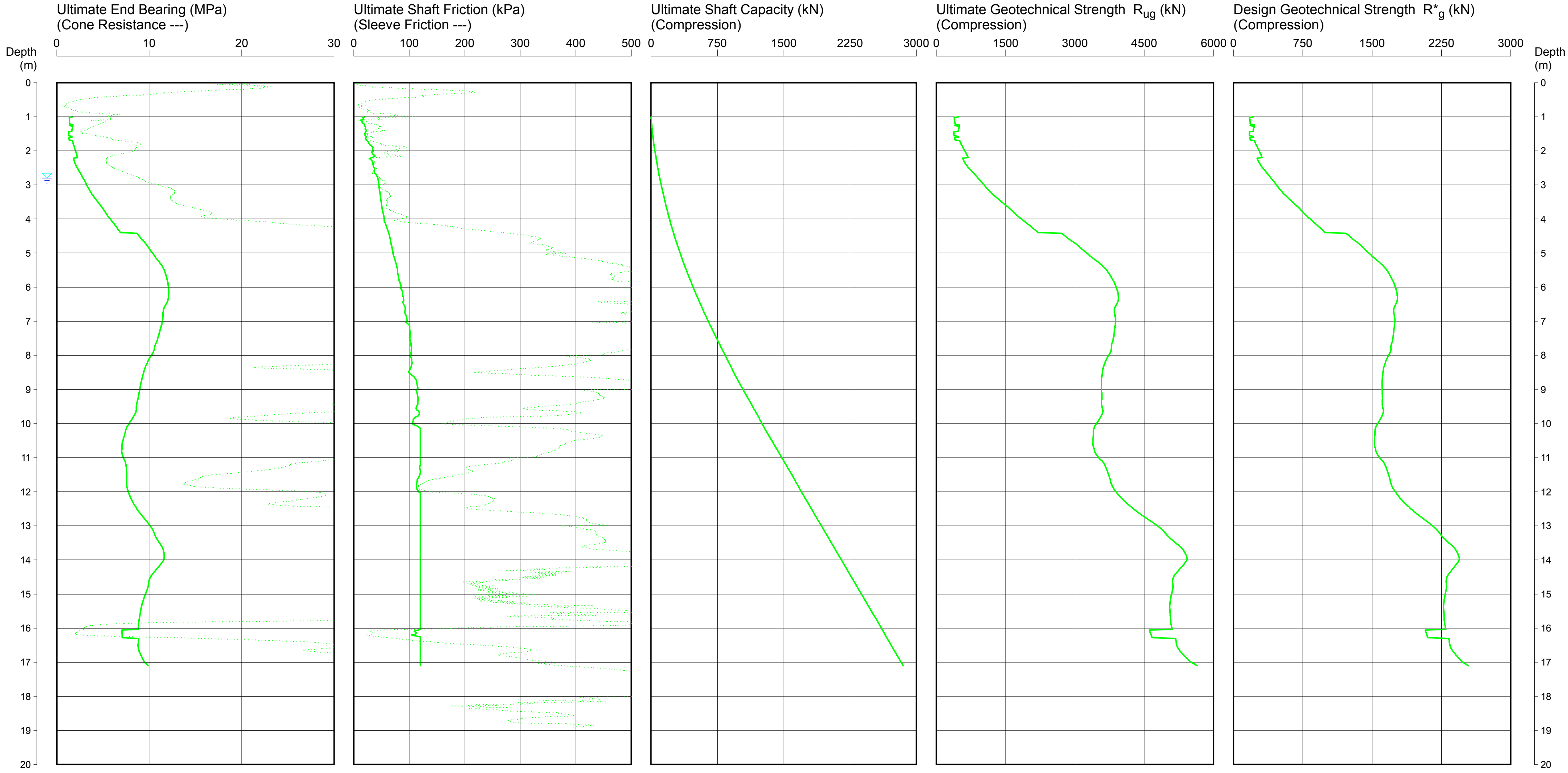
PROJECT: PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 4-18 DONCASTER AVENUE, KENSINGTON

CLIENT: SKY BLUE COMMERCIAL ASSET MANAGERS PTY LTD

CPT102

DATE 9/12/2015
PROJECT No: 73965.02
SURFACE RL: 28.4



DISCLAIMER:
These capacities have been estimated using accepted static theory, and are a guide only. Suitable verification procedures should be adopted (refer to AS2159), and piling contractors should confirm pile suitability and capacities. Structural capacity should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

Water depth after test: 2.80m depth

Coordinates: 336049 6246847

File: P:\73965.05 - KENSINGTON, 4-18 Doncaster Ave, Geo\4.0 Field Work\4.2 Testing\CPT102.CP5
Cone ID: 120620 **Type:** I-CFXY-10

ConePile Version 5.9.1

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