Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Desktop Assessment

Leger Lawn Development
Royal Randwick Racecourse
43 Alison Road, Randwick

Prepared for
Mostyn Copper Group Pty Ltd

Project 86781.00
May 2019

Douglas Partners
Geotechnics / Environment / Groundwater

Document History

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
Author		31 May 2019
Reviewer		31 May 2019

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Douglas Partners
Geotechnics / Environment / Groundwater

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Report on Geotechnical Desktop Assessment
Leger Lawn Development
43 Alison Road, Randwick

1. Introduction

This report presents the results of a geotechnical desktop assessment undertaken by Douglas Partners Pty Ltd (DP) for the proposed Leger Lawn development at Royal Randwick Racecourse. The work was commissioned by Mostyn Copper Group Pty Ltd (MCG) on behalf of Australian Turf Club Limited, and was carried out in consultation with M+G Consulting, structural engineers for the project.

It is understood that the proposed development includes a two storey structure for functions. This structure is to be designed to cater for one or two additional future levels.

This geotechnical desktop assessment provides a geotechnical model for the site, along with preliminary information for excavations and foundations for the proposed structure and recommendations for further geotechnical assessment.

A preliminary site contamination investigation was also undertaken and is being reported separately (DP Project 86781.01.R.001.Rev0)

2. Site Description

The Ledger Lawn is located toward the northern corner of the Royal Randwick Racecourse. The Spectator Precinct is located to the north-east and the recently constructed multi-storey car park is located to the north-west and west. The racecourse proper is located to the south-east and east of the site.

The site is located on relatively flat ground, with surface levels between about RL 30 m to RL 31 m, and it appears that the site has been raised above the racecourse level

The site is currently open grass with a marquee located in the western corner.

The St Leger Stand was previously located in the western corner of the site (where the existing marquee is) and is understood to have been demolished in the 1980's.

3. Geology

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by fine to medium grained, wind-blown dune sand.

4. Previous Investigations

DP has undertaken numerous investigations at Royal Randwick Racecourse, including the Leger Lawn for a previous similar development, the Spectator Precinct (north-east of site) and the Day Stalls / multi-storey car park (west of site). These investigations included numerous cone penetration tests (CPT) to at least 10 m depth. Boreholes and test pits have also been excavated on the site for previous contamination investigations. For the Spectator Precinct development, groundwater levels were monitored for a period of 9 months during construction – this monitoring included one groundwater well located within the Leger Lawn (BH201). Locations of these previous tests are shown on Drawing 1 in Appendix B.

5. Geotechnical Model

A geotechnical model of the site has been prepared based on the results of the previous investigations. A geotechnical cross-section (Section A-A') showing the interpreted subsurface profile between test locations is shown on Drawing 2 in Appendix B. This section shows interpreted geotechnical divisions of underlying soil. The descriptions shown on the cross section are generalised due to the variability in both material type and strength and should be used as a guide only.

The interpreted geotechnical model for the site is also summarised in Table 1. The previous investigations indicate that the site was previously raised with fill up to about 2 m in height. Some CPT locations indicate that the filling may extend to 4 m depth.

Table 1: Interpreted Geotechnical Model

Depth Range (m)	Base of Layer RL (m, AHD)	Layer Description
0 – 2	29	FILLING: sand, generally poorly to moderately compacted
2 – 5	26	SAND: loose to medium dense
5 – 7	24	SAND: medium dense to dense
7 – 10	21	SAND: dense to very dense, with 100-500 mm thick clay bands
10 – 30*	1*	SAND: very dense, with 100-500 mm thick clay bands

NOTE: * Rock level estimated to be at about 30 m depth.

Groundwater was measured in monitoring well BH201 between RL 25.5 m and RL 26.6 m over a 9 month period in 2012. Groundwater was measured in monitoring well GW3 at 5.5 m depth (approximately RL 25.2 m) in March 2009. Groundwater levels may rise by at least 1 m following periods of prolonged rainfall.

6. Proposed Development

A two storey structure to be used for functions is proposed on the site. This structure is to be designed to cater for one or two additional future levels. Shallow excavation is expected for foundation preparation and the lift core.

7. Comments

7.1 Excavation Conditions

Excavations will be carried out through sandy filling and 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 in the filling should be made (i.e. from the St Leger Stand demolition).

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

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 roadbase or crushed concrete (or similar), which may subsequently be used as sub-base.

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). Reference should be made to the preliminary site contamination investigation (Ref. 8781.01.R.001) by DP for information on the contamination status of the soil.

7.2 Excavation Support

During the bulk excavation phase, it is recommended that temporary batter slopes above the groundwater table do not exceed 1.5:1 (H:V). Batter slopes should be reduced to 3:1 adjacent to existing buildings or any other movement sensitive structures or services and these should also be subject to geotechnical review.

7.3 Subgrade Preparation

Sandy filling of variable compaction underlies the site to depths of about 1 m to 2 m. It is recommended that this material be removed and recompacted to ensure uniformity, if it is required to support slabs or footings. This will be particularly important for raft slab support.

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

- following stripping of existing uncontrolled fill and 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 or maximum depth of 0.5 m and replaced with engineered filling. Some allowance for removal and reworking of unsuitable material should be made. The extent of remedial work required will depend on the footing/pavement loads and settlement tolerances. Further geotechnical testing and review should be sought at the time of proof rolling;

- 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 filling and 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 its contamination status and should be assessed by an environmental consultant. 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.

Large earthworks plant / piling rigs may presumably be required to operate on the site and may require the construction of a working platform 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. Specialist track mats may also be considered as an alternative to granular platforms. The thickness of the platform will need to be assessed once specific details of the plant loads are known.

7.4 Foundations

7.4.1 Raft Slabs

It is understood that a raft foundation is proposed for the development. Distributed working loads of 15 kPa per floor have been provided by the structural engineer (for the two proposed floors and two additional floors).

It is assumed and recommended that the existing sandy filling is reworked and compacted in accordance with Section 7.3. If the filling is to be left in place there is a higher risk of differential settlement to the raft slab and the raft thickness would need to be designed to accommodate this.

As a guide, for raft slab foundations, preliminary settlement analyses have been carried out based on assumed distributed slab loads over:

- a broadly loaded area (20 m by 20 m);
- a deepened beam (3 m by 20 m); and
- a lift core (4 m by 4 m).

The simplified assumed loadings and results of the analysis are shown in Table 2, together with the estimated modulus of subgrade reaction (k) values. It is noted that the k value (which is not strictly a soil parameter) is very dependent on the size of the loaded area and the rigidity of the raft system.

Table 2: Modulus of Subgrade Reaction Values and Estimated Settlements for Raft Slabs

Assumed Applied Area (m x m)	Modulus of Subgrade Reaction, k (kPa / mm)	Assumed Applied Pressure (kPa)	Estimated Settlement (mm)
20 x 20 (broadly loaded area)	2.5 – 3	30 60	10 – 15 20 – 25
3 x 20 (deepened beam)	4 – 5	200 400	40 – 50 80 – 100
4 x 4 (lift core)	4.5 – 5.5	200	35 – 45

If the existing filling is not recompacted as per Section 7.3, as a guide the modulus of subgrade reaction value for the broadly loaded area would reduce to about 2.0 – 2.5 kPa/mm.

Soil design parameters used to estimate the settlements and modulus of subgrade reaction values are provided in Table 3.

Table 3: Geotechnical Parameters for Raft Slab Design

Depth Range (m)	Base of Layer RL (m, AHD)	Layer Description	Young's Modulus, E (MPa)	Unit Weight, γ (kN/m^3)	Friction Angle, ϕ (degrees)	Poisson's ratio
0 – 2	29	FILLING	15 – 25	18 – 20	30 – 36	0.3
2 – 5	26	SAND: l-md	12 – 15	18	32	0.3
5 – 7	24	SAND: md-d	50	19	34	0.3
7 – 10	21	SAND: d-vd	100	20	36	0.3
10 – 15	16	SAND vd	150	20	40	0.3
15 – 30	1	SAND: d-vd	100	20	36	0.3

Construction of the raft slabs should incorporate subgrade preparation as outlined in Section 7.3. 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.

Detailed geotechnical review and analysis of bearing pressures and settlements will be required once more specific details of the founding levels, column layout and slab loadings have been confirmed.

7.4.2 Shallow Foundations

Pad and strip footings may be considered for relatively light loads but are unlikely to be suitable and may require large footing sizes. The bearing capacity of sand foundations depends on the size and depth/embedment of the footing, the density of the sand and depth to groundwater. As a guide, it is expected that pad footings with a plan area of 2 m x 2 m and embedment of 0.5 m, founded on loose sand or engineered fill, could be designed for an allowable end bearing pressure of 250 kPa. This would be subject to further geotechnical review and analysis of bearing capacity and settlements once details of footing loads are confirmed.

7.4.3 Pile Foundations

The alternative to shallow foundations is to support the structural loads on piles founded within at least medium dense to dense sand which is expected below about RL26.0 m. Higher capacities, if required, could be achieved within the consistent dense to very dense sand below about RL24.0 m.

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, more efficiently using the in-situ capacity of the soil.

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.

Steel screw piles may be considered for relatively light loads (i.e. possibly for the Ledger Lawn development) 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, a preliminary estimate of the geotechnical capacity of concrete-injected piles (0.9 m diameter) is provided in Appendix C, at locations CPT307, CPT308 and CPT309. The pile capacity estimate is calculated using ConePile which is an in-house DP pile analysis and design program. The pile capacity estimate indicates the assessed ultimate end bearing and shaft friction values with depth together with an ultimate geotechnical ($R_{d,ug}$) and design strength ($R_{d,g}$) for the piles at varying depths. The design geotechnical strength is based on an assumed geotechnical strength reduction factor (ϕ_g) of 0.45. This ϕ_g value, however, should be determined by the designer in accordance with AS2159 (November 2009). The selection of ϕ_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.

7.5 Groundwater

Previous monitoring by DP has indicated groundwater levels on the site of between RL25.0 m to RL26.0 m. In the absence of long term monitoring of groundwater levels, it is suggested that a potential groundwater level of RL28.0 m should 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.

7.6 Earthquake Site Classification

In accordance with AS1170 –2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia" a hazard factor (Z) of 0.08 and a site subsoil Class C_e are considered to be appropriate for the site.

7.7 Additional Geotechnical Investigation

To refine the raft slab design, an additional five to six CPTs could be undertaken across the site to depths of at least 10 m. In particular the additional CPTs would allow for further definition of the upper profile in the sandy filling and loose sands, which will greatly impact the design stiffness of the raft slab.

8. Limitations

Douglas Partners (DP) has prepared this report for the proposed Ledger Lawn development at Royal Randwick Racecourse in accordance with the Consultancy Services Agreement between Australian Turf Club Limited (ATC) and Douglas Partners Pty Ltd (DP). This report is provided for the exclusive use of ATC 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 only at the specific sampling 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 anthropogenic influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during previous investigations on the site. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling locations.

This report must be read in conjunction with all of the attached notes 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 given 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 instruction 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.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

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

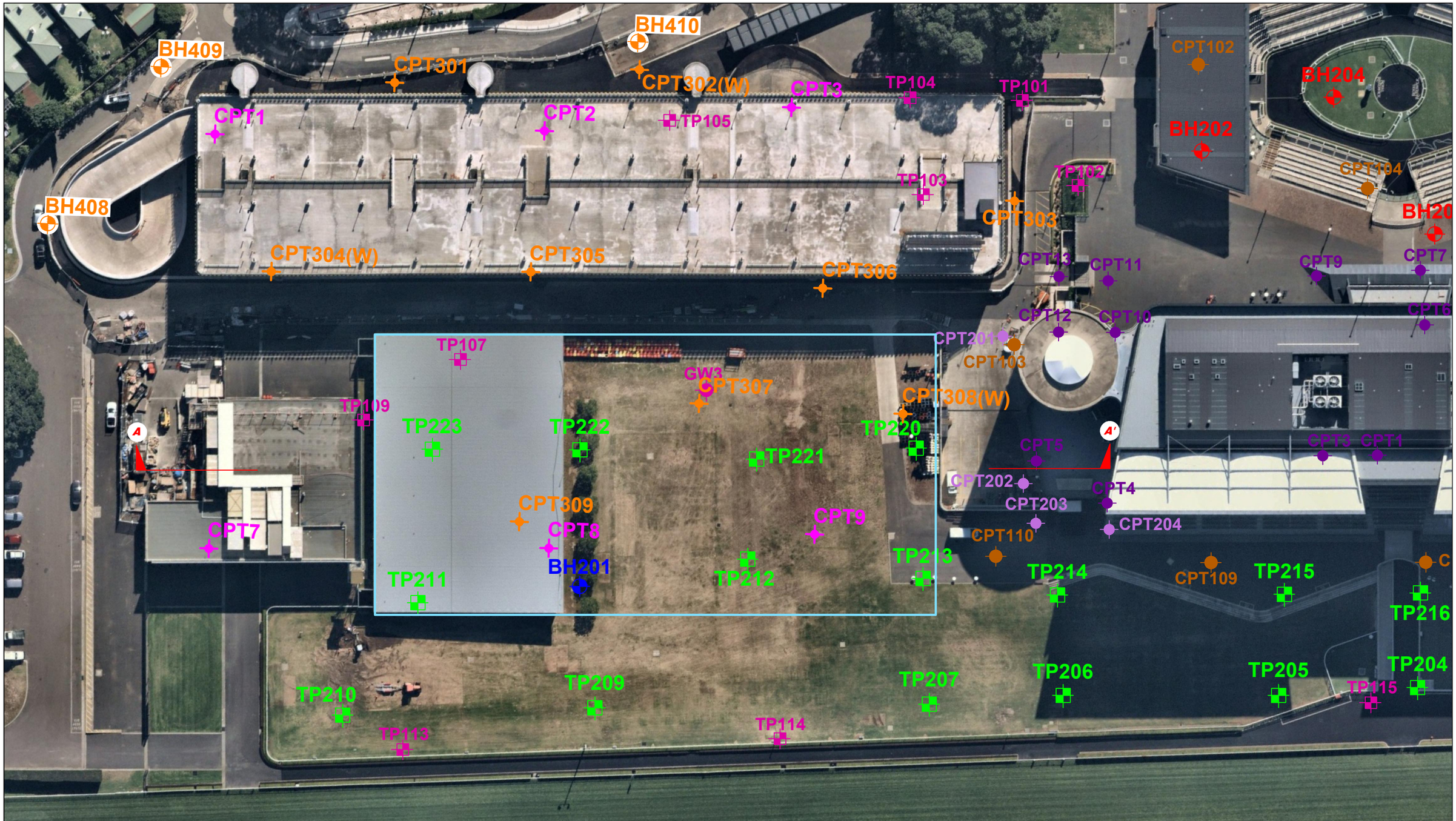
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

Appendix B

Drawings



LEGEND

A A' Geotechnical Cross Section A-A'

Leger Lawn Site Outline

- Previous borehole 2015
- Previous CPT 2015
- Previous borehole 2012
- Previous test pit 2012
- Previous monitoring well 2012
- Previous test pit - March 2010
- Previous monitoring well - March 2010
- Previous test pit - Sept 2010
- Previous borehole - Sept 2010
- Previous CPT 2008
- Previous CPT - Aug 2010
- Previous bore hole - Aug 2010
- Previous CPT - 1989
- Previous CPT - 1988
- Previous CPT - 1986
- Previous CPT - 1967

0 5 10 15 20 30 40 50 75m

1:750 @ A3

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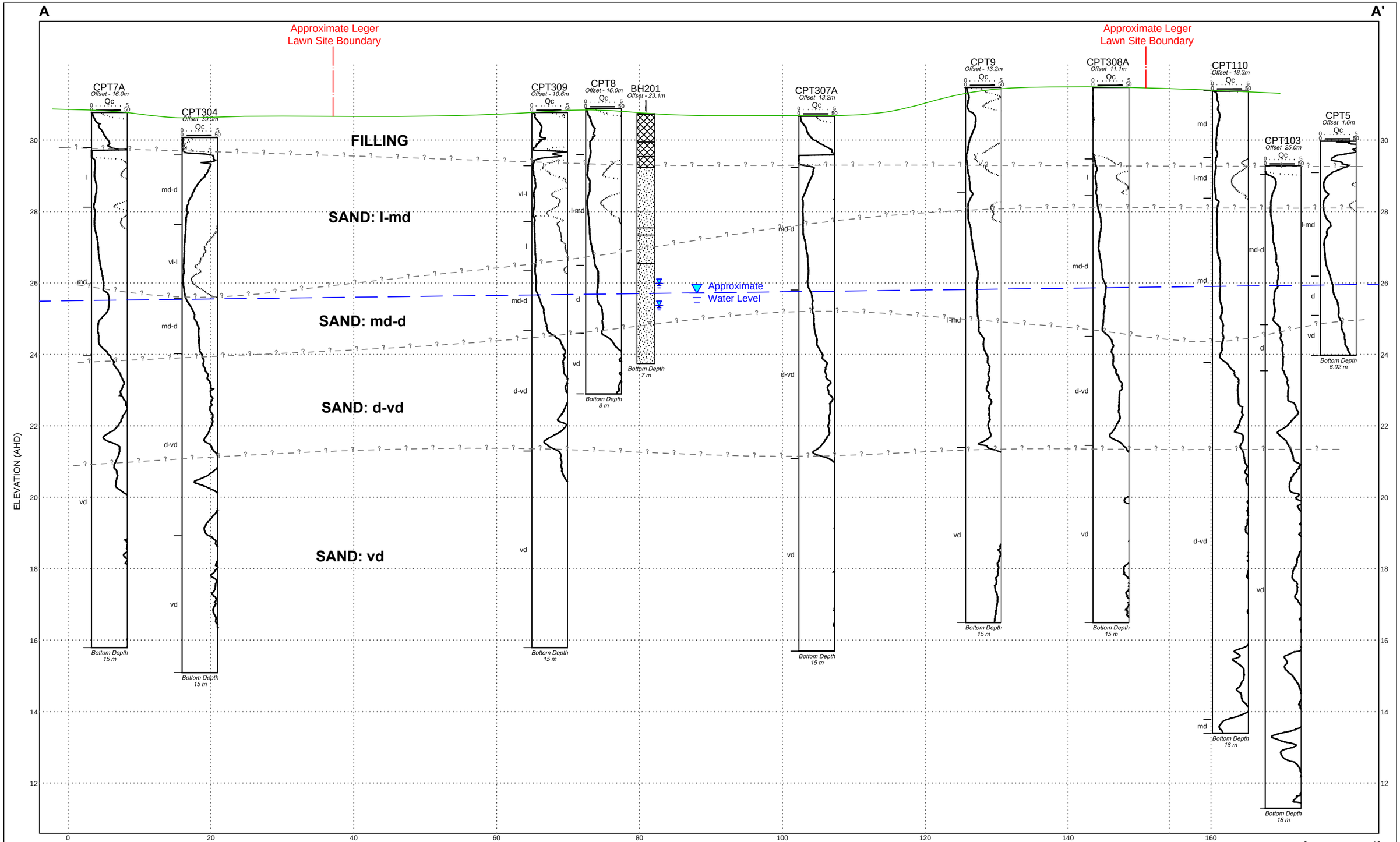
CLIENT: Australian Turf Club		TITLE: **Previous Test Location Plan and Site Outline**	PROJECT No: 86781.00
OFFICE: Sydney	DRAWN BY: LJH	**Leger Lawn Development**	DRAWING No: 1
SCALE: 1:750 @ A3	DATE: 20.05.2019	**Royal Randwick Racecourse**	REVISION: 0



NOTE:

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

SAND DENSITY
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
Water level
Inferred geological boundary

0 10
Horizontal Scale (metres)

Vertical Exaggeration = 5.0

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CLIENT: Australian Turf Club Limited		TITLE: Interpreted Geotechnical Cross Section A-A'	PROJECT No: 86781.00
OFFICE: Sydney	DRAWN BY: LJH	Leger Lawn Development	DRAWING No: 2
SCALE: 1:500 (H) 1:100 (V) @ A3	DATE: 20.05.2019	Royal Randwick Racecourse	REVISION: 0

Appendix C

Pile Capacity Estimates

PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.90
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

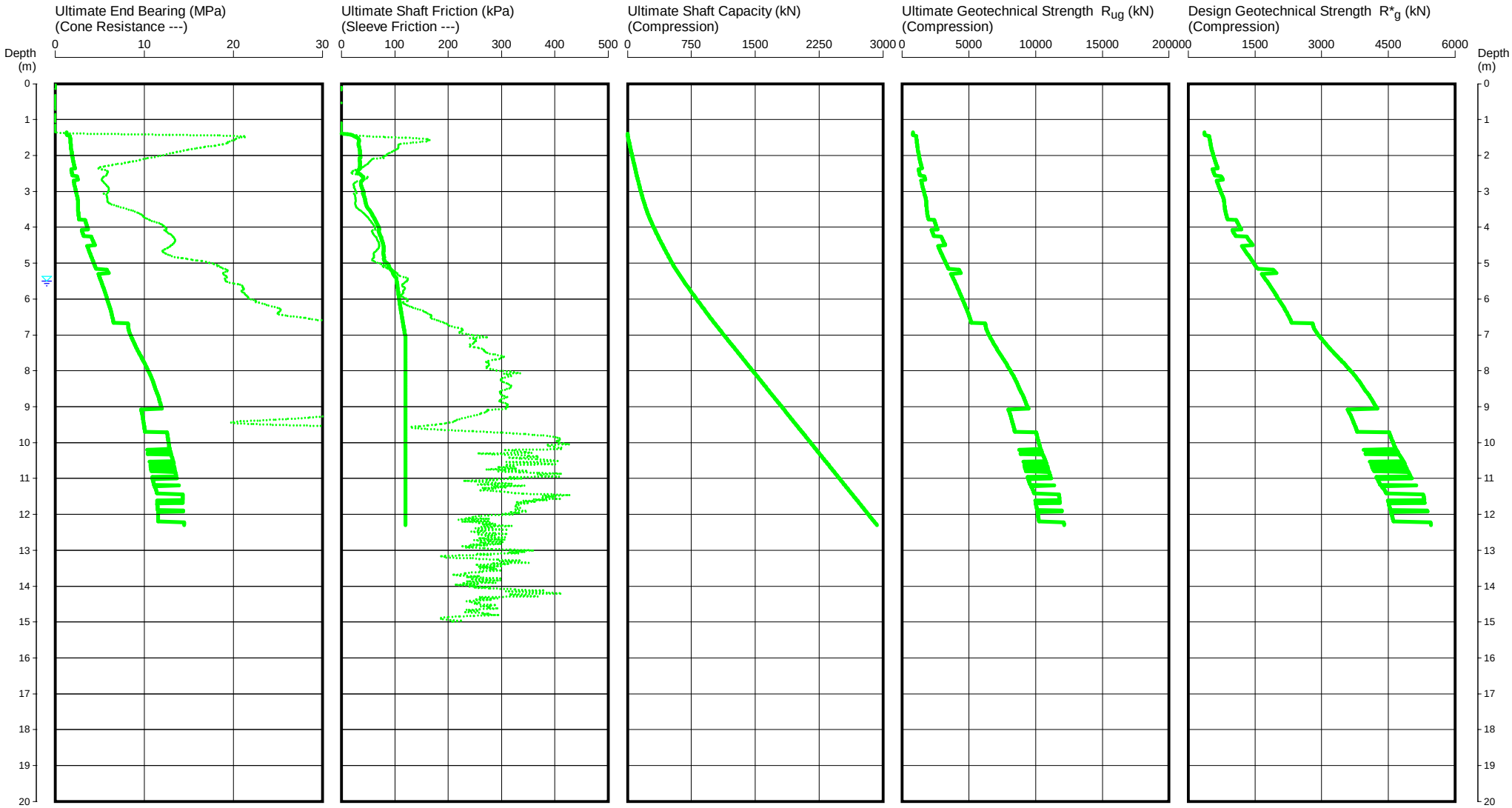
PROJECT: PROPOSED MULTI-DECK CARPARK

LOCATION: ROYAL RANDWICK RACECOURSE - 43 ALISON ROAD, RANDWICK

CLIENT: AUSTRALIAN TURF CLUB LIMITED

CPT7A CONTD

DATE 17/11/2015
PROJECT No: 85201
SURFACE RL:



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: 5.50m depth

Coordinates: 336141 6246600

File: P:\85201.00 - RANDWICK, Proposed Multi-deck Car Park\4.0 Field Work\85201 RANDWICK cpts\7A\CPT7A CONTD.cpt
Cone ID: 120631 **Type:** I-CFXY-10

ConePile Version 5.9.1

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PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.90
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

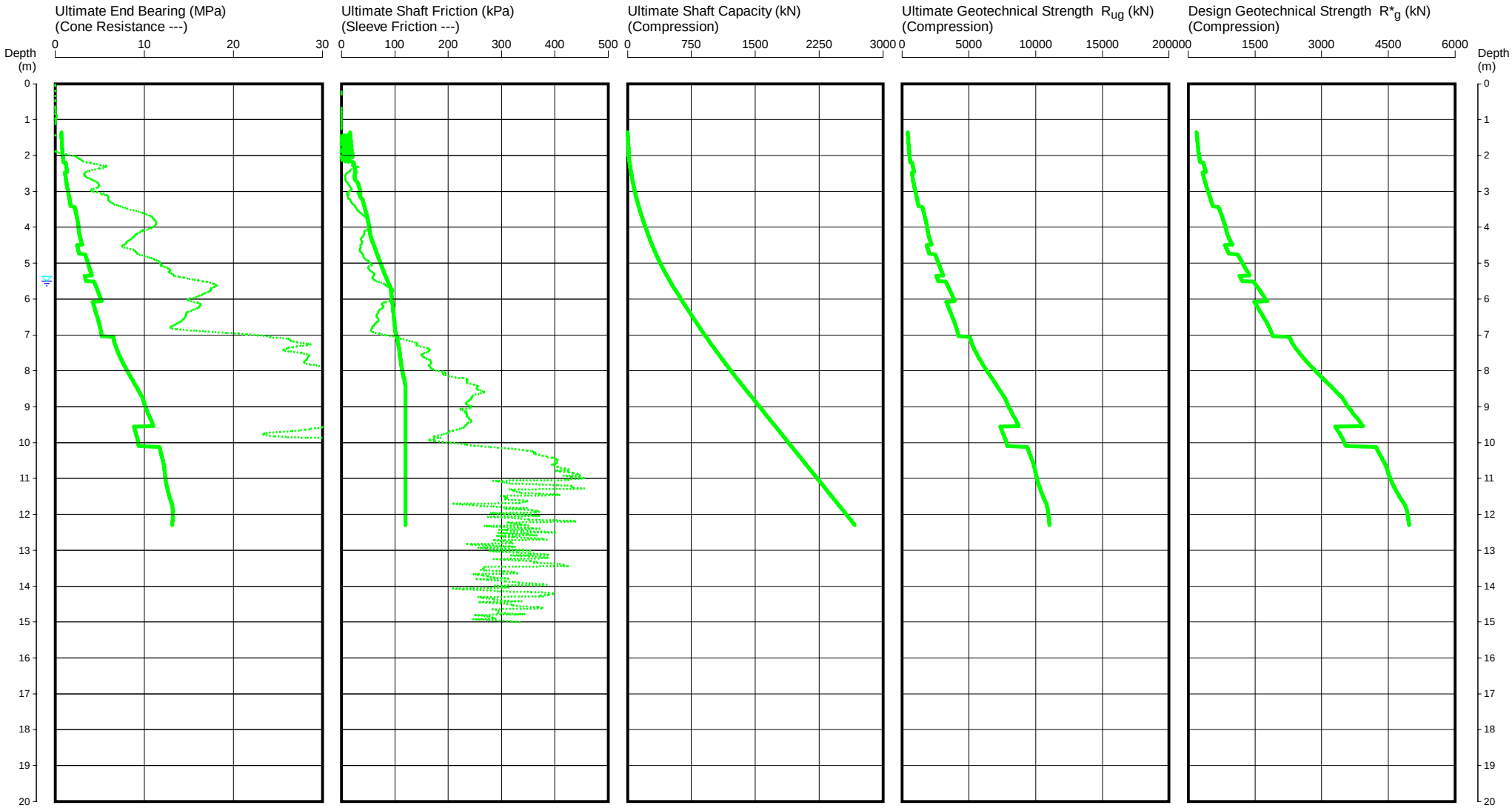
PROJECT: PROPOSED MULTI-DECK CARPARK

LOCATION: ROYAL RANDWICK RACECOURSE - 43 ALISON ROAD, RANDWICK

CLIENT: AUSTRALIAN TURF CLUB LIMITED

CPT308A

DATE 17/11/2015
PROJECT No: 85201
SURFACE RL: 31.5



DISCLAIMER:
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Water depth after test: 5.50m depth

Coordinates: 336173 6246626

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Cone ID: 120631 **Type:** I-CFXY-10

ConePile Version 5.9.1

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PILE CAPACITY ESTIMATE

PILE TYPE: Grout-Injected
PILE SHAPE: Round
PILE SIZE: Diameter = 0.90
STRENGTH REDUCTION FACTOR $Ø_g$: 0.45
CALCULATION METHOD: Douglas Method

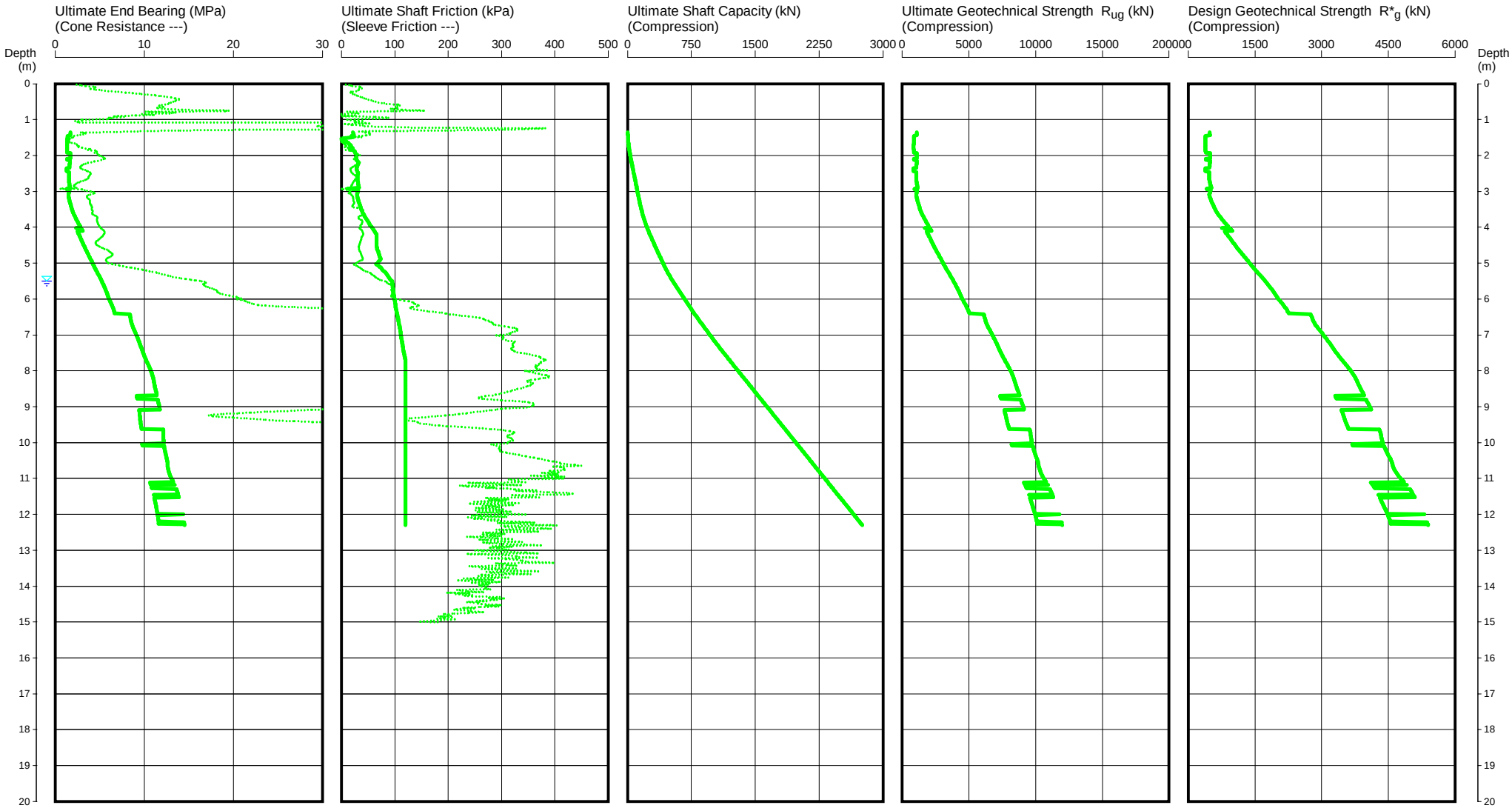
PROJECT: PROPOSED MULTI-DECK CARPARK

LOCATION: ROYAL RANDWICK RACECOURSE - 43 ALISON ROAD, RANDWICK

CLIENT: AUSTRALIAN TURF CLUB LIMITED

CPT309

DATE 17/11/2015
PROJECT No: 85201
SURFACE RL: 30.8



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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: 5.50m depth

Coordinates: 336130 6246558

File: P:\85201.00 - RANDWICK, Proposed Multi-deck Car Park\4.0 Field Work\85201 RANDWICK cpts\CPT309.CP5
Cone ID: 120631 **Type:** I-CFXY-10

ConePile Version 5.9.1

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