REPORT TO

CAMPBELL D TAYLOR & SARAH J CURTIS

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENCE

AT

18 OLPHERT AVENUE, VAUCLUSE, NSW

Date: 7 February 2020
Ref: 32829SCrptRev2

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Table of Contents

ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 18 Olphert Avenue, Vaucluse, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Louise St John Kennedy Designer, and was carried out in accordance with our proposal dated 21 August 2019 (Ref: P50149S).

As shown on the supplied architectural drawings by Louise St John Kennedy Designer (Project Name. Olphert Avenue Residence, Drawing Nos. DA.06 to DA.12, all dated 17 October 2019) and subsequent emails we understand that the proposed development will comprise the following:

- Demolition of the existing house and surrounding minor structures.
- Construction of a three-level residence, which includes a basement in the southern and central portions.
- The basement has a finished floor level of RL54.48m and will extend to within 2m of the eastern and western site boundaries and will be setback about 6m from the southern site boundary with excavations up to a maximum depth of about 4.6m.
- A swimming pool is proposed on the northern side of the residence and the pool coping and surrounding lawn will have a finished surface level at RL54m. At least the northern side of the swimming pool is likely to be a suspended structure as the existing ground surface level in this area is lower than the base of the proposed pool. The proposed lawn level which surrounds the pool is approximately 3m above the existing surface level along the northern side of the site.
- Consideration is being given to infiltration trenches and soak wells positioned within the rear lawn of the development. A large rainwater tank is proposed to be constructed on the new lower ground floor level at the southern end of the house.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, earthworks, retention, footings and hydrogeological issues.

2 INVESTIGATION PROCEDURE

Due to the limited site access the investigation was carried out using portable hand operated equipment. Boreholes BH4, BH8 and BH9 were drilled using a hand auger to refusal at depths ranging from 0.6m to 3m. Dynamic Cone Penetration (DCP) tests were carried out adjacent to the boreholes and at six additional locations (DCP1 to DCP3 and DCP5 to DCP7) to depths ranging from 0.7m to 5m. The purpose of the DCP tests was to assess the compaction of the fill and the relative density of the natural sands and to probe down to indicate whether bedrock was present.

The test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs and DCP test results, were estimated by interpolation between spot levels and contours shown on the supplied survey plan by Hill and Blume Pty Ltd (Ref:61199001A, dated 31/10/2019). The datum of the levels is Australian Height Datum (AHD).

A slotted PVC standpipe was installed in BH9 to allow the completion of a constant head infiltration test. Water from a garden hose was left running into the boreholes for 20 minutes to allow initial wetting of the soils around the test location. Following this time, the flow of water through the hose was adjusted to maintain a constant head of water in the borehole, and then the flowrate was estimated by measuring the time to fill a calibrated storage container with the hose. The borehole geometry and flow rate were then used to estimate the coefficient of permeability (k) for the soil.

Groundwater observations were made during and on completion of drilling of each borehole. No longer term groundwater monitoring was carried out.

Our Senior Engineering Geologist, Mr Thomas Clent, was on site full time during the fieldwork to make observations of the site and its surroundings, set out the test locations, nominate the sampling and testing, prepare the borehole logs and record the DCP test results. The borehole logs and DCP test results are attached to this report, together with our Report Explanation Notes which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located partway down a moderate to steep north facing hillside, which grades at overall slopes of 10° to 15°. Parsley Bay gully is located about 100m north-west of the subject site.

At the time of the fieldwork, the site contained a two-storey residence which was located within the southern (up-slope) portion of the site. The sub-floor walls comprised sandstone blocks whilst rendered bricks formed the main level of the house. The house appeared to be in poor external condition with areas of stepped cracking observed within the brick work on the western walls of the house. On the eastern and western sides of the house are timber framed planter beds and a some small to medium sized trees are located on the lawn to the rear of the house.

Along the southern boundary of the site is a concrete retaining wall about 1.2m high, which retains a grass verge along Olphert Avenue. The wall appeared in poor condition with an outward lean and cracking through-out. Above the concrete retaining wall is a brick fence about 0.8m in height.

To the north of the site is are 2 one to two storey residential buildings, which have surface levels approximately between 1.2m and 1.6m lower than the subject site. A sandstone block retaining wall is present on this boundary and facilitates the difference in surface levels.

To the east of the site is a three-storey rendered house, occupying the southern portion of the property. Where the adjoining house is located the ground surface across the common boundary was is about 2.7m to 1m lower than the subject site. However, the swimming pool at the northern end of the property is about 1.4m higher than the subject site.

To the west of the site is a three-storey rendered residence which is located within the central to southern portions of the property. The neighbouring building is set back about 3m from the common boundary and the ground surface level steps down the hillside similarly to the subject site. The northern end of the neighbouring property contains a tennis court which is approximately 2.6m higher than the subject site lawn level. A series of low height brick/rubble block walls exist along this part of the boundary.

3.2 Subsurface Conditions

Fill was encountered in all boreholes, to depths ranging from 0.3m to 0.6m. The fill comprised silty sand with traces of root fibres and inclusions of sandstone gravel. Based on DCP test results the fill was assessed to be poorly compacted.

In BH8 and BH9 natural silty sand was encountered below the fill and based on DCP test results was initially of very loose relative density becoming very loose to loose below about 1m depth.

Assuming that the DCP tests were carried out within natural sands, the sands elsewhere were also of very loose relative density, becoming medium dense at depths ranging from 1.1m (DCP1 and DCP3) to 3.1m (DCP8). However, since these tests do not provide sample recovery the presence of natural sandy soils can only be anticipated. Discounting DCP4 which refused at 0.7m depth, refusal of the DCP tests occurred at depths ranging from 2.3m (DCP3) to 5m (DCP1) and refusal may have occurred within dense sands or possibly on the surface of the sandstone bedrock. It is possible that refusal has occurred on hard layers such as cemented sand layers (coffee rock) or obstructions, such as floaters in the fill. The latter is likely to be the case at DCP4.

No groundwater seepage was encountered within the boreholes during or on completion of drilling.

Reference should be made to the boreholes logs and DCP test results for details of the subsurface conditions encountered.

3.3 Infiltration Test Result

The result of the constant head borehole infiltration test indicated an approximate mass coefficient of permeability of 7.3×10^{-4} m/sec. This result is of the order expected for a silty sand profile.

4 COMMENTS AND RECOMMENDATIONS

4.1 Inferred Subsurface Profile

Due to limitations of the portable equipment used for this investigation, the subsurface profile below the depth of refusal must be inferred. The results that could be obtained indicate that the site is underlain by granular fill but shallow depths, to variable depths covering natural sand that is initially of very loose to loose relative density becoming medium dense with depth. Medium dense sands were encountered between RL51.2m (DCP6) to RL57.9m (DCP1). Within the upper portion of the site DCP1, DCP2, DCP7 and DCP8 have been inferred to have refused within dense sands. Within the lower portion of the site DCP4, DCP5 and DCP9 have been inferred to have refused on sandstone bedrock, but refusal may have occurred on other hard layers either in the fill or cemented sands or sandstone bands above the continuous sandstone bedrock. Where variations from the profile summarised above would have significant impact on the design and construction, it will be necessary to carry out boreholes using a drilling rig for which access will have to be provided.

4.2 Excavation for Proposed Basement Level and Lower Floor Level

We understand that excavations to a maximum depth of about 4.5m will be required to form the proposed basement level, if the southern extension for the water treatment plant is included or about 3m depth if not. Excavation to such depths will encounter some sandy fill but for the most part natural silty sand which should be achievable using conventional excavation equipment such as buckets of hydraulic excavators.

Although no indication of groundwater was noted in the investigation, the methods used do not allow a reliable assessment. We therefore recommend that when rig drilled boreholes are completed that ground water monitoring wells are installed to confirm that groundwater is not present.

Prior to the start of excavation, and preferably prior to demolition, dilapidation surveys should be completed on the adjoining properties to the east and west.

The dilapidation surveys should comprise detailed inspections of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of these reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

The site is underlain by poorly compacted fill or very loose to loose sands and these may extend below the adjoining properties. Care must be taken during demolition and excavation not to cause vibrations within the soils as that can lead to settlement that may affect the neighbouring buildings if they are supported on shallow footings. During demolition large pieces should no be allowed to fall as this could cause vibration and concrete pieces should not be broken up using hydraulic hammers. Saws should be used to cut up concrete pieces and then these removed using an excavator bucket. During demolition and construction, the sudden stop/start movement of tracked plant should be avoided as this can cause vibrations that may induce settlement.

4.3 Proposed Swimming Pool and Lawn

The proposed swimming pool and surrounding lawn will be set back about 2m from the western site boundary and within 1m of the northern and eastern site boundaries. The swimming pool shell will be setback 1.8m from the eastern boundary, 4.2m from the western boundary and 4.3m from the northern boundary. Minimal excavation will be required on the southern side of the swimming pool and where the swimming pool is above the existing ground surface on the northern side it will likely be suspended. The new lawn which is to surround the pool has a finished surface level of RL54m approximately 4m above the existing ground surface level along the northern side of the site. Therefore, the walls of the swimming pool will have to be designed as retaining walls. Also, new retaining walls will be required on the eastern, northern and western sides of the proposed lawn to contain the fill platform. All retaining walls should be designed based on the parameters given in Section 4.5 below.

Due to the sloping nature of the site the pool shell and surround should be supported on piles founded within the underlying dense sand or inferred sandstone bedrock. The presence of a large sewer beneath the proposed footprint also needs to be taken into consideration during design so as not to adversely impact the sewer. Sydney Water are likely to require the sewer be concrete encased and the pool supported on piles founded below the zone of influence of the sewer.

The existing northern boundary retaining wall should be entirely removed and replaced during construction of the proposed swimming pool and surrounding lawn level. Return walls the eastern and western boundaries will also be required.

4.4 Earthworks

It is anticipated that fill will be placed to form the proposed lawn which surrounds the swimming pool (soft landscaping) to a maximum height of about 3m. Where hard landscaping such as walls, paths and pavements are proposed we recommend that they be uniformly supported piles as it is very difficult to successfully complete earthworks in small areas to the standard necessary to avoid settlement adversely affecting the performance of the structures.

Where fill is placed in soft landscaping areas it should comprise either a sandy material or ripped sandstone. Ripped sandstone must be well graded which will require processing wither on or off site. Fill should be placed in layers of approximately 0.1m thickness and compacted using either a vibratory plate compactor (for sandy soils) or a wacker packer (for ripped sandstone). The fill should be free from all organic or otherwise deleterious materials and should have particle sizes of no greater than 35mm. As the fill will not be placed as engineered or controlled fill it must be accepted that some differential settlement of the fill may occur. This could require periodic remediation in the form of topping up and re-levelling.

Should possible differential settlement of the fill be considered unacceptable, the fill must be placed as engineered or controlled fill. Suitable materials for engineered fill are as described above and must similarly be placed in roughly 0.1m layers but must be compacted to a minimum density index of 70% for the sandy soils and between 95% and 102% of Standard Maximum Dry Density (SMDD) and within +/- 2% of Standard Optimum Moisture Content (SOMC) for the ripped sandstone. A minimum of Level 2 earthworks control must be completed with density testing completed at a frequency of at least 1 test/50m^2/2 layers or three tests per visit, whichever is the greater. Reference should also be made to AS3798-2007 and the recommendations contained within followed.

4.5 Retention

Any temporary batters, if space permits, should be no steeper than 1 Vertical in 1.5 Horizontal (1V:1.5H) within the sandy soils. Such batters should remain stable in the short term, provided all surcharge loads, including construction loads are kept well clear of the crest of the batters. All stormwater runoff should be directed away from all batters to reduce erosion.

Considering the depth and proximity of the proposed basement excavation to the eastern, southern and western sides of the site, space may allow for the formation of temporary batters along these boundaries. However, we advise against the use of large slope batters in sandy material and excessive excavation within the site. Therefore, a retention system will need to be installed prior to the start of excavation along the southern and part way along the eastern and western sides of the basement. The shoring system should be relatively 'closed' or 'tight' system to avoid loss of the expected sandy materials that are to be retained, and may comprise contiguous piled walls. Piles would need to comprise auger, grout injected (CFA) piles or hand augered piles due to the sandy soils and will need to be embedded sufficiently below the proposed excavation level to achieve stability.

Temporary lateral restraint at the top of shoring walls should be provided by installation of a capping beam and bracing with the permanent lateral restraint provided by the floor slabs. The base of the shoring walls should be embedded sufficiently to ensure stability. Where the pile toes will extend below the refusal depth of the DCP tests, it is important that rig-drilled boreholes be carried out to confirm that the presence of bedrock will not affect pile installation.

Retaining walls maybe designed based on a triangular earth pressure distribution. Any walls that support structures should be designed based on an 'at rest' earth pressure coefficient, K_0, of 0.6 and a bulk unit weight of 20kN/m^3, in order to limit deflections. Passive resistance of the piled walls may be estimated based on a passive earth pressure coefficient, K_P, of 3.0, but a factor of safety of 2 should be used to limit deflections. The passive resistance should be ignored for a depth of at least 0.3m below the base of the existing, including local excavations for footings or services, to allow for disturbance.

The above coefficients assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the walls.

Any free standing walls constructed at the base of batters where some resulting ground movements are tolerable, such as within landscaped areas, may be designed based on an active earth pressure coefficient, K_a, of 0.33.

Considering the proposed water storage tank which is to be positioned on the lower ground floor level, the retaining wall between the lower ground floor level and basement (if the southern basement extensions for the water treatment plant is constructed) will need to be designed to accommodate the surcharge load from the water tank structure or the water tank should be supported on piles founded below the zone of influence of the basement retaining walls.

4.6 Footings

For the basement excavation, we expect that medium dense sand will be exposed at bulk excavation level for the majority of the basement, but in some areas the sand may only be of loose relative density. Loose to medium dense sand will likely be exposed at bulk excavation level for the proposed lower floor level. However, portions of the lower floor level structure extend past the basement footprint and the use of piles will be required in these areas so that surcharge loads are not placed on the basement walls and footings and floor slabs are not founded on backfill. Since piles are required for that portion of the structure we recommend that the entire structure be supported on piles to provide uniform support and reduce the risk of differential settlements, unless very detailed analysis of differential settlements for different sections of the structure is completed.

Suitable pile type would include machine drilled CFA piles, steel screw piles or hand auger drilled piles.

The allowable bearing pressure for the design of piles in sands is dependent on the relative density of the sand, the pile diameter, the pile embedment depth and the groundwater level and the designer should take all of these factors into account. As a guide, piles with a diameter of at least 0.3m, founded within medium dense sand, with an embedment depth of at least 2m below the surrounding ground surface may be designed based on an allowable bearing pressure of 600kPa. For piles drilled from the lower floor level, the embedment depth will be based on the zone of influence of the adjacent basement level and not the ground surface at the top of the basement walls.

Since the relative density of the sands cannot be confirmed during the installation of steel screw piles or CFA piles, we recommend that piles initially be installed close to borehole locations so that the operator can calibrate themselves to known conditions before installing piles at other locations. Where the zone of influence of piles extend below the termination depths of the DCP tests, it will be necessary to complete boreholes with a drilling rig where access is available.

Where the swimming pool is proposed close to the northern boundary, the existing ground surface level steps down to the north. The proposed finished floor levels of the swimming pool and pool coping are above the existing ground surface level and should be entirely supported piles founded within the underlying inferred sandstone bedrock. Footings founded within sandstone bedrock may be designed based on an allowable bearing pressure (ABP) of 700kPa. We recommend that boreholes be drilled with a rig in these areas where access is available to confirm the presence and quality of any bedrock present.

4.7 Infiltration Pits

The investigation has indicated that the subsurface conditions below the rear lawn comprise sandy fill and natural sand to between 0.6m and 0.8m depth with DCP refusal depths ranging from 2.3m to 2.7m on inferred sandstone bedrock or dense sand. The constant head borehole infiltration tests have indicated the soils to have a relatively high permeability. The test was undertaken in the natural silty sand and has indicated the site to be geotechnically suitable for on-site infiltration of stormwater. However, the capacity of the infiltration system will be limited as immediately downslope will be a new swimming pool and beyond that a step down to the neighbouring property, water which does migrate downslope to the northern boundary by flowing below the pool would discharge to neighbouring property and the situation should be considered holistically.

4.8 Further Geotechnical Input

We recommend that the following further work be completed, as described above.

- The drilling of rig boreholes so that the depth to and quality of the underlying bedrock can be determined. Installation of groundwater monitoring wells within the rig boreholes to confirm the presence of any groundwater.
- Geotechnical and Hydrogeological Monitoring Plan
- If groundwater is present in the boreholes then seepage analysis maybe required.
- Review of structural drawings to confirm geotechnical principals and recommendations have been properly addressed.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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BOREHOLE LOG

Borehole No. 4

1/1

Client: LOUISE ST JOHN KENNEDY DESIGNER

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 18 OLPHERT AVENUE, VAUCLUSE, NSW

Job No.: 32829SC **Method:** HAND AUGER **R.L. Surface:** ≈ 51.5m

Date: 20/11/2019 **Datum:** AHD

Plant Type: **Logged/Checked by:** T.C./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0 – 0.5			FILL: Silty sand, fine to coarse grained, dark brown and grey, trace of fine to coarse grained sandstone gravel, and brick fragments.	D			GRASS COVER APPEARS POORLY COMPACTED
						0.6			END OF BOREHOLE AT 0.6m				HAND AUGER REFUSAL DUE TO BOREHOLE COLLAPSE
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No.
8

Client: LOUISE ST JOHN KENNEDY DESIGNER

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 18 OLPHERT AVENUE, VAUCLUSE, NSW

Job No.: 32829SC **Method:** HAND AUGER **R.L. Surface:** ≈ 58.5m

Date: 20/11/2019 **Datum:** AHD

Plant Type: **Logged/Checked by:** O.E./T.C.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown and dark grey, trace of tile fragments and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
						0.5		SM	Silty SAND: fine to medium grained, grey.	M	VL		AEOLIAN
						1			Silty SAND: fine to medium grained, orange brown and yellow brown.	M	VL-L		
						1.5							
						2							
						2.5							
						3			END OF BOREHOLE AT 3.0m				
						3.5							

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BOREHOLE LOG

Borehole No. 9

Client: LOUISE ST JOHN KENNEDY DESIGNER

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 18 OLPHERT AVENUE, VAUCLUSE, NSW

Job No.: 32829SC | **Method:** HAND AUGER | **R.L. Surface:** ≈ 53.6m

Date: 20/11/2019 | **Datum:** AHD

Plant Type: | **Logged/Checked by:** O.E./T.C.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, with a trace of root fibres.	D			GRASS COVER
						0.5		SM	Silty SAND: fine to medium grained, grey.	D	VL		AEOLIAN
									Silty SAND: fine to medium grained, orange brown.	M			
						1			END OF BOREHOLE AT 0.8m				HAND AUGER REFUSAL DUE TO BOREHOLE COLLAPSE
						1.5							
						2							
						2.5							
						3							
						3.5							

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	LOUISE ST JOHN KENNEDY DESIGNER
Project:	PROPOSED ALTERATIONS AND ADDITIONS
Location:	18 OLPHERT AVENUE, VAUCLUSE, NSW
Job No.	32829SC
Date:	20-11-19
Tested By:	O.E.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	1	2	3	Test Location	1	2	3
Surface RL	≈ 59.0m	≈ 57.4m	≈ 54.7m	Surface RL	≈ 59.0m	≈ 57.4m	≈ 54.7m
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	SUNK	3000-3100	10	7	
100 - 200	1			3100-3200	10	9	
200 - 300	1		1	3200-3300	10	9	
300 - 400	1	1		3300-3400	10	10	
400 - 500	1		1	3400-3500	10	10	
500 - 600	1			3500-3600	10	11	
600 - 700	1	1	1	3600-3700	12	15	
700 - 800	2	1		3700-3800	10	17	
800 - 900	2	1	1	3800-3900	10	24	
900 - 1000	3	2	1	3900-4000	12	26	
1000 - 1100	4	2	3	4000-4100	12	25	
1100 - 1200	5	3	4	4100-4200	10	REFUSAL	
1200 - 1300	4	2	7	4200-4300	11		
1300 - 1400	5	3	10	4300-4400	11		
1400 - 1500	5	2	11	4400-4500	13		
1500 - 1600	5	4	10	4500-4600	16		
1600 - 1700	5	4	5	4600-4700	20		
1700 - 1800	5	3	5	4700-4800	21		
1800 - 1900	5	5	5	4800-4900	20		
1900 - 2000	5	3	4	4900-5000	14		
2000 - 2100	6	4	5	5000-5100	END		
2100 - 2200	7	3	5	5100-5200			
2200 - 2300	8	4	21	5200-5300			
2300 - 2400	7	4	REFUSAL	5300-5400			
2400 - 2500	9	4		5400-5500			
2500 - 2600	9	4		5500-5600			
2600 - 2700	10	2		5600-5700			
2700 - 2800	11	3		5700-5800			
2800 - 2900	10	4		5800-5900			
2900 - 3000	10	5		5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	LOUISE ST JOHN KENNEDY DESIGNER		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	18 OLPHERT AVENUE, VAUCLUSE, NSW		
Job No.	32829SC	Hammer Weight & Drop: 9kg/510mm	
Date:	20-11-19	Rod Diameter: 16mm	
Tested By:	O.E.	Point Diameter: 20mm	

Test Location	4	5	6	Test Location			6
Surface RL	≈ 51.5m	≈ 52.6m	≈ 54.7m	Surface RL			≈ 54.7m
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	SUNK	SUNK	SUNK	3000-3100			10
100 - 200	1			3100-3200			REFUSAL
200 - 300	2		2	3200-3300			
300 - 400	4	1	1	3300-3400			
400 - 500	4	1	2	3400-3500			
500 - 600	2	2	1	3500-3600			
600 - 700	3	2	2	3600-3700			
700 - 800	REFUSAL	3	1	3700-3800			
800 - 900		1	1	3800-3900			
900 - 1000		1	4	3900-4000			
1000 - 1100		1	17	4000-4100			
1100 - 1200			27	4100-4200			
1200 - 1300			18	4200-4300			
1300 - 1400			6	4300-4400			
1400 - 1500			4	4400-4500			
1500 - 1600			4	4500-4600			
1600 - 1700			3	4600-4700			
1700 - 1800		1	4	4700-4800			
1800 - 1900		2	4	4800-4900			
1900 - 2000		3	6	4900-5000			
2000 - 2100		2	5	5000-5100			
2100 - 2200		5	6	5100-5200			
2200 - 2300		7	6	5200-5300			
2300 - 2400		6	4	5300-5400			
2400 - 2500		8	5	5400-5500			
2500 - 2600		6/	4	5500-5600			
2600 - 2700		REFUSAL	4	5600-5700			
2700 - 2800			5	5700-5800			
2800 - 2900			6	5800-5900			
2900 - 3000			10	5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

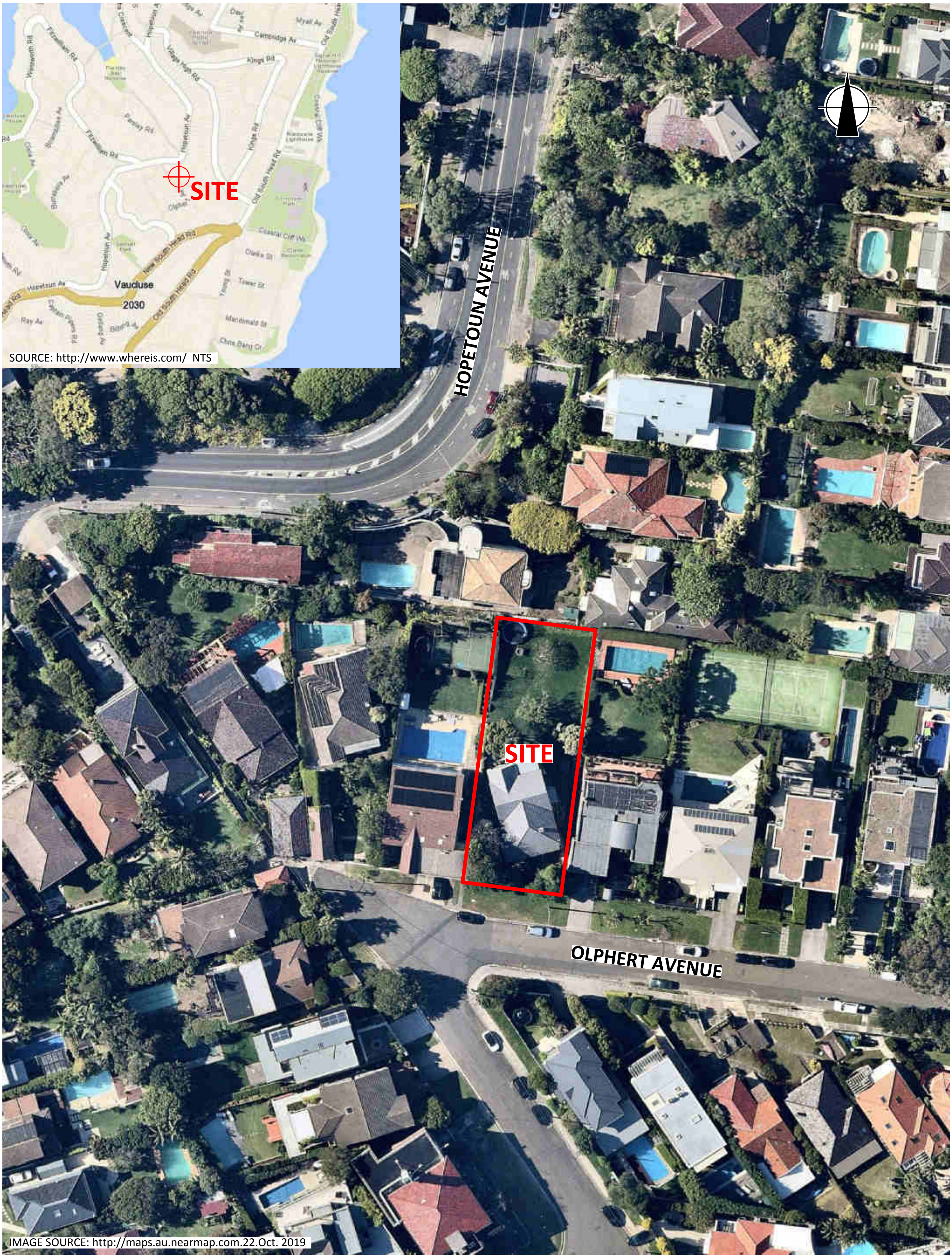
JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	LOUISE ST JOHN KENNEDY DESIGNER		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	18 OLPHERT AVENUE, VAUCLUSE, NSW		
Job No.	32829SC	Hammer Weight & Drop: 9kg/510mm	
Date:	20-11-19	Rod Diameter: 16mm	
Tested By:	O.E.	Point Diameter: 20mm	

Test Location	7	8	9	Test Location	7	8	
Surface RL	≈ 57.0m	≈ 58.5m	≈ 53.6m	Surface RL	≈ 57.0m	≈ 58.5m	
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	1	3000-3100	11	3	
100 - 200	2			3100-3200	12	6	
200 - 300			1	3200-3300	14	8	
300 - 400		1	1	3300-3400	18	10	
400 - 500		1		3400-3500	13	12	
500 - 600		1		3500-3600	14	12	
600 - 700			1	3600-3700	18	10	
700 - 800			1	3700-3800	22	10	
800 - 900		1	1	3800-3900	25	5	
900 - 1000		1	7	3900-4000	30	5	
1000 - 1100		3	3	4000-4100	REFUSAL	8	
1100 - 1200	2	2	2	4100-4200		12	
1200 - 1300	2	3	12	4200-4300		19	
1300 - 1400	2	2	11	4300-4400		24	
1400 - 1500	1	2	9	4400-4500		24	
1500 - 1600	1	1	4	4500-4600		30	
1600 - 1700		2	4	4600-4700		REFUSAL	
1700 - 1800	4	1	3	4700-4800			
1800 - 1900	5	1	3	4800-4900			
1900 - 2000	5	2	2	4900-5000			
2000 - 2100	4	2	4	5000-5100			
2100 - 2200	5	2	2	5100-5200			
2200 - 2300	5	2	4	5200-5300			
2300 - 2400	6	2	4	5300-5400			
2400 - 2500	5	2	4	5400-5500			
2500 - 2600	5	1	9	5500-5600			
2600 - 2700	5	2	10	5600-5700			
2700 - 2800	6	1	REFUSAL	5700-5800			
2800 - 2900	7	2		5800-5900			
2900 - 3000	8	2		5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD



0 10 20 30 40 50

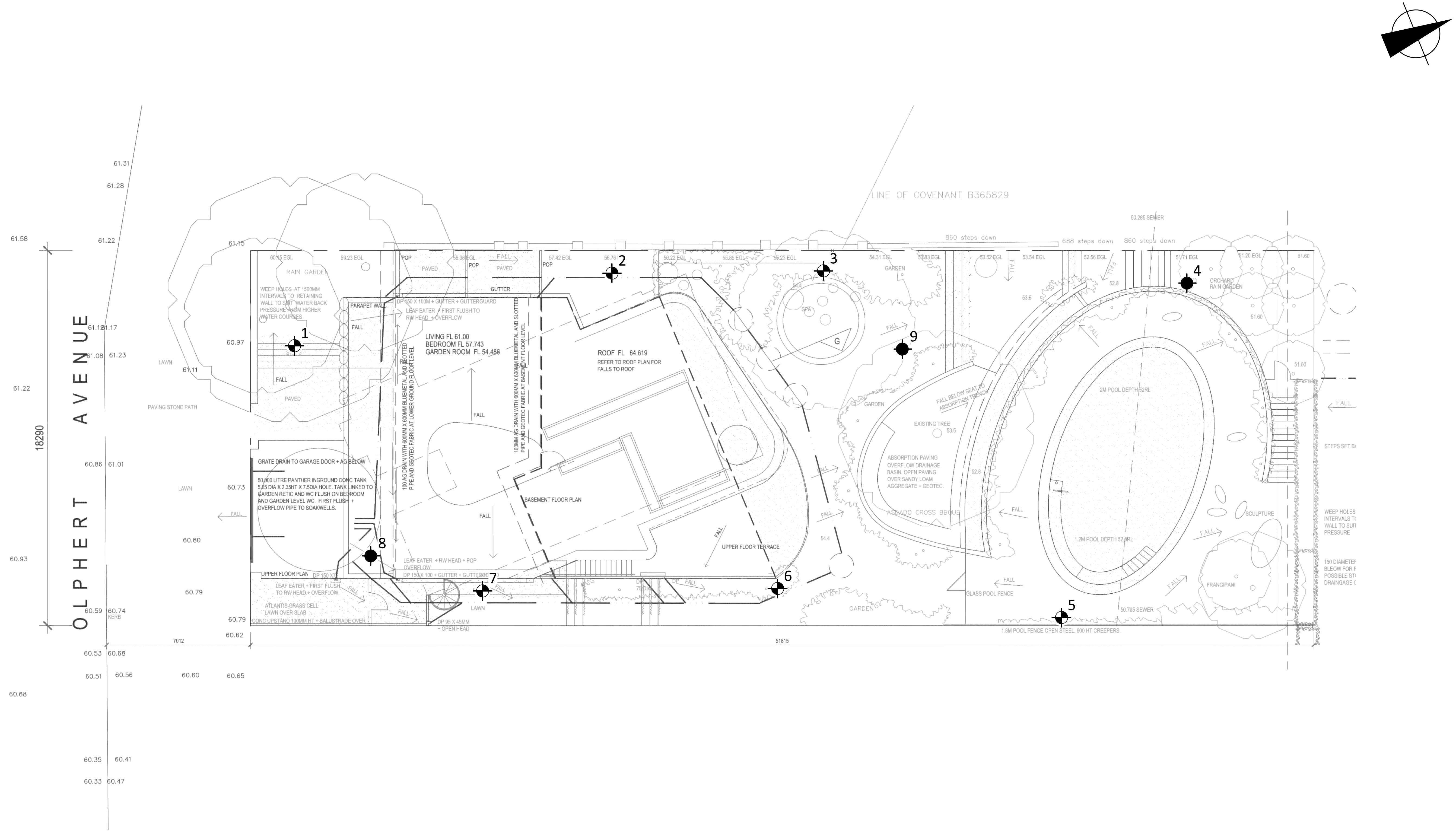
SCALE 1:1000 @A4 m

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**		
Location:	18 OLPHERT AVENUE VAUCLUSE, NSW		
Report No:	32829SC	Figure:	1

JKGeotechnics

PLOT DATE: 5/12/2019 3:07:55 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32829SC VAUCLUSE\CAD\32829SC.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST

Title:	**TEST LOCATION PLAN**
Location:	18 OLPHERT AVENUE VAUCLUSE, NSW
Report No:	32829SC
Figure:	2

0 2 4 6 8 10

SCALE 1:200 @A3 m

This plan should be read in conjunction with the JK Geotechnics report.

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PLOT DATE: 5/22/2019 3:08:06 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32829SC VAUCLUSE\CAD\32829SC.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

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

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

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

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- 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 be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to 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 perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

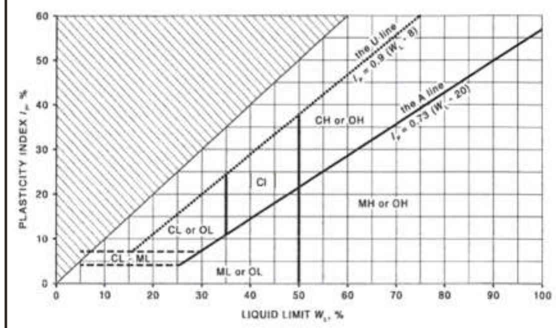
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE: ≤ 15; 0 – 4
	L	LOOSE: > 15 and ≤ 35; 4 – 10
	MD	MEDIUM DENSE: > 35 and ≤ 65; 10 – 30
	D	DENSE: > 65 and ≤ 85; 30 – 50
	VD	VERY DENSE: > 85; > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres