REPORT TO

ST AUGUSTINES COLLEGE SYDNEY

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED CARPARK

AT

33 CONSUL ROAD, BROOKVALE, NSW

Date: 20 October 2020

Ref: 33465PNrpt Rev1

JKGeotechnics

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33465PNrpt	Final Report	9 October 2020
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed carpark at 33 Consul Road, Brookvale. The location of the site is shown in Figure 1. The investigation was commissioned by Artazan Property Group (APG) contract dated 26 August 2020. The commission was on the basis of our fee proposal (Ref: P52185PN), dated 9 July 2020.

From the provided civil drawings (Project No. 2290, Dwg No.s C000 to C003, C100 to C102, C200 [2 options], C201, C202, C300 and C301, all Revision 2), we understand an on-grade carpark with 16 parking spaces is proposed on the subject site. Further, the proposed pavement is to comprise thin asphaltic concrete over a conventional granular base and subbase. A silt arrestor, approximately 2m deep below finished pavement surface level, is proposed towards the eastern end of the site. The proposed carpark level will be a maximum of about 1m lower than existing site levels, and excavation will be required. Further, retaining walls to a maximum height of about 1m are expected to be required around the eastern end of the site. We have assumed that only light vehicles will access the carpark, i.e. no truck access will be required.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions and to use this as a basis for providing comments and recommendations on site preparation and earthworks, retaining wall design, and pavement design.

2 INVESTIGATION PROCEDURE

Four test pits (TP1 to TP4), were excavated to depths between 0.3m (TP2) and 2.3m (TP4) using a 6 tonne excavator. The compaction of the fill and relative density of the natural sands were assessed from the results of Dynamic Cone Penetrometer (DCP) tests completed adjacent to TP1, TP3 and TP4. Groundwater observations were made during and on completion of excavating each test pit. No longer term groundwater monitoring was completed.

The test pit locations, as shown on the attached Test Pit Location Plan (Figure 2) were set out by taped measurements from existing surface features. Figure 2 is based on available Nearmap imagery. As no topographical survey information was provided, existing surface levels at the test pits were not estimated.

Our geotechnical engineer, Mr Ben Sheppard, was on site full time during the fieldwork and set out the test pit locations, nominated the sampling and testing, and prepared the test pit logs and DCP test result sheet. The test pit logs and DCP test results are attached to this report, along with our Report Explanation Notes which describe the investigation techniques adopted and define the logging terms and symbols used.

Selected bulk soil samples were returned to a NATA accredited laboratory, Soil Test Services Pty Ltd (STS), for Standard compaction and four day soaked CBR testing. The results of the tests are summarised in the attached STS Table A.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated across the lower reaches of a hillside which grades down to the east and south-east at about 11°. The site is bound by Consul Road to the west.

At the time of the fieldwork, the site had been cleared, and graded down to the east, predominantly at about 1°, but at between 5° and 6° over the easternmost 5m of the site. The eastern portion of the site appeared to have been raised by the placement of fill, resulting in the 5° to 6° filled batter to the eastern boundary and a steep batter down to this eastern filled area.

To the north and south of the site were single storey brick houses, set back between about 1m and 3m from the respective site boundaries.

To the east of the site was St Augustine's College, with a multi storey school building, garden area, and basketball courts adjacent to the subject site.

Surface levels across the site boundaries were similar.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney (geological Survey of NSW, Geological Series Sheet 9130) indicates that the site is underlain by Hawksbury Sandstone.

The test pits disclosed a generalised subsurface profile comprising a variable depth of fill over natural sands. Neither bedrock nor groundwater were encountered within the depth of the investigation. For details of the encountered subsurface conditions, reference should be made to the attached test pit logs. A summary of the encountered subsurface conditions is presented below:

Fill

Fill with a variable composition of predominantly clay, sand and gravel, was encountered from the surface in all of the test pits and extended to depths of 0.7m (TP1), 0.8m (TP3) and greater than 2.3m (TP4). TP2 refused at a depth of 0.35m within the fill profile on a concrete obstruction. A 0.5m thick concrete inclusion was exposed in the side of TP3 closest to TP2 and may have been the same obstruction. Based on the DCP test results, the fill was assessed as being moderately or well compacted, however, the presence of thick concrete inclusions indicates there may be areas of poor compaction. Inclusions of igneous and sandstone gravel, brick, tile and concrete fragments and sandstone cobbles were observed within the fill.

Natural Soils

Natural sands were encountered below the fill in TP1 and TP3, and extended to the test pit termination depths. In TP1, the sands were of loose then very loose to loose relative density, and in TP3, the sands were of medium dese then loose relative density.

Groundwater

The test pits were 'dry' during and a short time after completion of test pit excavation. No long-term groundwater monitoring has been undertaken.

3.3 Laboratory Test Results

The four-day soaked CBR tests carried out on samples of fill from TP1 and TP4 returned CBR values of 40% and 8% respectively when compacted to 98% of their Standard Maximum Dry Density (SMDD) at close to their Standard Optimum Moisture Content, and surcharged with 4.5kg.

4 COMMENTS AND RECOMMENDATIONS

4.1 Earthworks

4.1.1 Existing Fill

The existing fill on site comprised variable material, and whilst assessed to be generally moderately and well compacted, as no information on their placement or compaction has been provided, the fill is considered to be uncontrolled. Nevertheless, as about 1m of the existing fill will be removed to achieve design levels, and subject to their performance during proof rolling, we consider they would likely be suitable to support the proposed on-grade carpark, though with a somewhat increased risk of settlement and differential settlement compared to a properly engineered fill platform. Possible settlements and lateral movements would likely be worst near the crests of the steep batters. Replacing the entire depth of fill would be an expensive option, though would reduce the performance risk. Provided the slightly increased risk of settlement can be accepted, consideration could be given to adopting higher than usual surface falls to limit the potential for ponding from differential settlement.

4.1.2 Site Preparation

All vegetation as well as topsoil and root affected soils must be stripped from the site.

Stripped topsoil/root affected soils must be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes, subject to approval by the environmental consultant. Notwithstanding, reference should be made to Section 5 for guidance on the offsite disposal of excavated materials.

Where required, excavation may then be carried out to achieve design subgrade levels. We have assumed a maximum depth of excavation of about 1m except for at the proposed silt arrestor, where excavation will be required to a maximum depth of about 2.5m below existing levels. Such excavations are expected to encounter the fill profile only, but may also extend into the natural soil profile, and can be readily completed using buckets fitted to hydraulic excavators, however, alternative techniques may be required to break up the concrete obstruction(s) encountered in TP2 and TP3.

We note that the concrete obstruction encountered in TP2 must also be removed to avoid the formation of a 'hard spot' below the pavement.

Where space permits, temporary batter slopes through the soil profile are considered feasible and should be cut no steeper than 1 Vertical (V) on 1.5 Horizontal (H). Where temporary batters cannot be accommodated within the site geometry, temporary shoring will be required.

Any permanent batter slopes should be graded no steeper than 1V on 2.5H, provided the slopes are protected from erosion by quickly establishing a vegetative cover, together with surface drains along the crests of the batters to intercept surface water run-off. Where access for mowing etc. is required, permanent batters will likely need to be flattened to no steeper than 1V on 4H.

4.1.3 Subgrade Preparation

Following excavation to design levels, the exposed soil subgrade should be proof rolled with at least eight passes of a static (non-vibratory) smooth drum roller of at least 10 tonnes deadweight. The final passes of proof rolling should be witnessed by an experienced geotechnical engineer or earthworks superintendent for the detection of unstable or soft areas.

If soft or heaving areas are detected during proof rolling, then the heaving areas should be locally removed to a stable base and replaced with engineered fill, as outlined in Section 4.1.4 below, or further geotechnical advice should be sought. Further guidance on the treatment of heaving areas (e.g. subgrade improvement by provision of bridging layers) must be provided by the geotechnical engineer immediately following the proof rolling inspection. Based on the investigation results, heaving may occur where under compacted existing fill is present, or moisture contents are elevated.

If soil softening occurs after rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If a clayey subgrade exhibits shrinkage cracking, then the surface should be lightly watered and rolled until the shrinkage cracks are no longer evident.

4.1.4 Engineered Fill

Preferably, engineered fill should comprise a well graded granular material, e.g. the granular fill materials encountered on site, with a maximum particle size not exceeding 75mm. Such materials are less susceptible to softening than clayey soils, and provide a better subgrade for pavements. From a geotechnical perspective, the existing fill materials are considered suitable for reuse as engineered fill on condition that they are 'clean', free of organic matter and contain a maximum particle size not exceeding 75mm. Any excavated clays should be thoroughly blended with the granular materials at a ratio of no greater than 1 part clay to 3 parts granular material prior to being used as engineered fill. Any imported fill must be Virgin

Excavated Natural Material (VENM) or Excavated natural Material (ENM) and have a maximum particle size not exceeding 75mm.

We strongly recommend against the importation of clayey soils to site for use as engineered fill as they will inherently form a poorer quality subgrade than the existing granular soils on site, and would likely require a lower design CBR.

Engineered fill comprising granular materials should be compacted in maximum 200mm thick loose layers using a large static (non-vibratory) roller to a density ratio of at least 98% of SMDD. The vibratory mode on the roller should not be used due to the potential for vibration induced damage to nearby buildings.

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of the compaction plant that can be placed in trenches, backfilling should be carried out in maximum 150mm thick loose layers and compacted using a trench roller, a pad-foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a 'Wacker Packer'). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also be reduced to no more than 50mm. The compaction specification provided above is applicable.

As for services trenches, retaining wall backfilling, including to the silt arrestor, must also be carried out using engineered fill in order to reduce post-construction settlements. Compaction of the engineered backfill should be carried out using a trench roller or hand operated vertical rammer compactor for the lower layers and immediately behind the wall in the upper layers. Elsewhere a small static roller could be used. As per service trenches, backfilling should be carried out in 150mm thick loose layers and the maximum particle size of the backfill material should be reduced to no more than 50mm. The compaction specification provided above is also applicable to trench backfill.

Compaction of engineered fill behind retaining walls is difficult and laborious. Consideration should be given to using a single sized, hard and durable, free draining aggregate, such as 'Blue Metal' or crushed concrete aggregate (free of fines), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion into the aggregate. Provided the aggregate backfill is placed as recommended above, density testing would not be required in that material. The geotextile should then be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of engineered fill, with this 0.3m of engineered fill being below subgrade level, to reduce the potential for surface water infiltration.

In-situ density tests must be carried out on the engineered fill to confirm the above specification is achieved. The frequency of testing must be at least three tests per layer.

We consider Level 2 testing of fill compaction in accordance with AS3798-2007, to be appropriate for the proposed carpark, including for the trench and retaining wall backfill. Due to a potential conflict of interest, we strongly recommend that the geotechnical inspection and testing authority (GITA) be directly engaged by the Client or builder, and not by the earthworks contractor or sub-contractors.

4.2 Retention

Low height retaining walls, a maximum of say 1m high, are expected to be required around the eastern end of the site, however, the details of such walls have not been confirmed at this stage. The silt arrestor will also be acting as a retaining structure.

Free standing cantilever walls, where minor wall movements are tolerable, should be designed using a triangular lateral earth pressure distribution with an ‘active’ earth pressure coefficient (K_a) of 0.3 for the soil profile, assuming a horizontal retained/backfill surface.

Cantilever walls where wall movements are to be limited, should be designed using a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (K_0) of 0.5 for the soil profile, assuming a horizontal retained/backfill surface.

A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.

Any surcharge affecting the walls (e.g. traffic loading, construction loads, inclined backfill, adjacent high level footings, etc.) should be allowed for in the design using the appropriate earth pressure coefficient from above.

Retaining walls should be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

Lateral toe restraint of cantilever walls may be achieved by the passive resistance of the soil profile in front of the wall using a triangular lateral earth pressure distribution with a 'passive' earth pressure coefficient (K_P) of 2.8 for the existing fill or natural sands. We note that significant movement is required in order to mobilise the full passive pressure of a soil, and therefore a factor of safety of at least 2 should be adopted to reduce such movements. Any localised excavations in front of the wall should be taken into account in the embedment design. Friction on the base of a wall can be calculated using a friction angle of 28° between the retaining wall base and the existing fill below, provided the base is clean, rough and dry when the retaining wall footing is poured.

For retaining walls founded in the existing fill and/or natural sands, and founded at a depth of at least 0.5m below surrounding surface level an allowable bearing pressure of 50kPa can be adopted. Following excavation of such footings, the bases should be rigorously compacted using an upright rammer compactor or self-propelled trench roller to improve the near surface compaction of the foundation material. All retaining walls must incorporate articulation at no greater than 5m centres to allow for differential movements, which may occur due to settlement of the foundation material. Due to the nature of the fill material, accurate prediction of possible settlements is not possible, however, we suggest allowing for settlements in the order of 10mm to 15mm.

All retaining wall footings must be inspected by a geotechnical engineer to confirm the nature of the foundation material. Dynamic Cone Penetrometer (DCP) tests must also be completed in all retaining wall footings to confirm the suitability of the foundation material.

4.3 Pavement Design

Provided the subgrade has been prepared in accordance with the recommendations described in Section 4.1 above, and no clayey fill is utilised within 0.5m of subgrade level, a CBR value of 6% can be adopted for design of the proposed carpark pavement. Whilst this is lower than the lowest CBR of 8% returned from the laboratory testing, we consider this to be reasonable given the variability of the fill on site.

We recommend that all basecourse materials for flexible pavements and sub-base materials for rigid pavements comprise DGB20 complying with TfNSW QA Specification 3051 unbound base. The DGB20 material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller, operating in non-vibratory mode, to at least 98% of Modified Maximum Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement to reduce the potential for material breakdown.

We further recommend that all sub-base materials for flexible pavements comprise DGS40, DGS20 or DGB20 complying with TfNSW QA Specification 3051. The sub-base material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller, operating in non-vibratory mode, to at least 95% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement.

The final material and compaction specification must be determined by the pavement designer.

Density tests should be carried out on the granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least three tests per layer. Level 2 testing in accordance with AS3798-2007 would be considered acceptable for the pavement layers. The geotechnical testing authority (GTA) should be directly engaged by the Client or builder and not by the earthworks contractor or sub-contractors.

Subsoil drains should be provided below the perimeter of the proposed pavement, including any internal planters etc. with invert levels at least 200mm below design subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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. 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.

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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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33465PN
Project:	Proposed Carpark	**Report:**	A
Location:	33 Consul Road, Brookvale, NSW	**Report Date:**	10/09/2020
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TESTPIT NUMBER		TP 1	TP 4
DEPTH (m)		0.10 - 0.70	0.20 - 0.70
Surcharge (kg)		4.5	4.5
Maximum Dry Density (t/m^3)		1.81 STD	1.85 STD
Optimum Moisture Content (%)		13.9	15.5
Moulded Dry Density (t/m^3)		1.78	1.82
Sample Density Ratio (%)		98	98
Sample Moisture Ratio (%)		98	100
Moisture Contents			
Insitu (%)		9.4	16.5
Moulded (%)		13.7	15.4
After soaking and			
After Test, Top 30mm(%)		17.3	20.1
	Remaining Depth (%)	16.8	17.3
Material Retained on 19mm Sieve (%)		6*	3*
Swell (%)		0.0	0.0
C.B.R. value:	@2.5mm penetration		8
	@5.0mm penetration	40	

NOTES: Sampled and supplied by client. Samples tested as received.

- Refer to appropriate Test Pit logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 31/08/2020.
- * Denotes not used in test sample.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing. This document shall not be reproduced except In full without approval of the laboratory. Results relate only to the items tested or sampled.

10/09/2020

Authorised Signature / Date (D. Treweek)

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TEST PIT LOG

Test Pit No. **1**

Client: ST AUGUSTINES COLLEGE SYDNEY
Project: PROPOSED CARPARK
Location: 33 CONSUL ROAD, BROOKVALE, NSW

Job No.: 33465PN **Method:** TOOTHED BUCKET **R.L. Surface:** N/A
Date: 31/8/20 **Datum:** -
Plant Type: BOBCAT E60 **Logged/Checked by:** B.S./N.E.S.

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		REFER TO DCP TEST RESULTS	0			FILL: Gravelly sand, fine to medium grained, dark brown, fine to coarse grained igneous gravel, with brick and concrete fragments, trace of sandstone cobbles.	M			APPEARS MODERATELY TO WELL COMPACTED
			0.5		SP	SAND: fine to medium grained, light grey and grey, trace of silt fines, ash and roots.	M	VL		ALLUVIAL
			1					VL-L		
			1.5		SM	Silty SAND: fine to medium grained, orange brown, trace of cemented sandy clay nodules and quartz gravel.				
			2			END OF TEST PIT AT 2.05m				
			2.5							
			3							
			3.5							

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TEST PIT LOG

Test Pit No. **2**

Client: ST AUGUSTINES COLLEGE SYDNEY

Project: PROPOSED CARPARK

Location: 33 CONSUL ROAD, BROOKVALE, NSW

Job No.: 33465PN **Method:** TOOTHED BUCKET **R.L. Surface:** N/A

Date: 31/8/20 **Datum:** -

Plant Type: BOBCAT E60 **Logged/Checked by:** B.S./N.E.S.

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 COMPLE-TION				■		0			FILL: Gravelly sandy clay, low plasticity, dark brown, fine to medium grained sand, fine to coarse grained igneous gravel.	w>PL			
									FILL: Gravelly silty sand, fine to medium grained, dark grey, fine to coarse grained concrete fragments.	M			REFUSAL ON CONCRETE SLAB TEST PIT 3 EXPOSED EDGE OF SLAB, WHICH WAS ASSESED TO BE 500mm.t
						0.5			END OF TEST PIT AT 0.35m				
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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TEST PIT LOG

Test Pit No. **3**

1/1

Client: ST AUGUSTINES COLLEGE SYDNEY

Project: PROPOSED CARPARK

Location: 33 CONSUL ROAD, BROOKVALE, NSW

Job No.: 33465PN **Method:** TOOTHED BUCKET **R.L. Surface:** N/A

Date: 31/8/20 **Datum:** -

Plant Type: BOBCAT E60 **Logged/Checked by:** B.S./N.E.S.

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		REFER TO DCP TEST RESULTS	0			FILL: Gravelly sandy clay, low plasticity, brown, fine to medium grained sand, fine to medium grained igneous gravel, with brick, concrete and ceramic fragments, trace of sandstone cobbles.	w<PL			APPEARS WELL COMPACTED
			0.5			FILL: Gravelly silty sand, fine to medium grained, dark grey, fine to coarse grained igneous and sandstone gravel, trace of brick and concrete fragments.	M			
			1		SP	SAND: fine to medium grained, light grey, trace of silt fines and roots.	M	MD		ALLUVIAL
			1.5		SM	Silty SAND: fine to medium grained, brown and orange brown, trace of cemented sandy clay nodules.		L		
			2			END OF TEST PIT AT 2.0m				
			2.5							
			3							
			3.5							

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TEST PIT LOG

Test Pit No. **4**

Client: ST AUGUSTINES COLLEGE SYDNEY

Project: PROPOSED CARPARK

Location: 33 CONSUL ROAD, BROOKVALE, NSW

Job No.: 33465PN **Method:** TOOTHED BUCKET **R.L. Surface:** N/A

Date: 31/8/20 **Datum:** -

Plant Type: BOBCAT E60 **Logged/Checked by:** B.S./N.E.S.

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 COMPLE-TION		REFER TO DCP TEST RESULTS	0			FILL: Sandy clayey gravel, fine to coarse grained igneous, orange brown, fine to medium grained sand, with brick, concrete and tile fragments.	M			APPEARS MODERATLEY COMPACTED
						FILL: Gravelly sandy clay, low plasticity, orange brown, fine to coarse grained igneous gravel, with brick, tile and concrete fragments.	w≈PL			
			0.5							
			1			FILL: Silty sand, fine to medium grained, dark grey, with brick, tile and concrete fragments, trace of fine to coarse grained igneous and sandstone gravel, and clay fines.	M			
			1.5			as above, but trace of concrete, tile and brick fragments, without igneous and sandstone gravel.				
			2							
						END OF TEST PIT AT 2.3m				
			2.5							
			3							
			3.5							

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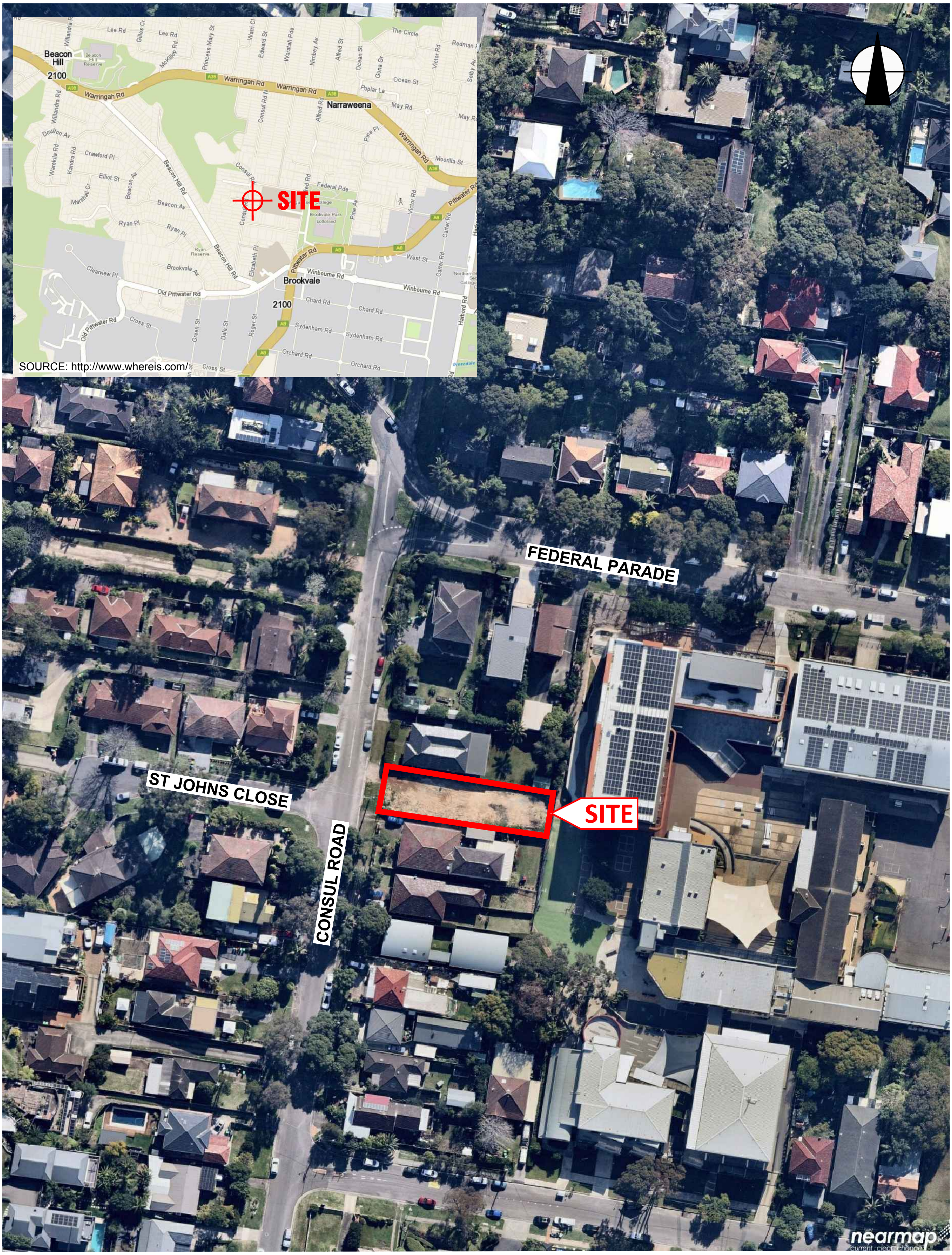
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ST AUGUSTINES COLLEGE SYDNEY		
Project:	PROPOSED CARPARK		
Location:	33 CONSUL ROAD, BROOKVALE, NSW		
Job No.	33465PN	Hammer Weight & Drop: 9kg/510mm	
Date:	31-8-20	Rod Diameter: 16mm	
Tested By:	B.S.	Point Diameter: 20mm	

Test Location	**TP1**	**TP3**	**TP4**				
Surface RL							
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	7	25	28				
100 - 200	14	15	8				
200 - 300	13	12	5				
300 - 400	8	22	5				
400 - 500	7	7	6				
500 - 600	5	8	5				
600 - 700	4	9	3				
700 - 800	2	3	4				
800 - 900	2	9	3				
900 - 1000	2	9	3				
1000 - 1100	2	5	5				
1100 - 1200	2	7	9				
1200 - 1300	3	5	12				
1300 - 1400	4	5	8				
1400 - 1500	1	5	7				
1500 - 1600	3	4	4				
1600 - 1700	2	3	4				
1700 - 1800	2	3	3				
1800 - 1900	2	3	5				
1900 - 2000	3	3	4				
2000 - 2100	9	3	4				
2100 - 2200	12	3	4				
2200 - 2300	17	2	4				
2300 - 2400	15	2	5				
2400 - 2500	16	3	4				
2500 - 2600	16	10	4				
2600 - 2700	19	19	3				
2700 - 2800	19	14	3				
2800 - 2900	25	9	3				
2900 - 3000	27 END	8 END	3 END				

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



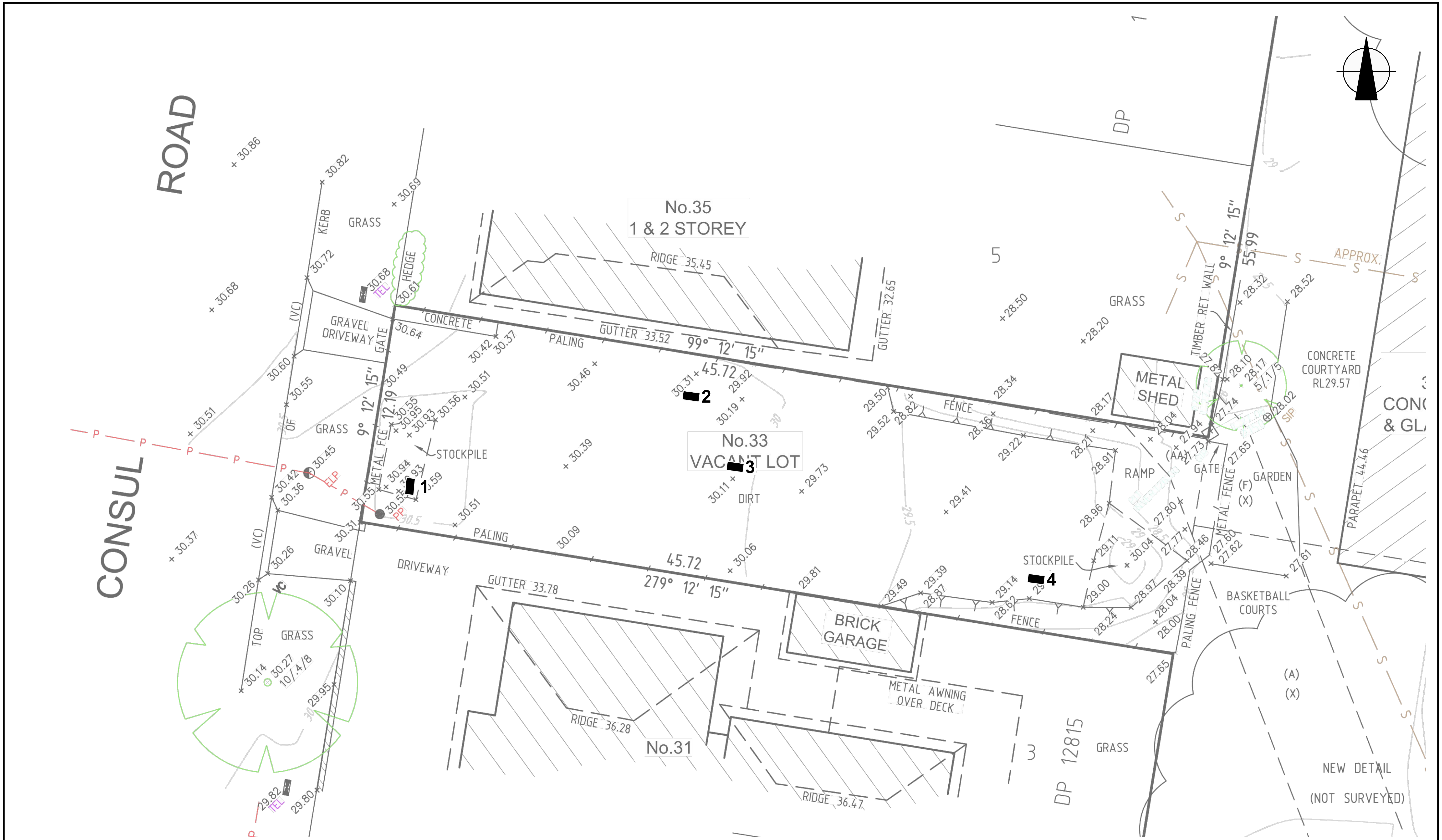
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	33 CONSUL ROAD, BROOKVALE, NSW
Report No: 33465PN	Figure No: 1

JKGeotechnics

PLOT DATE: 2/09/2020 3:30:52 PM DWG FILE: Y:\33000'S\33465PN BROOKVALE\CAD\33465PN.DWG



LEGEND

▬ TEST PIT

SCALE 0 2 4 6 8 10 METRES 1:200 @A3	Title: **TEST PIT LOCATION PLAN**
	Location: 33 CONSUL ROAD, BROOKVALE, NSW
	Report No: 33465PN — Figure No: 2
This plan should be read in conjunction with the JK Geotechnics report.	**JKGeotechnics**

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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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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:

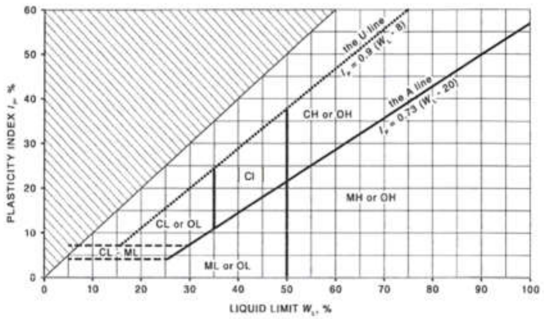
$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

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



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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	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	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	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	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

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $I_{s(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>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.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>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.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>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.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

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