REPORT

TO
JDH ARCHITECTS

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED SPORTS COMPLEX

AT
ST IVES HIGH SCHOOL, ST IVES, NSW

28 November 2018
Ref: 31754BCrpt

dlcs ISO 9001 certified AS/NZS 4801 certified ISO 14001 certified

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 28 November 2018
Report No: 31754BCrpt
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For and on behalf of
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for proposed sports complex within St Ives High School, Yarrabung Road, St Ives, NSW. The location of the site is shown on Figure 1. The investigation was commissioned by Ms Zoya Kuptsova of JDH Architects and was carried out in accordance our proposal dated 12 January 2018, Ref: P46569S.

The proposed development is currently at the concept stage and detailed architectural drawings are not available. From discussions with Ms Kuptsova, we understand that a two or four court indoor sports complex is proposed within the south-western corner of the school. The sports complex will be constructed at about the level of the existing outdoor sports courts and may require some minor earthworks as the proposed building will have a larger footprint than the existing courts. The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, earthworks, retention, footings and pavements.

The geotechnical investigation was carried out in conjunction with an environmental site assessment by our specialist division, Environmental Investigation Services (EIS). Reference should be made to the separate report by EIS, Ref: EP31754KT, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

The geotechnical investigation comprised the auger drilling of 5 boreholes (BH1 to BH5) using our track mounted JK205 drilling rig to refusal depths ranging from 3.6m to 5.6m below the existing ground surface.

The borehole locations, as shown on Figure 2, were located as close as possible to the locations nominated by JDH Architects and were set out by taped measurements from existing surface features.

The estimated compaction of the fill and the strength of the residual silty clays was assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on cohesive samples returned by the SPT split tube sampler. The strength of the underlying weathered bedrock was assessed from observation of the resistance to penetration by a Tungsten carbide (TC) bit attached to the augers, together with inspection of the recovered rock chip samples and subsequent correlation with moisture content test results. It must be noted that rock strength estimated in this way are approximate only and variations of one strength order should not be unexpected.

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

Our Geotechnical Engineer, Ms Cherry Alom, set out the borehole locations, nominated the testing and sampling locations and prepared logs of the subsurface conditions encountered. The borehole logs are attached, together with a set of explanatory notes, which describe the investigation procedures, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) a NATA accredited laboratory, for testing to determine moisture contents, Atterberg limits, linear shrinkages, standard compaction properties and four day soaked CBR values. The results of the laboratory testing are summarised in the attached STS Tables A and B. Samples were also collected from the boreholes for testing as part of the environmental site assessment by EIS.

3 RESULTS OF INVESTIGATION

3.1 Site Description

St Ives High School is located within gently undulating topography and on the side of hill that slopes down to the south at about 10°. The school is bound by Horace Street to the west, Yarrabung Road to the east and Hunter Avenue to the south. The site of this investigation is located within the south-western corner of the school.

The site comprises the western end of the grassed sports oval at the southern end of the school and the synthetic sports courts to the north. The grassed area at the western end of the oval slopes down towards the south at about 5° to 6° and has been formed by filling. Batters are located on the southern and western sides sloping down to the respective streets at about 20°. These batters contain several medium to large sized trees. This portion of the site contains cricket nets and sports fields.

Along the northern edge of the grassed area is a batter sloping up at about 15° to a relatively level area containing fenced synthetic surfaced courts. To the north of the courts is another grassed batter at about 8° leading up to the school buildings. To the west of the synthetic court area is an asphaltic concrete paved car park, which is accessed from Horace Street. To the north the school buildings comprise one and two storey brick and metal buildings. To the east are more grassed sporting fields.

3.2 <u>Subsurface Conditions</u>

The Sydney 1:100,000 Geological Series Sheet indicates the site lies close to the geological boundary of Ashfield Shale and Hawkesbury Sandstone of the Wianamatta Group. In summary, the boreholes encountered fill covering residual silty clay that graded into weathered siltstone and sandstone bedrock. Further comments on the subsurface conditions encountered are summarised below. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered at each borehole location.

Fill

BH1 to BH3, which were located in the northern portion of the site, encountered fill to depths ranging from 0.4m to 0.75m. BH4 and BH5, located within the southern portion, encountered deeper fill to depths of 2m and 3m, respectively. The fill comprised sand, silty sand, sandy clay and silty clay with ironstone and siltstone gravel. Based on the SPT 'N' values, the fill was assessed to be moderate to well compacted.

Residual Silty Clay

The residual silty clay was assessed to be of medium to high plasticity and of very stiff to hard strength.

Weathered Bedrock

Weathered siltstone and sandstone was encountered in all boreholes, generally shallower within the northern portion of the site and deeper in the southern portion where the overlying fill was deeper. In BH1, BH2 and BH3 siltstone or sandstone was encountered at depths of 0.9m, 1.9m and 1m, respectively, and in BH4 and BH5 it was deeper at depths of 4m and 3.7m, respectively. In BH1 and BH4, the siltstone was initially extremely weathered and became distinctly weathered and of low strength shortly thereafter. Within the remaining boreholes the rock was assessed to be distinctly weathered and of at least low strength on first contact. With depth the rock was assessed to be of medium to high strength, which caused refusal of the TC bit at depths ranging from 3.6m to 5.6m. In BH4 and BH5, the rock contained ironstone bands of up to 0.9m thickness.

No groundwater was encountered during or on completion of drilling. However, in BH4 groundwater was measured at a depth of 4.5m, one hour following completion of drilling.

3.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results the residual silty clay tested is of medium to high plasticity and is assessed to have a moderate to high potential for shrink/swell movements with changes in moisture content. The moisture content test results on samples of the weathered rock showed reasonably good correlation with our field assessment of rock strength.

The four day soaked CBR tests on samples of the residual silty clay from BH2 and silty clay fill from BH4 compacted to 98% of their Standard Maximum Dry Density (SMDD) measured CBR values of 6% and 5%, respectively.

4 COMMENTS AND RECOMMENDATIONS

Since detailed architectural drawings are not available at the time of preparing this report exact deign levels of the proposed building are not known. For the purposes of this report we have assumed that the sports centre building will be constructed at close to the current ground surface level of the existing synthetic sports courts. We have assumed that some minor earthworks of generally less than 1m will be required. The comments and recommendations contained herein are based on these assumptions and must be reviewed once the exact details, including design levels, of the proposed development have been determined.

4.1 Excavation and Groundwater

Given the expected shallow depth of any excavations of less than 1m we expect that these would encounter fill and residual soils, which should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators.

Groundwater seepage is not expected within any such shallow excavations, but if seepage does occur it should be able to be controlled using gravity drainage given the slope of the site.

4.2 Subgrade Preparation and Filling

Fill was encountered within the boreholes to depths ranging from 0.4m to 3m. We are unaware of any records of placement or compaction control of the fill and as such it must be considered uncontrolled. Such uncontrolled fill is not suitable for support of footings or floor slabs. Where shallow fill was encountered this may be excavated and replaced with controlled fill to allow the use of floor slabs supported on the fill, if the fill is not removed as part of any proposed excavations.

Within areas where floor slabs are proposed all existing fill should be fully stripped to expose the residual silty clay. Within pavement areas the vegetation and root affected soils should be stripped, but the fill below may be left in place. This root affected fill is not suitable to reuse as engineered fill, but may be reused within landscaped areas.

Following stripping, the exposed subgrade should be proof rolled with at least 7 passes of a minimum 8 tonne dead weight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and in the presence of a geotechnical engineer to detect any weak subgrades areas. Care must be taken during rolling due to the risk of damage to adjoining structures from the vibrations generated by the roller. If vibrations are of concern the rolling may need to be carried out with a static roller only.

Any weak or unstable areas detected during proof rolling should be locally excavated to a sound base and the excavated material replaced with controlled, engineered fill, or as directed by the geotechnical engineer during proof rolling. Some weak subgrade areas may be experienced where the existing fill is left in place or where the clays are allowed to soften due to water ponding. Following treatment of weak areas, engineered fill should be placed in thin layers as recommended in section 4.2.1 below.

In view of the high reactivity potential of some of the residual clays, particular attention should be given to providing adequate drainage both during construction and for long term site maintenance. The principal aim of the drainage should be to promote run-off and reduce ponding. Placement of a blinding layer of durable granular fill or subbase material to provide a trafficable surface during construction may be necessary or desirable. The earthworks should be carefully planned and scheduled to maintain cross-falls during construction. If the clay is exposed to prolonged periods of rainfall, softening will result and site trafficability will be poor. If soil softening occurs, the subgrade should be over-excavated to below the depth of moisture softening and the excavated material replaced with engineered fill.

4.2.1 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated material may be reused as engineered fill, provided it is free of deleterious materials and particles greater than 75mm in size. All excavated material should be inspected and approved by a geotechnical engineer prior to reuse. Any clay fill should be compacted in maximum 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of Standard Optimum moisture content (SOMC).

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. Where fill is to support footing loads it should be placed under Level 1 control as defined by AS3798-2007. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.3 Batters and Retaining Walls

Details of any batters or retaining walls are not known at this stage, but given the space available on site we expect that the use of temporary batters to allow construction of retaining walls will be possible. The following recommendations are appropriate for batters and retaining walls of less than 3m in height located well clear of any existing structures. If larger batters or retaining walls are proposed or excavation close to existing structures is proposed then additional geotechnical advice should be obtained.

Temporary batters of no more than 3m in height should be no steeper than 1 Vertical in 1 Horizontal (1V:1H). 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. Permanent batters should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. All permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent batters to also reduce erosion.

Permanent retaining walls constructed at the base of the batters may be designed as cantilevered walls based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of 20kN/m^3. Where walls are restrained from some lateral movements, such as by other structural elements in front of the wall, or where movements are to be kept low, an 'at rest' earth pressure coefficient, K_0, of 0.6 should be used.

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 must be allowed for in the design, plus full hydrostatic pressures unless measures are undertaken to provide complete and permanent drainage behind the wall.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The excavated clay will be difficult to properly compact within the limited space available behind the walls and consideration should be given to the use of more readily compactable materials, such as ripped or crushed rock or gravel. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas a lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. If clay fill is to be used a greater control of fill compaction and moisture control will be required and further geotechnical advice on the use of such material should be obtained. An alternative for backfill would also be to use a uniform granular material, such as crushed concrete of 30mm to 70mm in size, surrounded in a geofabric, with a capping layer of clay to reduce infiltration behind the wall.

4.4 Footings

The appropriate footing system for the proposed sports complex building will depend on the final design levels. The rock is generally at shallow depths and our preferred footing system would be to found all footings within the rock.

Where rock is exposed or is at shallow depths of less than about 1m, pad or strip footings may be used. Where the depth of rock is more than about 1m bored piers would be more practical.

Footings founded within siltstone or sandstone of at least low strength may be designed based on an allowable bearing pressure of 1000kPa. Where piers are used, an allowable shaft adhesion of 100kPa may be used for the design of piles in compression, provided socket cleanliness and roughness is maintained. At least the initial stages of footing excavation or pile drilling should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

Higher bearing pressures are likely to be possible within the rock of medium or higher strength, but additional cored boreholes would be required to assess the rock strength and defects in more detail and allow the use of higher bearing pressures.
Where all existing fill is excavated and replaced with controlled fill and footing excavations for a structure will not encounter the rock, shallow footings founded within the soils may be used, such as a stiffened raft slabs. This would only be appropriate for structures independent of the main structure supported on rock. Such footings may be designed based on an allowable bearing pressure of 100kPa for engineered fill or residual silty clays of stiff strength or 200kPa for residual silty clay of at least very stiff strength. Such footings must be designed to accommodate shrink/swell movements of the soils, which will depend on the reactivity of the material used for fill. We expect that movements similar to a Class H2, as defined by AS2870-2011 would be appropriate, but this must be confirmed following completion of the earthworks.

4.5 Pavements

The soaked CBR tests carried out for this investigation gave CBR values of 6% for the residual silty clays and 5% for the silty clay fill. However, we recommend that the pavements be designed based on a soaked CBR of 3%, or an estimated modulus of subgrade reaction of 20kPa/mm (750mm plate), to allow for variability in the quality of the fill. This assumes that the pavement subgrade is prepared as recommended in Section 4.2. Where fill is used to raise site levels, or replace unsuitable subgrade by the appropriate depth, pavement design may reflect the thickness and four day soaked CBR value of the imported material.

Concrete pavements should have a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2014) unbound base material (or similar good quality and durable fine crushed rock), which is compacted to at least 100% of SMDD. Concrete pavements should be designed with an effective shear transmission at all joints by way of either doweled or keyed joints.

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.

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.

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
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations & Additions
Location: St Ives High School, St Ives, NSW

Ref No: 31754BC
Report: A
Report Date: 29/08/2018
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	0.50-0.90	13.5	40	19	21	10.5
1	2.60-3.00	7.1				
1	3.50-3.70	5.5				
2	3.00-3.50	2.2				
3	0.50-0.95	25.1	53	23	30	14.5
3	2.50-3.00	4.9				
4	4.00-4.20	15.3				
5	4.60-5.00	7.6				

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/8/18

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	31754BC
Project:	Proposed Alterations and Additions	**Report:**	B
Location:	St Ives High School, St Ives, NSW	**Report Date:**	29/08/2018
		Page 1 of 1	

BOREHOLE NUMBER	2	4
DEPTH (m)	0.40 - 1.30	0.30 - 1.50
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.59 STD	1.67 STD
Optimum Moisture Content (%)	22.5	19.0
Moulded Dry Density (t/m^3)	1.57	1.63
Sample Density Ratio (%)	99	98
Sample Moisture Ratio (%)	98	101
Moisture Contents		
Insitu (%)	20.9	17.7
Moulded (%)	22.1	19.3
After soaking and		
After Test, Top 30mm(%)	27.9	23.9
Remaining Depth (%)	25.0	20.5
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	0.5	0.5
C.B.R. value: @5.0mm penetration	6	5

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample:15/8/18

Authorised Signature / Date
(A. Tatikonda) 29/8/18

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 1

1/1

Client: JDH ARCHITECTS

Project: PROPOSED SPORTS COMPLEX

Location: ST IVES HIGH SCHOOL, ST IVES

Job No. 31754BC **Method:** SPIRAL AUGER JK205 **R.L. Surface:** N/A

Date: 16/8/18 **Datum:**

Logged/Checked by: C.A./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Sand, fine to coarse grained, brown.	D			
				FILL: Silty sand, fine to medium grained, dark brown, with bark material.	M			
	N = 29 5,12,17		CI	Silty CLAY: medium plasticity, light grey and orange brown, with root fibres.	w<PL	Hd	>600 >600 >600	RESIDUAL
		1	-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey.	XW	Hd		ASHFIELD SHALE
				SILTSTONE: dark grey and red brown, with iron indurated bands.	DW	L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
		2	-	SANDSTONE: fine to medium grained, grey and red brown, with dark grey siltstone bands and iron indurated bands.		L-M		HAWKESBURY SANDSTONE MODERATE RESISTANCE WITH HIGH BANDS
		3				M		
				END OF BOREHOLE AT 3.7m				'TC' BIT REFUSAL
		4						
		5						
		6						
		7						

SAMPLES: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: JDH ARCHITECTS
Project: PROPOSED SPORTS COMPLEX
Location: ST IVES HIGH SCHOOL, ST IVES

Job No. 31754BC
Method: SPIRAL AUGER JK205
R.L. Surface: N/A
Date: 16/8/18
Datum:
Logged/Checked by: C.A./D.B.

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			FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	M			GRASS COVER APPEARS MODERATELY COMPACTED
		N = 12 3,5,7				FILL: Silty clay, high plasticity grey and orange brown, with root fibres.	w≈PL		550 560 >600	
			1		CH	Silty CLAY: high plasticity, grey and orange brown, with root fibres.	w≈PL			RESIDUAL
		N > 24 8,11, 13/100mm REFUSAL				Silty CLAY: high plasticity, orange brown and light grey, with medium to coarse grained ironstone gravel.	w<PL	Hd	>600 >600 >600	
			2		-	SILTSTONE: dark grey, with iron indurated bands.	DW	L-M		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			3		-	SANDSTONE: fine to medium grained, light grey and orange brown, with iron indurated bands.		M		HAWKESBURY SANDSTONE MODERATE RESISTANCE HIGH RESISTANCE 'TC' BIT REFUSAL
			4			END OF BOREHOLE AT 3.7m				
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: JDH ARCHITECTS
Project: PROPOSED SPORTS COMPLEX
Location: ST IVES HIGH SCHOOL, ST IVES

Job No. 31754BC
Date: 16/8/18
Method: SPIRAL AUGER JK205
Logged/Checked by: C.A./D.B.
R.L. Surface: N/A
Datum:

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			0			FILL: Sandy clay, low plasticity, dark brown, with root fibres and fine to medium grained sand.	w<PL			GRASS COVER
		N = 15 4,6,9				FILL: Silty clay, medium plasticity, brown.				
					CH	Silty CLAY: high plasticity, orange brown and grey, trace of medium to coarse grained ironstone gravel.	w>PL	Hd	500 580 >600	RESIDUAL
			1		-	SANDSTONE: fine to medium grained, orange and red brown, with iron indurated bands.	DW	L		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			2			SANDSTONE: fine to medium grained, orange and red brown, with dark grey siltstone bands and iron indurated bands.		M		MODERATE RESISTANCE WITH HIGH BANDS
			3			SANDSTONE: fine to medium grained, light grey and orange brown.				
						END OF BOREHOLE AT 3.6m				'TC' BIT REFUSAL
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. 4
1/1

Client: JDH ARCHITECTS
Project: PROPOSED SPORTS COMPLEX
Location: ST IVES HIGH SCHOOL, ST IVES

Job No. 31754BC
Date: 16/8/18
Method: SPIRAL AUGER JK205
R.L. Surface: N/A
Datum:
Logged/Checked by: C.A./D.B.

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			FILL: Sandy clay, low plasticity, dark brown, fine to medium grained sand, with root fibres.	w<PL			GRASS COVER
		N = 10 4,5,5				FILL: Silty clay, high plasticity, orange brown, light grey and brown.	w≈PL		450 400 500	APPEARS MODERATELY COMPACTED
			1							
		N = 10 5,5,5				as above, but with trace of medium to coarse grained siltstone gravel.	w<PL		>600 >600 >600	
			2		CI	Silty CLAY: medium plasticity, light grey mottled red brown.	w≈PL	VSt		RESIDUAL
		N = 11 4,4,7	3						300 310 300	
			4		-	Extremely Weathered siltstone: silty CLAY, medium plasticity, light grey.	XW	Hd		HAWKESBURY SANDSTONE
AFTER 1 HR					-	SANDSTONE: fine to coarse grained, light grey.	DW	L		VERY LOW 'TC' BIT RESISTANCE
					-	IRONSTONE BAND: 200mm.t		H		LOW RESISTANCE HIGH RESISTANCE
			5		-	SANDSTONE: fine to coarse grained, light grey.		M-H		MODERATE RESISTANCE HIGH RESISTANCE 'TC' BIT REFUSAL
						END OF BOREHOLE AT 5.6m				
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 5
1/1

Client: JDH ARCHITECTS
Project: PROPOSED SPORTS COMPLEX
Location: ST IVES HIGH SCHOOL, ST IVES

Job No. 31754BC
Method: SPIRAL AUGER JK205
R.L. Surface: N/A
Date: 16/8/18
Datum:
Logged/Checked by: C.A./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Sandy clay, low plasticity, dark brown, with root fibres, fine to medium grained sand.	w<PL			GRASS COVER
	N = 13 4,6,7			FILL: Silty clay, medium plasticity, brown, orange brown and light grey, trace of root fibres and medium to coarse grained ironstone gravel.				APPEARS MODERATELY TO WELL COMPACTED
	N = 23 9,14,9	1		FILL: Silty clay, medium plasticity, light grey and orange brown, trace of root fibres.				
	N = 20 5,9,11	3	CI	Silty CLAY: medium plasticity, grey and orange brown, with medium to coarse grained ironstone gravel.	w<PL	Hd	600 >600 >600	RESIDUAL
			-	IRONSTONE BAND: 900mm.t	DW	H		HAWKESBURY SANDSTONE HIGH 'TC' BIT RESISTANCE
			-	SANDSTONE: fine to coarse grained, light grey and orange brown.		M		MODERATE RESISTANCE
				END OF BOREHOLE AT 5.2m				'TC' BIT REFUSAL

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	**SITE LOCATION PLAN**
Location:	ST IVES HIGH SCHOOL ST IVES, NSW
Report No: 31754BC	Figure No: 1

JK Geotechnics

PLOT DATE: 28/11/2018 11:11:39 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31754BC ST IVES\CAD\31754BC.DWG

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

0 10 20 30 40 50
SCALE 1:1000 @A4 METRES

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

Title:	**BOREHOLE LOCATION PLAN**
Location:	ST IVES HIGH SCHOOL ST IVES, NSW
Report No: 31754BC	Figure No: 2

JK **Geotechnics**

PLOT DATE: 24/08/2018 1:52:28 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000's\31754\CAD\31754BC.DWG

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:

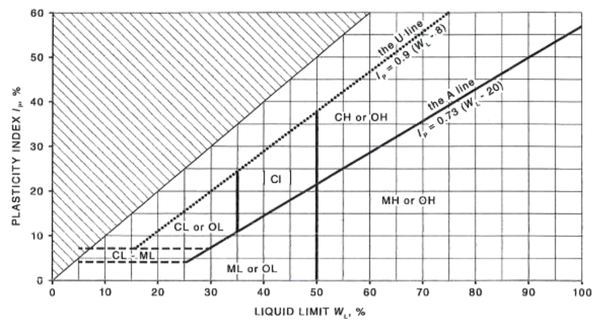
$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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	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

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**Point Load Strength Index $Is_{(50)}$ (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