REPORT

TO
LAM CONSULTING ENGINEERS

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MANLY LODGE REDEVELOPMENT

AT
22 VICTORIA PARADE, MANLY, NSW

23 June 2015
Ref: 28431SBrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5001
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as
JK Geotechnics ABN 17 003 550 801

Date: 23 June 2015
Report No: 28431SBrpt
Revision No: 0

Report prepared by:
Daniel Bliss
Senior Associate Geotechnical Engineer

Report reviewed by:
Paul Stubbs
Principal Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

This Report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JK) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JK and its Client and is therefore subject to:

a) JK's proposal in respect of the work covered by the Report;

b) the limitations defined in the Client's brief to JK;

c) the terms of contract between JK and the Client, including terms limiting the liability of JK.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JK which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JK does so entirely at their own risk and to the fullest extent permitted by law, JK accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed redevelopment of the Manly Lodge Hotel at 22 Victoria Parade, Manly, NSW. The investigation was commissioned by Mr Allan Lam of Lam Consulting Engineers and was carried out in accordance with our proposal dated 15 May 2015, Ref: P40518SB.

As shown on the supplied development drawings by Morson Group (Drawing Nos SK01 to SK05, Issue 1P, dated 17/2/15) the existing hotel will be demolished and a new hotel constructed with three above ground levels and one basement level. Excavations for the basement will be required to depths of about 3m. The basement will be offset from the north-western and south-eastern boundaries by about 2m, from the north-eastern boundary by about 1m and from the south-western boundary by about 0.7m.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions where access was possible for our small drilling rig, as a basis for preliminary comments and recommendations on geotechnical issues for the proposed development, such as excavation, groundwater, retention, footings and further geotechnical investigation required following demolition.

This geotechnical investigation was carried out in conjunction with an acid sulphate soil assessment by our specialist division, Environmental Investigation Services (EIS). Reference should be made to the separate report by EIS, Ref: E28431Krpt, for the results of the acid sulphate soil assessment.

2 INVESTIGATION PROCEDURE

Due to the existing building on the site access was only possible along the pathway on the south-western side of the building for our small track mounted JK205 rig. Borehole BH1 was auger drilled using the JK205 rig to a depth of 6.4m below the existing ground surface. A second borehole was attempted, but following coring of the concrete at BH2 a PVC pipe was discovered and the borehole abandoned. The rig position was adjusted slightly and the concrete cored again in BH2A, but buried concrete was encountered and this borehole was also terminated in case the buried concrete was part of a concrete encased service pipe.

The borehole locations, as shown on Figure 1, were set out by taped measurements from existing surface features and inferred site boundaries. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plan by Survcorp Pty Ltd (Ref: 2676, dated 13/5/15). The datum of the levels is Australian Height Datum (AHD).

Within BH1, the apparent compaction of the fill and the relative density of the natural sands was assessed from Standard Penetration Test (SPT) ‘N’ values.

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

Our engineering geologist, Mr Ian Squibbs, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, which include field test results and groundwater observations, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Envirolab Services Pty Ltd, a NATA registered laboratory, for testing to determine pH, sulphate content, chloride content and resistivity. The results of the laboratory testing are summarised in the attached Envirolab Report No. 129032. Samples were also collected from BH1 for testing as part of the acid sulphate soil assessment by EIS.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is situated on the relatively flat, low lying topography between Manly Cove and Manly Beach and has a north-western frontage on Victoria Parade.

The site is occupied by the Manly Lodge Hotel, which is two storey rendered building that covers the majority of the site and extends up to its north-eastern and north-western boundaries. At the rear of the site is a second one and two storey brick building. The two buildings are separated by a small paved courtyard, which is accessed from Victoria Parade by a paved pathway running along the south-western boundary. The buildings appeared to be generally in good external condition, with the exception of several hairline cracks noted in some walls.

To the north-east of the site is a four storey rendered apartment building with a 2.5m wide concrete driveway running along the common boundary. The building appeared to be in good external condition, although the driveway contained several longitudinal cracks. Beyond the south-western boundary is the garden of a three and four storey brick apartment building offset by about 4m from the common boundary. The building appeared to be in good external condition. A row of palm trees of about 10m high run along this boundary. The properties to the south-east could not be seen from within the subject site, but we understand that adjoining the common boundary are the rear yards of houses fronting Ashburner Street.

3.2 Subsurface Conditions

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Quaternary marine sand deposits.

Concrete paving was encountered in all boreholes being of 150mm to 200mm thickness. BH2 and BH2A were terminated on penetration of the concrete due to possible services.

In BH1, fill was encountered below the concrete to a depth of 0.7m. The fill comprised sand and silty sand, with a trace of sandstone gravel and root fibres. Based on the SPT results, the fill was assessed to be poorly compacted.

The natural sand was of loose relative density to a depth of 4.5m where sand of medium dense relative density was encountered. The SPT at a depth of 6m refused at a depth of 6.4m, which indicates that either a cemented layer is present or the density of the sands may increase below this depth. The refusal is not expected to be on bedrock as from experience with previous geotechnical investigations on the north-western side of Victoria Parade the bedrock is at depths of about 30m.

The sand below a depth of 4m was assessed to be wet, indicating groundwater seepage. On completion of drilling the borehole collapsed at a depth of 4.3m, which in sands tends to occur at the groundwater level.

Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered.

3.3 Laboratory Test Results

The soil pH values indicate that the soils are alkaline at 8.3 to 8.6. The sulphate and chloride contents were found to be low. Based on these results and the resistivity results, the soils would be classified as 'non-aggressive' exposure classification for both concrete and steel piles in accordance with Tables 6.4.2(C) and 6.5.2(A) of AS2159-2009 'Piling – Design and Installation'.

4 COMMENTS AND RECOMMENDATIONS

4.1 Subsurface Conditions and Additional Geotechnical Investigation

Due to restricted access to the site and obstructions encountered during drilling, this preliminary geotechnical investigation was limited to one borehole that was able to be drilled to below the depth of the proposed excavation. Therefore the preliminary comments and recommendations provided herein are based on the results of the one borehole.

There may be variation in the subsurface conditions throughout the site, in particular to the depth of changes in the relative density of the sand and the depth of fill. Therefore, following demolition of the existing buildings, further geotechnical investigation of the site must be carried out to provide details of the subsurface conditions throughout the site. Due to the sandy soils, the most appropriate method of investigation is Electrical Friction Cone Penetration (EFCP) testing, which involves pushing an instrumented cone into the soil to measure the resistance on the conical tip and the friction on a following sleeve. These tests provide a near continuous profile of the soils and can identify thin clay layers or very loose layers that may not be identified within boreholes.

Another issue that should be investigated in the depth of groundwater and to achieve this we recommend that standpipes be installed as part of the detailed geotechnical investigation to measure the groundwater levels over time.

For this site, we recommend that least four EFCP tests be carried out and at least two standpipes be installed as part of the detailed geotechnical investigation. The comments and recommendations provided herein may be used for preliminary design, but must be confirmed and amplified following completion of the detailed geotechnical investigation.

Overall, we consider that the site is geotechnically suitable for the proposed development and will impose no more risk that other similar developments on nearby sites.

4.2 Excavation

Prior to the start of excavation, dilapidation surveys should be completed on the adjoining buildings located within a horizontal distance from the basement walls of twice the excavation depth. This would comprise the adjoining buildings to the north-east and south-west, and possibly the buildings to the south-east. The dilapidation surveys should comprise detailed inspections of the buildings, both externally and internally, with all defects rigorously described, i.e. defect type, defect location, crack width, crack length, etc. The respective owners of the adjoining buildings should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation reports will also help to guard against opportunistic claims for damage that was present prior to the start of excavation.

Excavation to the required depth of about 3m will encounter fill and natural sand. These soils will be able to be excavated using conventional excavation equipment, such as the buckets of hydraulic excavators. However, the soils will not be self supporting and full depth retention systems will be required as recommended in Section 4.4 below.

The excavated material will need to be tested for contamination so it can be classified for appropriate disposal.

4.3 Groundwater

Groundwater seepage was encountered at a depth of 4m and the borehole collapsed on completion at a depth of 4.3m, which is an indication of the groundwater level. The groundwater levels should be assessed as part of the detailed geotechnical investigation.

If the groundwater is at depths of about 4m to 4.3m then it will be below the base of the proposed excavation and dewatering to construct the proposed basement will not be required. In the long term, allowance should be made for groundwater rise of at least 1m above the measured groundwater levels. If this is above the base of the basement slab then the basement would need to be designed to resist hydrostatic uplift forces, i.e. a tanked basement unless pumped drainage is allowed to manage such occasional situations as they may arise. Waterproofing of the basement walls requires careful consideration.

4.4 Retention

The basement is proposed within a few metres of the site boundaries, so insufficient space will be available for temporary batters and full depth retention systems will need to be installed prior to the start of excavation. The retention system could comprise contiguous or secant piles founded below the base the proposed excavation or a cutter soil mix (CSM) wall. If contiguous piles are adopted the gaps between the piles should be progressively filled during excavation, say at depth intervals of no more than 1.5m, to reduce the loss of soils from between the piles. Bored piers would not be suitable for this site due to the sandy soils and auger, grout injected (CFA) piles should be used. If a CSM wall is preferred then durability issues must be satisfied.

Given the limited depth of the walls of 3m, they may be designed as cantilevered walls based on a triangular earth pressure distribution. However, we recommend that this be used within an 'at rest' earth pressure coefficient, K_0, of 0.6 and a bulk unit weight of 20kN/m^3 to limit deflections. This coefficient assumes a horizontal backfill surface and if inclined backfill is proposed the coefficient should be increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic loads, unless measures are undertaken to provide complete and permanent drainage behind/through the walls (if this is permitted). If deflections of a cantilever wall are greater than the adjoining structures can tolerate, then anchors or props would be required. Anchored and propped walls are subject to a different earth pressure distribution which may be taken as rectangular and of magnitude 6H kPa where there are no movement sensitive structures within the zone of influence of the excavation or 8H kPa where there are. H is defined as the depth of excavation in metres.

A triangular passive earth pressure coefficient, K_p, of 3 may be used for the loose sands below the base of the excavation, including an allowance for over-excavation and localised excavations for footings, services, lift pits, etc. A factor of safety of at least 2 must be applied to the above passive earth pressure coefficient to reduce movements.

4.5 Footings

Assuming that the subsurface conditions within the entire site are similar to those encountered within the current borehole, natural sand of loose relative density will be exposed within the basement excavation. Therefore, the proposed structure could be supported on shallow pad or strip footings or a raft slab founded within the loose sand. Alternatively, piles could be used to found within sands of higher relative density.

The allowable bearing pressure for shallow pad or strip footings founded within sand is dependent on the footing size and the depth of embedment. For footings at least 0.5m wide embedded at least 0.5m into loose sands an allowable bearing pressure of 100kPa may be used. For the design of a raft slab on the loose sand allowable bearing pressures should be limited to 100kPa.

The allowable bearing pressure of piles founded within sands is dependent on the relative density of the sands, the embedment depth, and the pile diameter. All these factors would need to be taken into account during the design of the piles. As a guide, for single piles of at least 0.45m diameter founded within sands of medium dense relative density, with a pile embedment depth of at least 3m below the bulk excavation level an allowable bearing pressure of 650kPa would be appropriate. Where piles of at least 0.6m diameter are used, with an embedment depth of at least 4m, an allowable bearing pressure of 800kPa would be appropriate. The allowable bearing of pile groups or where contiguous or secant pile walls are load bearing is less than for single piles. If pile walls are load bearing these will be differential settlements to consider if internal footings are not piled. Further geotechnical advice should be obtained at detailed design stage on these issues.

4.6 Basement Floor Slab

The natural sands exposed within the basement excavation should be inspected by a geotechnical engineer and the subgrade proof rolled. The purpose of the proof rolling is to compact the surface sands that would have been loosened during excavation and to detect any weak subgrade areas. A subbase layer of good quality granular material of say 100mm thickness should be placed over the sand subgrade and below the basement slab.

Although the groundwater levels are inferred to be below the base of the proposed excavations, allowance should be made for drainage below the basement slab to control any seepage or rise in groundwater level. In addition, hydrostatic relief valves could be provided within the slab in case groundwater levels rise above the basement level. Alternatively, the slab could be designed to resist hydrostatic uplift pressures.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the detailed design and construction phases of the project. In the event that any of the detailed design or 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.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 129032

Client:

JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: I Squibbs

Sample log in details:

Your Reference:	**28431SB, Manly**		
No. of samples:	3 Soils		
Date samples received / completed instructions received	3/6/2015	/	3/6/2015

Analysis Details:

Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	11/06/15	/	9/06/15
Date of Preliminary Report:	Not Issued		

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Jacinta Hurst
Laboratory Manager

Misc Inorg - Soil				
Our Reference:	UNITS	129032-1	129032-2	129032-3
Your Reference	-------------	BH1	BH1	BH1
Depth	------------	0.5-0.7	1.5-1.95	3.0-3.45
Date Sampled		2/06/2015	2/06/2015	2/06/2015
Type of sample		Soil	Soil	Soil
Date prepared	-	04/06/2015	04/06/2015	04/06/2015
Date analysed	-	04/06/2015	04/06/2015	04/06/2015
pH 1:5 soil:water	pH Units	8.6	8.4	8.3
Chloride, Cl 1:5 soil:water	mg/kg	140	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	67	10	<10
Resistivity in soil*	ohm m	54	260	340

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			04/06/2015	[NT]	[NT]	LCS-1	04/06/2015
Date analysed	-			04/06/2015	[NT]	[NT]	LCS-1	04/06/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	95%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	102%
Resistivity in soil*	ohm m	1	Inorg-002	<1.0	[NT]	[NT]	[NR]	[NR]

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: LAM CONSULTING ENGINEERS
Project: PROPOSED MANLY LODGE REDEVELOPMENT
Location: 22 VICTORIA PARADE, MANLY, NSW

Job No. 28431SB
Date: 2/6/15
Method: SPIRAL AUGER
JK205
R.L. Surface: ≈ 4.8m
Datum: AHD
Logged/Checked by: I.S./D.B.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
						0			CONCRETE: 200mm.t				
	■						▨	-	FILL: Sand, fine to medium grained, brown, trace of sandstone gravel.	M			APPEARS POORLY COMPACTED
				■	N = 9 4,4,5		▨		FILL: Silty sand, fine to medium grained, dark brown, trace of sandstone gravel and root fibres.				
	■					1		SP	SAND: fine grained, light brown, trace of silt.	M	L		
									SAND: fine to medium grained, light orange brown, trace of silt.				
				■	N = 5 2,2,3								
	■					2							
	■			■	N = 6 2,3,3	3							
▶	■					4			SAND: fine to coarse grained, light brown, trace of silt.	W			
				■	N = 15 3,5,10						MD		
	■					5			as above, but medium to coarse grained.				
	■			■	N > 20 4,9, 11/100mm REFUSAL	6							
									END OF BOREHOLE AT 6.4m				
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: LAM CONSULTING ENGINEERS

Project: PROPOSED MANLY LODGE REDEVELOPMENT

Location: 22 VICTORIA PARADE, MANLY, NSW

Job No. 28431SB

Method: SPIRAL AUGER
JK205

R.L. Surface: ≈ 4.7m

Date: 2/6/15

Datum: AHD

Logged/Checked by: I.S./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
						0			CONCRETE: 150mm.t				
								-	FILL: Sand, fine to medium grained, light brown.				100mm HDPE PIPE ENCOUNTERED AT 0.19m DEPTH. BOREHOLE TERMINATED AND MOVED TO LOCATION BH2A
									END OF BOREHOLE AT 0.19m				
						1							
						2							
						3							
						4							
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2A
1/1

Client: LAM CONSULTING ENGINEERS

Project: PROPOSED MANLY LODGE REDEVELOPMENT

Location: 22 VICTORIA PARADE, MANLY, NSW

Job No. 28431SB

Date: 2/6/15

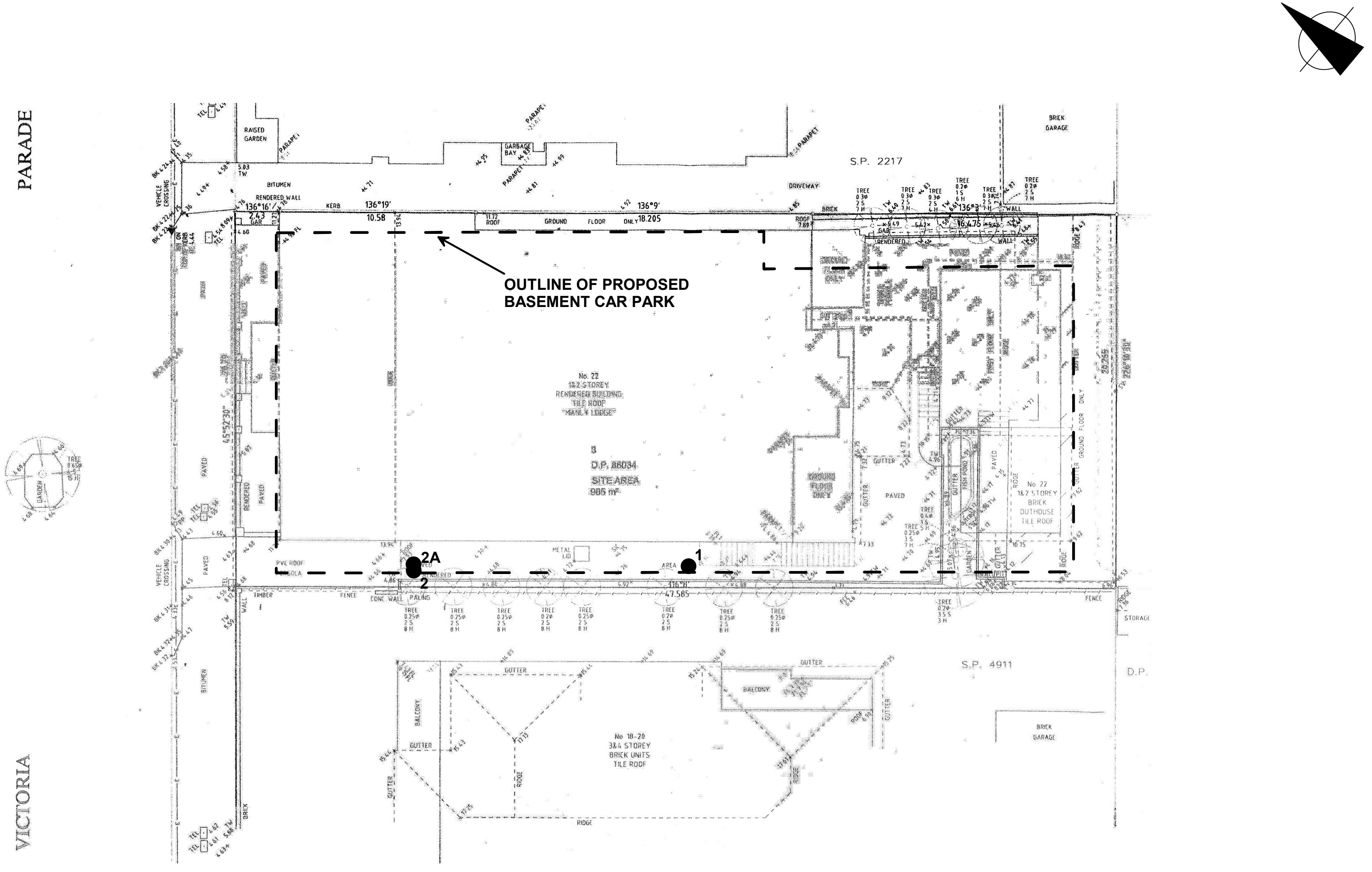
Method: SPIRAL AUGER
JK205

R.L. Surface: ≈ 4.7m

Datum: AHD

Logged/Checked by: I.S./D.B.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			CONCRETE: 200mm.t				
								-	FILL: Sand, 20mm.t				CONCRETE ENCOUNTERED AT 0.22m DEPTH. BOREHOLE TERMINATED
									END OF BOREHOLE AT 0.22m				
						1							
						2							
						3							
						4							
						5							
						6							
						7							



SCALE (m)

0 10

BOREHOLE LOCATION PLAN

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 28431SB Figure No. 1

LEGEND

- ● BOREHOLE
- ● BOREHOLE AND DCP TEST
- ⊕ DCP TEST
- ■ TEST PIT
- ■ TEST PIT AND DCP TEST
- ▼ ELETRICAL FRICTION CONE PENETRATION TEST

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'* , published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection 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 a senior geotechnical engineer.

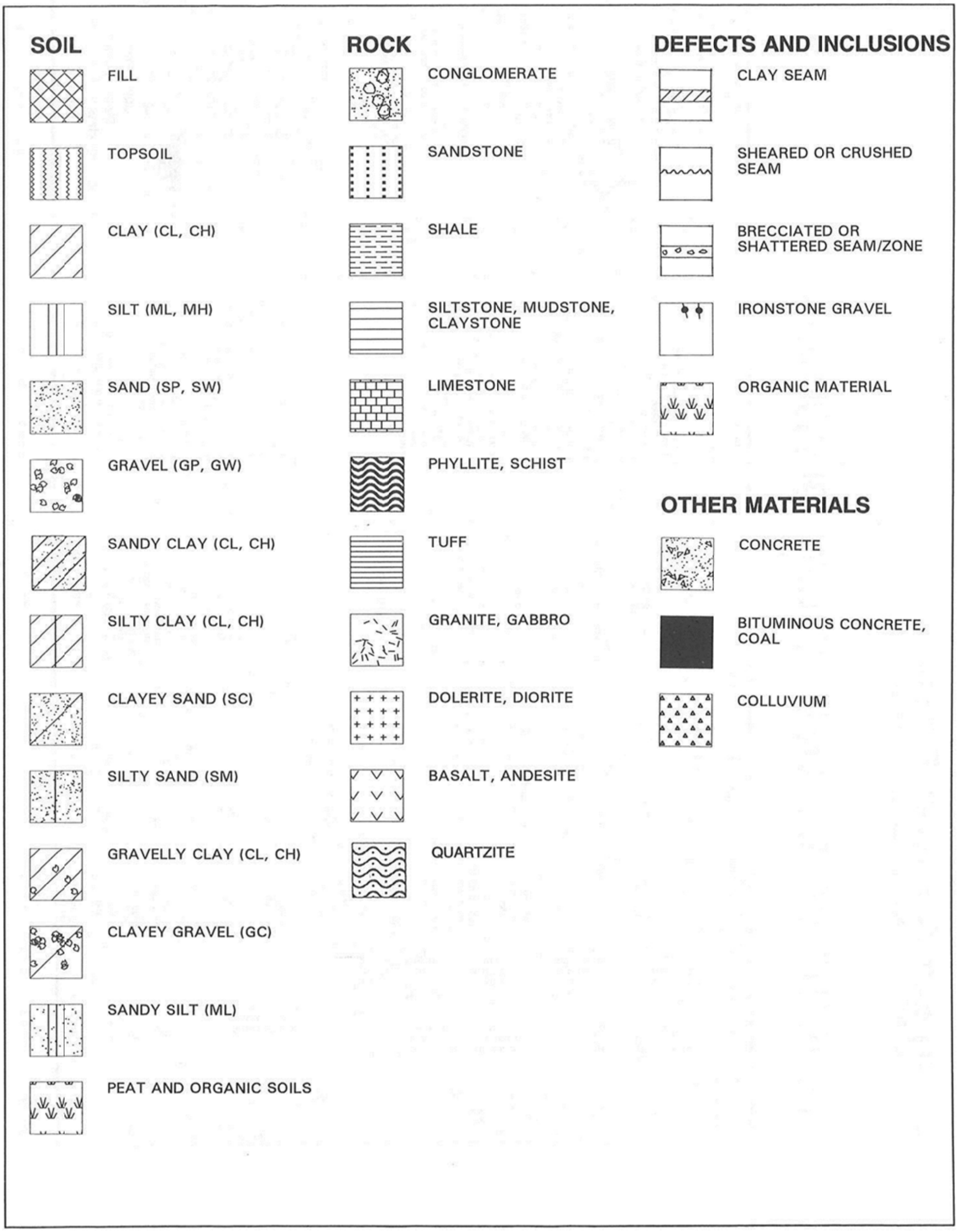
SITE INSPECTION

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

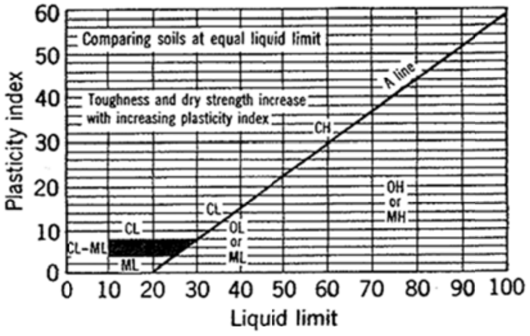
Requirements could range from:

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirements for *GW*	
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures				Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures				Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines				$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirements for *SW*	
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line with *PI* greater than 7	
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size											
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		 Plasticity chart for laboratory classification of fine grained soils		
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays					
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity					
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts					
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays					
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity					
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils					

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶—	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil 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 screening.
	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. 'R' as noted below.
	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 degree 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 (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless 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 less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
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 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense >85 >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
-----------------------	--------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
-----------------------	--------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
-----------------------	--------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
-----------------------	--------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
-----------------------	---------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
-----------------------	---------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	