REPORT

TO
KMT CONSTRUCTIONS PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
15 RYNAN AVENUE, EDMONDSON PARK, NSW

23 March 2016
Ref: 28733SBrpt

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Date: 23 March 2016
Report No: 28733SBrpt
Revision No: 0

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For and on behalf of
JK GEOTECHNICS
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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 15 Rynan Avenue, Edmondson Park, NSW. The investigation was commissioned by Mr Timothy Shiu of Joshua Farkash & Associates Pty Ltd, on behalf of KMT Constructions Pty Ltd, and was carried out in accordance with our proposal, Ref: P39889SB.

As shown in the supplied preliminary development drawings by Joshua Farkash & Associates Pty Ltd (Project No. 13-23665, Drawing Nos A-2101 to 2105, dated 10/2/16) No. 15 Rynan Avenue will be developed with the adjacent property to the north, No. 5 Rynan Avenue. Residential unit buildings are proposed within about the eastern one third of the site, surrounded by new access roads off Rynan Avenue. The new buildings will have four or five above ground levels, over one or two basement levels. Excavations for the proposed basements are expected to depths of about 3m to 6m. We assume that the new roads will be constructed at close to the existing surface levels. No development of the western about two thirds of the site is proposed and this area may be left as open space.

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

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: E28733K, for the results of the environmental assessment.

2 INVESTIGATION PROCEDURE

The geotechnical investigation was limited to one day on site for our track mounted JK305 drilling rig. In that time, BH1 to BH6 were auger drilled to depths ranging from 4.5m to 5.9m below the existing ground surface. The boreholes locations, as shown on Figure 1, were set out using a hand held GPS. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels and contours shown on the supplied survey plan by C-Side Surveyors (Ref: 140608-DET, Revision A, Sheets 1 to 3, dated 4/7/13). The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength of the residual silty clay were assessed by Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples recovered in the SPT split tube sampler. The strength of the underlying weathered shale was assessed from observation of the drilling resistance of a tungsten carbide (TC) bit attached to the augers, together with examination of the recovered rock cuttings and subsequent correlation with laboratory moisture content test results. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Groundwater observations were made during, on completion and a short time after drilling. Groundwater monitoring wells were installed in BH1, BH3 and BH5 as part of the environmental site assessment and groundwater levels measured by EIS 13 days after drilling. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Mr Arthur Billingham, 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 techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA registered laboratories, for testing to determine moisture contents, Atterberg limits, linear shrinkages, soil pH, sulphate content, chloride content and resistivity. The laboratory test results are summarised in the attached STS Table A and Envirolab Report No. 134084. 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

The site is located within relatively flat to gently sloping topography. Cabramatta Creek crosses the site about one third from the western end of the site and the site slopes down towards the creek at about 1° from both the eastern and western ends.

A single storey brick house is located in the south-eastern corner of the site, together with a number of single storey metal clad sheds to the west of the house. The house and sheds are surrounded by grassed areas, with livestock paddocks located within the western portion of the site. Numerous trees of small to moderate height are situated within the property. Vegetation is particularly thick along the banks of Cabramatta Creek. On the western side of Cabramatta Creek vegetation is initially very thick with long grasses and trees before opening up at the very western end of the site. Transmission lines cross the site towards its western edge. A small creek about 0.8m deep is located on the western boundary near Sunday Circuit and appears to run into Cabramatta Creek from a detention basin to the south-west.

To the north of the site is No. 5 Rynan Avenue, which the proposed development will extend into. No. 5 Rynan Avenue contains a two storey brick residence offset about 5m from the common boundary. To the west of the house is an inground swimming pool and sheds. The remainder of the adjoining site is grass covered, with scattered trees, similar to the subject site. The ground surface levels adjacent to the common boundary are similar to those within the subject site.

To the south of the site is a largely vacant lot with a gravel driveway running just south of the common boundary. A number of single storey buildings are situated at the western end of the driveway close to Cabramatta Creek. The ground surface levels adjacent to the common boundary are similar to those within the subject site.

3.2 Subsurface Conditions

Reference to the 1:100,000 Penrith Geological Series Sheet indicates that the site is mapped to be underlain by Bringelly Shale. The boreholes encountered fill covering residual silty clays and gravelly clay grading into weathered shale. Further comments on the subsurface conditions encountered are provided below. A graphical summary of the borehole information is presented as Figure 2.

Fill

Fill was encountered in all boreholes to depths ranging from 0.4m to 0.9m and comprised silty clay with varying proportions of ash, brick fragments, ironstone gravel and sand. Based on the SPT ‘N’ values the fill was assessed to be poorly compacted.

Residual Clays

The residual clays initially comprised silty clay with gravelly clay containing ironstone gravel encountered with depth. The clays were assessed to be of medium plasticity and generally of stiff to very stiff strength. However, some firm to stiff clays were encountered in BH5 and BH6.

Weathered Shale

Weathered shale was encountered at depths ranging from 2.0m to 4.2m. The shale was initially assessed to be extremely weathered and of extremely low strength, becoming distinctly weathered and low strength with depth. In BH1, BH3 and BH5, the deeper shale was assessed to be of medium to high strength.

Groundwater

Groundwater seepage was encountered during the drilling of BH2 at a depth of 2.6m. Groundwater was measured on completion of BH2 to BH5 at depths ranging from 1.5m to 5.2m. Within the wells installed in BH1, BH3 and BH5 groundwater was measured 13 days after drilling at depths of 2.4m, 1.3m and 1.3m, respectively.

3.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the clays tested are of medium plasticity and are assessed to have a low to moderate potential for shrink/swell movements with changes in moisture content.

The moisture content test results on samples of the shale showed reasonably good correlation with our field assessment of rock strength.

The soil pH values ranged from 8.2 to 9.3, indicating alkaline soil conditions. The sulphate and chloride contents and resistivity values were found to be low. Based on the results the soils would be classified as ‘non-aggressive’ exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 ‘Piling – Design and Installation’. For steel piles, the soils would be classified as ‘non-aggressive’ to ‘mild’ in accordance with Table 6.5.2(C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues and Further Geotechnical Investigations

The boreholes drilled for this investigation show that the subsurface conditions comprise surface fill covering residual silty clay grading into weathered shale. Excavations for the one or two basements will encounter the shale and therefore, building footings should be founded within the shale. Temporary batters may be suitable where excavations are limited, but retention systems installed prior to excavation will be required for the deeper excavation.

The boreholes drilled for this investigation were only auger drilled and terminated above the base of any excavations for two basements. In addition, the proposed development extends into the adjoining property to the north, No. 5 Rynan Avenue. We previously prepared a geotechnical assessment report for No. 5 dated 10 September 2014 (Ref: 27532SBrpt), but this was based on environmental boreholes and no geotechnical boreholes have been drilled within No. 5. Therefore, we recommend that to allow detailed design of the proposed structure additional geotechnical boreholes be drilled within the footprint of the proposed buildings, within both No 5 and No. 15. These boreholes should involve core drilling of the shale in order to optimise bearing pressures for the design of footings and so that the boreholes can penetrate below the base of the proposed excavation. Boreholes should also be drilled along the alignment of the proposed roads to allow sampling of the subgrade soils for CBR testing.

Preliminary comments and recommendations are provided within the following sections of this report and these may be used for planning and preliminary design. The comments and recommendations provided herein should be confirmed and amplified as part of the additional geotechnical investigation.

4.2 Excavation and Groundwater

Excavation to the required depths of 3m to 6m will encounter surface fill, residual clays and weathered shale, possibly up to high strength. Excavation of the soils and possibly the upper shale of extremely low strength will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators.

Excavation of the shale of low strength or higher strength will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. The use of hydraulic rock hammers will depend on the nature of any developments adjacent to the site at the time of excavation. At present existing structures are well away from the area of the proposed excavation, but if at the time of excavation structures have been built close to the excavation then the vibrations transmitted by hydraulic rock hammers may need to be monitored. Excavation using rock hammers should commence away from any structures and the vibrations transmitted to the structures monitored to assess how close the hammer can operate to the structures while maintaining transmitted vibrations within acceptable limits. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are unacceptable it would be necessary to change to alternative excavation equipment, such as ripping hooks, rotary grinders or rock saws. The final monitoring required should be assessed at the time of excavation.

Groundwater was encountered above the base of the excavation and should be allowed for during excavation. Given the expected low permeability of the clays and weathered shale such seepage should be able to be controlled using conventional sump and pump techniques. Some areas of higher permeability may be encountered within the more gravelly clays where seepage may be greater. In the long term, drainage should be provided behind all retaining walls and below the basement slabs. Collected seepage should be directed to sumps containing automatic and failsafe pump systems to remove water from the basements. The final extent of the drainage systems should be determined following completion of the excavation when the hydraulic consultant can assess the actual seepage flows.

Approval for drained basements and discharge of the collected seepage may need to be obtained from relevant authorities. If drained basements and discharge of the seepage is not allowed the basements would then need to be designed as tanked basements to resist the hydrostatic uplift pressures.

4.3 Retention

For the shallower excavations of less than about 3m temporary batters may be possible. However, for the deeper basement excavations or where insufficient space is available for batters retention systems will need to be installed prior to the start of excavation.

Temporary batters of no more than about 3m in height should be no steeper than 1 Vertical in 1 Horizontal (1V:1H), but if groundwater seepage is encountered then flatter batters or temporary stabilisation works may be required. Such batters should remain stable in the short term provide all surcharge loads, including construction loads, are kept well clear of the crest of the batters.

Permanent batters, if required, 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. Permanent batters should be covered with topsoil and planted with deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should be directed away from all temporary and permanent slopes to also reduce erosion.

Retaining walls of no more than 3.5m in height may be designed as cantilevered walls, provided structures or movement sensitive services are located a horizontal distance from the wall equal to at least twice the wall height. Such walls may be provisionally designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of $20kN/m^3$.

For excavations greater than about 3m in depth or where temporary batters cannot be accommodated, full depth retention system will need to be installed prior to the start of excavation. Where movements will not be critical, such as where structures or movement sensitive services are located more than twice the wall height behind the wall, soldier pile walls with shotcrete infill panels would be appropriate. Where movements are to be kept low, such as where structures or movement sensitive services are located within a horizontal distance of twice the wall height behind the wall, more rigid walls will be required, such as closely spaced soldier piles or contiguous pile walls.

Lateral restraint of the retaining walls will be required in the form of external anchors or internal props, which must be installed progressively as each restraining point is uncovered. Where anchors extend below adjoining properties permission will need to be obtained from the owners of those properties prior to the installation of the anchors. Such permission can take some time to obtain and this should be allowed for in the project program. Long term lateral support will be provided by the floor slabs inside the excavation.

Propped or anchored retaining walls may be provisionally designed based on a trapezoidal earth pressure distribution of magnitude 6H kPa, where H is the retained height in metres, where structures or movement sensitive services are located beyond a horizontal distance of 2H of the wall. Where structures or movement sensitive services are located within 2H of the wall a trapezoidal earth pressure distribution of 8H kPa should be used. These maximum pressures should be held constant for the central 50% of the trapezoidal pressure distribution.

The above coefficients and lateral pressures assume horizontal backfill surfaces and where inclined backfill is proposed it will need to be 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.

Anchors should have their bond formed within shale of at least low strength, outside of a line drawn up at 45° from the base of the wall. Provisional design of the anchors may be based on an allowable bond stress of 200kPa for shale of at least low strength. Anchors should have a minimum free length of 3m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of their design load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their design loads. Higher bond stress values may be appropriate within the higher strength shale, but this would need to be assess following the drilling of cored boreholes. Generally anchors are installed on a design and construct basis so that the optimisation of bond stress does not become a contractual issue in the event of an anchor failing the test load.

Passive toe resistance of the retention system below the base of the bulk excavation may be estimated based on a maximum allowable lateral resistance of 200kPa for shale of at least low strength. The passive resistance should be ignored for at least 0.5m below the base of the excavation, including excavations for services and footings.

4.4 Footings

Since shale will be encountered within the basement excavations all structures should be supported on footings founded within the shale to provide uniform support and reduce the risk of differential settlements. Where shale is exposed, or is at shallow depths of less than say 1m, pad or strip footings may be used. Where shale is at greater depths bored piers would be more practical.

Footings founded within shale of extremely low strength may be designed based on an allowable bearing pressure of 700kPa. Where footings are founded within shale of low strength an allowable bearing pressure of 1200kPa may be used. Higher bearing pressures would be possible within shale of medium or high strength, but this would need to be assessed from cored boreholes drilled below the base of the proposed excavation.

If portions of the buildings extend outside of the footprint of the basement piers will be required so that the entire structure is supported on footings founded within the shale. These piers should be founded below the zone of influence of the basement retaining walls, which may be taken as a line drawn up at 1V:1H from the base of the walls.

4.5 Subgrade Preparation for Pavements

For construction of the proposed surface roads around the proposed buildings subgrade preparation works should initially comprise stripping of vegetation and all root affected soils. Following stripping and excavation to the design subgrade level, 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 or unstable subgrade areas. Any weak areas detected should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during proof rolling.

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 clay and shale may be reused as engineered fill, provided it is free of deleterious materials and particles in excess of 75mm in size. Such 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. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

Testing of the subgrade soils should be carried out as part of the further geotechnical investigation of the site to assess the design CBR for the pavements. For the residual silty clay we would expect CBR values of about 2% to 3%, but this must be confirmed by site specific testing.

Surface and subsoil drainage should be provided on both sides of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 300mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain. In addition, the surface of the adjacent pavement subgrade should be provided with a uniform cross fall towards the subsoil drain to assist with drainage.

Concrete pavements should have a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2013) 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 detailed design and construction phase of the project. In the event that any of the detailed design and 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 Residential Development
Location: 15 Rynan Avenue, Edmondson Park, NSW

Ref No: 28733SB
Report: A
Report Date: 21/09/2015
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	4.00-4.30	11.0				
2	1.50-1.95	18.8	36	16	20	7.0
3	4.00-4.40	8.5				
3	4.90-5.20	3.7				
4	3.50-3.80	12.0				
5	5.50-5.80	7.4				
6	1.50-1.95	17.2	34	14	20	5.5
6	4.20-4.50	6.7				

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: 10/09/2015

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 134084

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Arthur Billingham

Sample log in details:
Your Reference: **28733SB, Edmondson Park**
No. of samples: 3 Soils
Date samples received / completed instructions received 10/09/2015 / 10/09/2015

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: 17/09/15 / 17/09/15
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	134084-1	134084-2	134084-3
Your Reference	-------------	BH2	BH3	BH5
Depth	------------	0.5-0.9	0.5-0.95	1.5-1.95
Date Sampled		9/09/2015	9/09/2015	9/09/2015
Type of sample		Soil	Soil	Soil
Date prepared	-	11/09/2015	11/09/2015	11/09/2015
Date analysed	-	14/09/2015	14/09/2015	14/09/2015
pH 1:5 soil:water	pH Units	8.4	8.2	9.3
Chloride, Cl 1:5 soil:water	mg/kg	<10	170	540
Sulphate, SO4 1:5 soil:water	mg/kg	40	35	260
Resistivity in soil*	ohm m	64	36	16

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 ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			11/09/2015	[NT]	[NT]	LCS-1	11/09/2015
Date analysed	-			14/09/2015	[NT]	[NT]	LCS-1	14/09/2015
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	103%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	97%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	LCS-1	106%
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: KMT CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB
Date: 9-9-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: A.B./D.B.
R.L. Surface: ≈ 42.5m
Datum: AHD

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 clay, medium plasticity, brown.	MC>PL			APPEARS POORLY COMPACTED
		N = 10 2,5,5				FILL: Silty clay, low plasticity, light brown, trace of ash and fine grained sand.				
					CL	SILTY CLAY: medium plasticity, red brown, trace of ash.	MC>PL	St-VSt	200 180 120	RESIDUAL
			1							
		N = 16 6,7,9							220 130 210	
			2		-	SHALE: light grey and grey, with iron indurated bands and clay seams.	XW	EL		VERY LOW TO LOW 'TC' BIT RESISTANCE
▼ 22/9/15										
		N = SPT 11/70mm REFUSAL	3			SHALE: dark grey, with iron indurated bands.	DW	VL-L		LOW RESISTANCE WITH MODERATE BANDS
			4			SHALE: dark grey.		L		MODERATE RESISTANCE
			5					M-H		HIGH RESISTANCE
								H		
						END OF BOREHOLE AT 5.6m				MONITORING WELL INSTALLED TO 4.8m DEPTH, SLOTTED FROM 4.8m TO 1.8m, SAND FILTER FROM 4.8m TO 1.6m, BENTONITE SEAL, FINISHED WITH GATIC COVER AND LOCKABLE CAP
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: KMT CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB
Date: 9-9-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: A.B./D.B.
R.L. Surface: ≈ 41.6m
Datum: AHD

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			FILL: Silty clay, low plasticity, with fine to medium grained igneous gravel, fine to coarse grained brick fragments, trace of ash and fine grained sand.	MC≈PL			APPEARS POORLY COMPACTED
				■	N = 1 2,0,1								
AFTER 4 HRS ▼	■					1		CL	SILTY CLAY: medium plasticity, orange brown, trace of ash and fine grained ironstone gravel.	MC>PL	St	100 190	RESIDUAL
▼ ON COMPLETION	■			■	N = 11 5,5,6						St-VSt	290 220 190	
						2							
▶													
									GRAVELLY CLAY: medium plasticity, orange brown, fine to medium grained ironstone gravel.				
	■			■	N = 12 6,6,6	3							
								-	SHALE: dark grey and grey, with iron indurated bands.	XW	EL-VL		
	■			■		4							LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
										DW	L		MODERATE RESISTANCE
									END OF BOREHOLE AT 4.7m				
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: KMT CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB
Date: 9-9-15
Method: SPIRAL AUGER
JK305
R.L. Surface: ≈ 40.6m
Datum: AHD
Logged/Checked by: A.B./D.B.

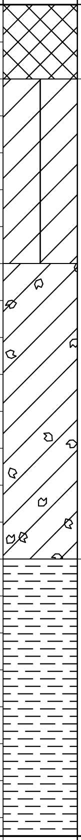
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	■					0			FILL: Silty clay, medium plasticity, dark grey brown, trace of ash.	MC>PL			GRASS COVER
	■			■	N = 14 5,6,8			CL	SILTY CLAY: medium plasticity, grey brown, with fine grained ironstone gravel.	MC>PL	VSt	360 400 280	RESIDUAL
▼ 22/9/15						1			SILTY CLAY: medium plasticity, grey mottled orange brown, with fine to medium grained ironstone gravel.				
	■			■	N = 11 5,5,6						St- VSt	130 120 210	
						2			GRAVELLY CLAY: medium plasticity, orange brown, fine to coarse grained ironstone gravel.				
	■												
				■	N = 22 8,9,13	3							
								-	SHALE: light grey and dark grey.	DW	VL		
				■		4					L-M		MODERATE 'TC' BIT RESISTANCE BANDS
	■										M		MODERATE TO HIGH RESISTANCE
ON COMPLETION ▼				■		5							
				■							H		HIGH RESISTANCE
						6 7			END OF BOREHOLE AT 5.38m				MONITORING WELL INSTALLED TO 5.38m DEPTH, SLOTTED FROM 5.38m TO 2.38m, SAND FILTER FROM 5.38m TO 1.6m, BENTONITE SEAL, FINISHED WITH LOCKABLE CAP

BOREHOLE LOG

Borehole No.
4
1/1

Client: KMT CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB
Date: 9-9-15
Method: SPIRAL AUGER
JK305
R.L. Surface: ≈ 42.0m
Datum: AHD
Logged/Checked by: A.B./D.B.



Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	■					0		FILL: Silty clay, low plasticity, dark grey brown, trace of fine grained ironstone gravel and root fibres.	MC>PL			GRASS COVER
	■ ■			■	N = 13 5,7,6	1	CL	SILTY CLAY: medium plasticity, red brown, with fine grained ironstone gravel.	MC>PL	VSt	410 420 400	RESIDUAL
	■			■	N = 14 9,7,7	2		GRAVELLY CLAY: medium plasticity, red brown, fine to coarse grained ironstone gravel.	MC≈PL			
	■					3	-	SHALE: grey, with iron indurated bands.	XW-DW	EL-VL		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
ON COMPLET-ION ▼				■ ■		4		SHALE: dark grey, with iron indurated bands.	DW	L-M		MODERATE RESISTANCE
						5 6 7		END OF BOREHOLE AT 4.5m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: KMT CONSTRUCTIONS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB
Date: 9-9-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: A.B./D.B.
R.L. Surface: ≈ 41.1m
Datum: AHD

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	▩		FILL: Silty clay, low plasticity, dark grey brown, trace of fine grained ironstone and igneous gravel, medium grained sand and ash.	MC>PL			GRASS COVER
22/9/15 ▼ ▼ ON COMPLET-ION	■ ■			■ ■	N = 12 4,6,6 N = 9 3,4,5	1 2	▨	CL	SILTY CLAY: medium plasticity, red brown, trace of fine grained ironstone gravel.	MC>PL	St-VSt F-St	170 230 330 80 70 100	RESIDUAL
	■			■	N = 19 8,9,10	3	▨		SILTY CLAY: low plasticity, grey mottled orange brown.			140	
						4	▨		GRAVELLY CLAY: low plasticity, red brown, fine grained ironstone gravel.	MC<PL	(St)	90	MONITORING WELL INSTALLED TO 5.9m DEPTH, SLOTTED FROM 5.9m TO 2.9m, SAND FILTER FROM 5.9m TO 0.5m, BENTONITE SEAL, FINISHED WITH LOCKABLE CAP
	■ ■			■ ■		5	▤	-	SHALE: grey and dark grey, with iron indurated bands.	XW DW	EL VL-L L-M H		LOW 'TC' BIT RESISTANCE LOW TO MODERATE RESISTANCE HIGH RESISTANCE
						6 7			END OF BOREHOLE AT 5.9m				'TC' BIT REFUSAL

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
6
1/1

Client: KMT CONSTRUCTIONS PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 15 RYNAN AVENUE, EDMONDSON PARK, NSW

Job No. 28733SB

Date: 9-9-15

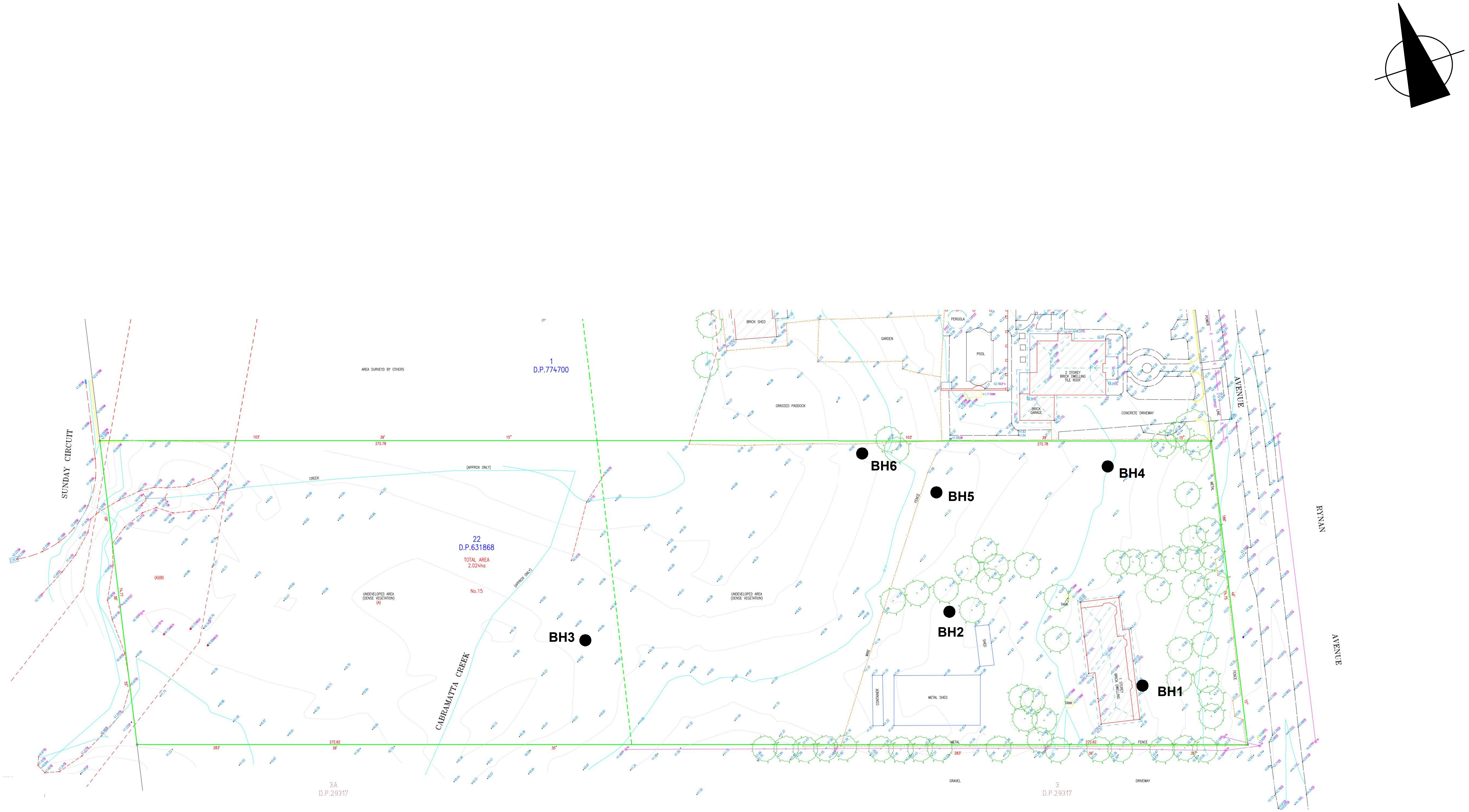
Method: SPIRAL AUGER
JK305

R.L. Surface: ≈ 40.7m

Datum: AHD

Logged/Checked by: A.B./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
DRY ON COMPLETION	■					0			FILL: Silty clay, low plasticity, dark grey brown, with ash, trace of fine grained igneous gravel and medium grained sand.	MC>PL			
	■			■	N = 12 4,6,6	1		CL	SILTY CLAY: medium plasticity, yellow brown, trace of fine grained ironstone gravel and root fibres.	MC>PL	VSt	260 210 310	RESIDUAL
											F-St		
	■			■	N = 9 3,4,5	2						110 80 80	
▼ AFTER 1.5 HRS													
									GRAVELLY CLAY: medium plasticity, grey and orange brown, fine to coarse grained ironstone gravel.		(St)		
	■			■	N = 23 8,10,13	3							
						4		-	SHALE: grey, with iron indurated bands.	XW	EL-VL		LOW 'TC' BIT RESISTANCE
	■			■						DW	L		LOW RESISTANCE WITH MODERATE BANDS
						5			END OF BOREHOLE AT 4.5m				
						6							
						7							



Scale (m):

0 50

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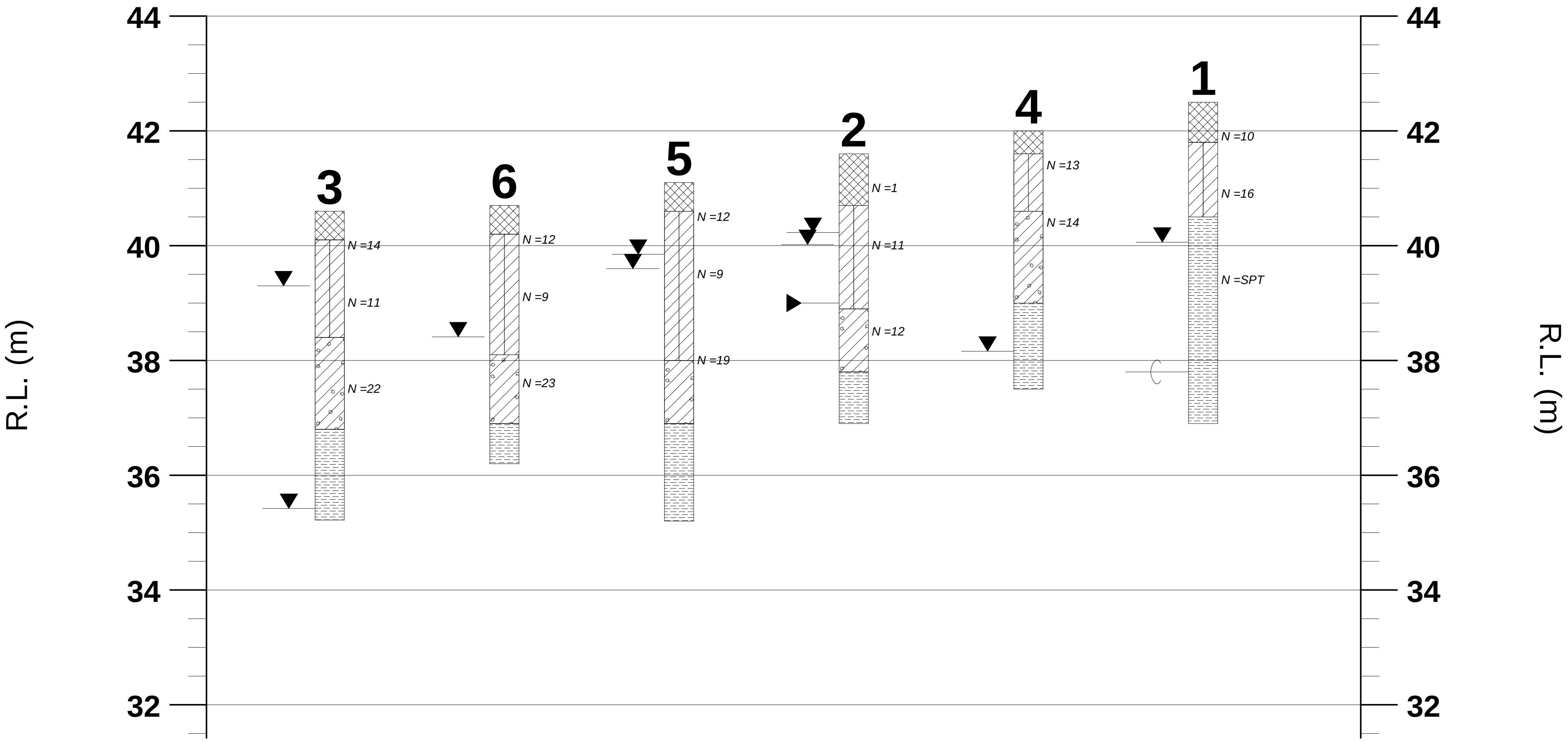
Title:

BOREHOLE LOCATION PLAN

Report Number:	Figure Number:
28733SB	1

GRAPHICAL BOREHOLE SUMMARY



▩	Fill	▤	Shale	—c—	Borehole Collapse Depth
▨	Silty Clay	▼	Observed water level	N	SPT "N" VALUE
▨	Gravelly clay	▶	Groundwater seepage level	Nc	SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 28733SB

Figure No.: 2

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.

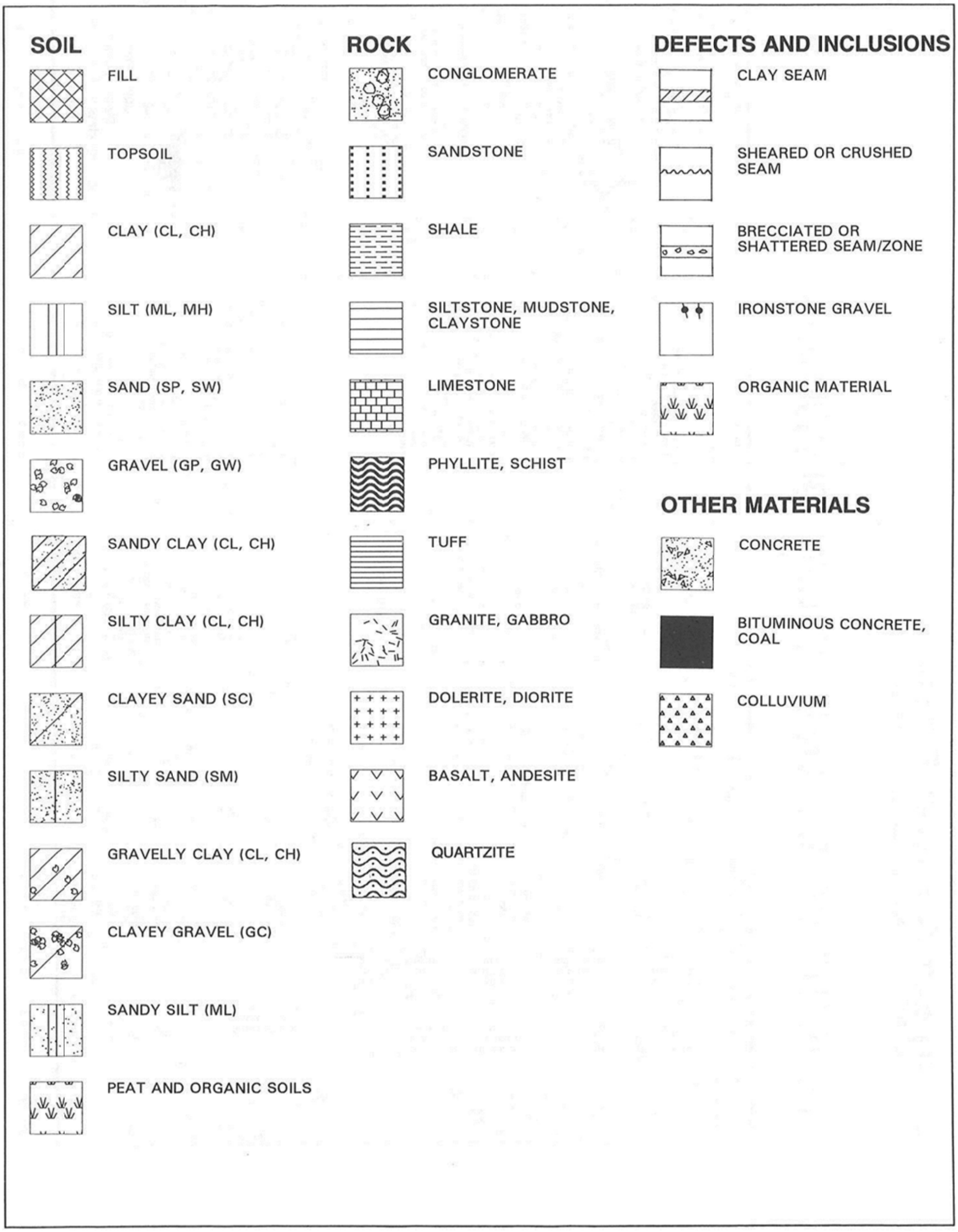
Requirements could range from:

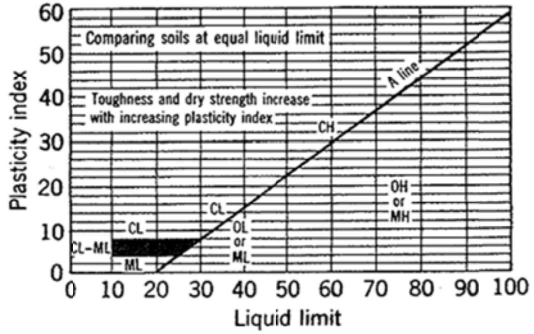
i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

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



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									 Plasticity chart for laboratory classification of fine grained soils		
	Silts and clays liquid limit less than 50		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
			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*)				
			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	