REPORT

TO
WATERMARK CHATSWOOD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED REDEVELOPMENT OF CHATSWOOD GOLF CLUB

AT
BEACONSFIELD ROAD, CHATSWOOD, NSW

12 May 2017
Ref: 27168Zrpt Rev1

JK Geotechnics
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For and on behalf of
JK GEOTECHNICS
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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for proposed redevelopment of Chatswood Golf Club ("the Club"), Beaconfield Road, Chatswood, NSW. A site plan is included as Figure 1. The investigation was commissioned by Mr Tony Olding of Watermark Chatswood via return of 'Acceptance of Proposal Form' dated 23 November 2016. The commission was on the basis of our proposal (Ref P43884Z Chatswood) dated 21 November 2016.

To assist with the geotechnical investigation, we have been supplied with the following information:

1. Survey plan of the site and its immediate surrounds, prepared by Mitchell Land Surveyors Pty Ltd (Dwg No. 1066AE, Sheets 1/3 to 3/3, Revision 00, dated 9 December 2013).
2. Architectural concept design plans by Marchese Partners (Drawing Nos 01 to 09).

Based on the supplied information, we understand that the redevelopment will extend as a relatively narrow curved north to south trending building along the western end of the site. It will include a three to four storey Club House towards the southern end, with the lowest finished floor (Level 02) at Reduced Level (RL) 27.0m. Five levels of Independent Living Units (ILUs) will extend southwards from the Club House with their lowest finished floor level (Level 02) at RL28.4m. Five and four levels of ILUs will extend northwards from the Club House with their lowest finished floor levels (Levels 01 and 02) at RL24.4m and RL27.5m, respectively. Parking along the eastern side of the building will extend between Level 03 and Level 05 with driveway access off Beaconsfield Road to Level 05. The proposed development will be excavated into the hillside and a maximum vertical excavation depth up to approximately 15m has been estimated. We have assumed that typical structural loads for this type of building apply.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on site stability, excavation conditions and support, retaining walls, footings, on-grade floor slabs, and external pavements.

Environmental Investigation Services (EIS) [the environmental consulting division of the JK Group] have completed two previous Environmental Site Assessments at the site, and the results were presented in their reports (Ref E27168Klet) dated 12 February 2014 and (Ref E27168KDrpt) dated 30 August 2016. EIS have also undertaken sampling concurrently with this geotechnical investigation, reporting for which will be contained in their report (Ref E27168KF). This report must be read in conjunction with the current and previous EIS reports.

2 INVESTIGATION PROCEDURE

Prior to the commencement of the fieldwork, a ‘Dial Before you Dig’ search was undertaken and the borehole locations were electromagnetically scanned by a specialist subcontractor for buried services.

The fieldwork was carried out on 30 and 31 January 2017 and comprised the drilling of five boreholes (BH101 to BH105), using our track mounted JK308 drilling rig. The boreholes were auger drilled to depths between 0.65m and 4.0m. BH101, BH102 and BH103 were then extended into the underlying bedrock using rotary diamond coring techniques with an NMLC triple tube core barrel and water flush to final depths of 11.75m, 8.77m, and 11.50m, respectively.

The borehole locations, as shown on attached Figure 2, were to some extent dictated by access constraints, and were set out by taped measurements from existing surface features. The surface RLs shown on the attached borehole logs were interpolated between spot level heights and ground contours shown on the survey plans and are therefore only approximate. The survey datum is the Australian Height Datum (AHD). Figure 2 is based on the supplied survey plan.

The nature and composition of the subsurface soil and rock horizons were assessed by logging the recovered materials during drilling. The relative compaction of the subsoil profile (fill) was assessed from the Standard Penetration Test (SPT) ‘N’ values. The strength of the augered upper bedrock profile was assessed by observation of the auger resistance to penetration when using a tungsten carbide (‘TC’) bit, together with examination of recovered rock cuttings and correlation with subsequent moisture content tests. The strength of the cored bedrock was assessed by examination of the recovered rock core, together with correlation with subsequent laboratory Point Load Strength Index (I_{S50}) tests. Groundwater observations were made in each borehole during the fieldwork. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineer was present on a full-time basis during the fieldwork to set out the borehole locations, direct the electromagnetic scanning, nominate the testing and sampling and prepare the attached borehole logs. The Report Explanation Notes define the logging terms and symbols used.

Selected rock chip samples were returned to a NATA registered laboratory (Soil Test Services [STS]) for moisture content testing. The test results are summarised in STS Table A.

The recovered rock cores were also returned to STS for photographing and I_{S50} testing. The photographs are enclosed accompanying the borehole logs. The results of the laboratory I_{S50} testing are plotted on the borehole logs and are also summarised in the attached STS Table B. The Unconfined Compressive Strengths (UCSs), as estimated from the Point Load Strength Index test result, are also summarised in Table B.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located over and behind the crest of a spur which extends toward the west.

The site generally falls toward the west at slopes of around 8º to 10º, however significantly steeper slopes are present (up to about 30º) to the west and particularly to the south-west of the site. To the north, west and south beyond the toe of the spur, the site is surrounded by golf course landscaping. To the east, residential properties abut the site boundaries.

At the time of the investigation, the site contained a two tiered asphaltic concrete (AC) surfaced carpark (large open area near the crest of the hill, and thin strip mid-slope). A two storey brick and concrete golf club building (which appeared to be in good condition) and a metal green keepers shed on dry packed irregular sandstone block footings were located to the south-west.

Outcrops of sandstone bedrock were visible adjacent to the uphill (eastern) boundary of the site, on the vegetated slope between the two carparks and at the toe of the slope below the carparks. The sandstone bedrock outcrops generally appeared to be massive in structure, distinctly weathered at the surface, and of at least low to medium strength. The remainder of the site was heavily vegetated, containing medium to large sized trees and smaller shrubs.

Properties to the east of the site typically contained two storey brick dwellings, offset at least 10m from the site boundary. The neighbouring dwellings appeared to be in good condition based on a cursory inspection from within the site.

3.2 Subsurface Conditions

With reference to the 1:100,000 geological map of Sydney, the site is underlain by Hawkesbury Sandstone. The presence of sandstone was confirmed by the outcrops present on site.

In summary, the boreholes have disclosed a generalised subsurface profile comprising fill overlying weathered sandstone bedrock at shallow depth. Reference should be made to the attached borehole logs for specific details at each location.

A summary of the subsurface conditions is presented below:

Pavements

A 30mm thick AC wearing surface was encountered at the surface of BH103 and BH105.

Fill

Fill was encountered below the pavements in BH103 and BH105 and from the ground surface in the remaining boreholes. The fill extended down to depths between 0.1m (BH103) and 2.1m (BH104) below existing grade and predominantly comprised granular materials. However, in BH104, the fill materials were predominantly clayey and moderately compacted near the surface, but poorly compacted and over-wet below a depth of 1.2m. Inclusions of sandstone gravel, igneous gravel and concrete fragments were present in the fill. Where tested, the fill was assessed to be poorly to moderately compacted.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered below the fill each borehole at depths between 0.1m (BH103) and 2.1m (BH104) and extended down to the borehole termination depths.

The weathered sandstone bedrock profile was generally distinctly or slightly weathered and of medium or high strength. We note, however, that in BH103 the bedrock was extremely weathered and of extremely low strength between depths of 0.3m and 1.1m. Between 1.1m and 5.2m depth, the sandstone bedrock in BH103 improved to distinctly weathered and low to medium strength. Further, BH103 also contained a half metre section of core loss, from a depth of about 2.2m below the existing ground surface level. We infer that this may have been a band of clay or extremely weathered rock that was washed away by the coring process.

The cored portions of the bedrock contained occasional sub-horizontal defects including thin extremely weathered seams/bands, clay seams and bedding partings. Occasional inclined joints were also encountered in BH102 and BH103.

An indicative engineering classification of the bedrock, in accordance with Pells *et al* (1998) has been carried out and is tabulated below:

Borehole	Approx Surface RL (m) AHD	Indicative Engineering Classification of Bedrock Depths (m)				
		Class V	Class IV	Class III	Class II	Class I
BH101	43.9	–	–	7.7 – 8.5	1.2 – 7.7 8.5 – 11.8	–
BH102	32.1	–	–	0.45 – 1.9 3.8 – 4.4	1.9 – 3.8 4.4 – 8.8	–
BH103	35.7	0.1 – 1.2 2.2 – 2.8	0.1 – 0.3 1.2 – 2.2 5.4 – 7.5	2.8 – 5.4	1.2 – 2.2 7.5 – 0.5	
BH104*	18.0	–	–	2.1 – 3.0	3.0 – 4.0	–
BH105*	23.5	–	–	–	0.7 – 1.0	–

* Note that the indicative rock classification at BH104 and BH105 are inferred from auger drilling. Rock classification at these locations has not been carried out based on Pells *et al* (1998).

Groundwater

All boreholes were 'dry' during auger drilling and on completion of auger drilling. Groundwater measurements on completion of coring are often influenced by the addition of water flush associated with the coring process, and are therefore not representative of true groundwater conditions. There was generally a full return of the drill flush water, which indicates a relatively impermeable rock mass, however, we note that a 20% return was estimated below a depth of 2.5m at the location of BH103, which indicates the possible presence of an open defect, or deflects, in the rock mass.

3.3 Laboratory Test Results

Laboratory moisture content and Point Load Strength Index tests correlated well with our field assessment of the sandstone bedrock. The Unconfined Compressive Strength (UCS) of the rock core, as estimated from the Point Load Strength Index test results, indicated values between 4MPa and 40MPa, but generally in the range 12MPa to 26MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The primary geotechnical issues associated with the proposed redevelopment will be to maintain stability to the adjoining residential properties and buried services and reduce the likelihood of vibration induced damage to nearby buildings and structures.

The above geotechnical issues are addressed in the following sections of this report.

The geotechnical investigation has provided a basis for the comments and recommendations which follow. However, once the architectural drawings have been finalised, we recommend that the comments and recommendations which follow be reviewed and revised, if appropriate, so that the specific details of the proposed development are addressed. Irrespective, it will be essential during excavation and construction work that frequent geotechnical inspections are carried out to assess exposed subsurface conditions, so as to provide appropriate geotechnical advice.

4.2 Excavation

The excavation recommendations provided below should be complemented by reference to the Safe Work Australia *'Code of Practice – Excavation Work'*.

4.2.1 Dilapidation Surveys

Prior to the commencement of excavation, we recommend that detailed dilapidation reports be compiled on the adjacent residential properties to the east, and on the pavement surface of Beaconsfield Road.

Dilapidation surveys should include detailed inspections, where all defects are vigorously described (including defect type, length and width) and photographed.

The respective owners should be asked to confirm that the reports present a fair record of existing conditions. The dilapidation reports may be used as a benchmark against which to assess possible future claims for damage arising from the works. The excavation procedures and dilapidation reports should be carefully reviewed prior to excavation commencing so that appropriate equipment is used. We could prepare a fee proposal to carry out the dilapidation surveys, if requested.

4.2.2 Site Preparation

The site preparation works will comprise demolition of the existing structures on site as well as removal of plants and trees, including their root balls. All grass, topsoil, root affected soils and any deleterious or contaminated existing fill should also be stripped. Reference should be made to the EIS reports for guidance on the offsite disposal of soil.

4.2.3 Excavation Methods

Based on the investigation results, the proposed bulk excavation will encounter a shallow soil profile and extend to significant depths into the sandstone bedrock, of variable but generally medium and high strength.

Excavation of the soil and extremely weathered bedrock profiles can be completed using large hydraulic excavators. It may be possible to remove the upper very low to low strength sandstone bedrock, such as encountered in BH103 using a 'digging' bucket fitted to a very large excavator, however, ripping tyne and/or rock hammer assistance may also be required.

During bulk excavations, we expect that the excavation of the medium and higher strength bedrock will present 'hard rock' excavation conditions. Ripping of Class II sandstone bedrock may be possible with a Caterpillar D10 dozer or equivalent, however to improve excavation production rates, a very generous allowance should be made for rock hammer assistance to the ripping. Excavation production rates are likely to be very low and shoe wear rates high, particularly in the more competent bedrock. Grid sawing the sandstone bedrock in conjunction with ripping and/or hammering would also help to facilitate excavation.

For detailed excavations below bulk level, eg. for footings, trenches, lift pits etc., we suggest that the perimeter of the proposed excavation be saw cut and hydraulic hammers or ripping tynes be used for breaking up the bedrock. Dust suppression by spraying with water should be carried out whenever rock saws are used.

Particular care is required during bulk excavation with respect to equipment and personnel working at the crest of the spur slope and materials rolling downhill. We recommend that a geotechnical engineer progressively inspects the slope below the excavated level and that a minimum 10m wide fenced off exclusion zone be installed beyond the toe of the slope. The surface of the exclusion zone should be covered with at least 0.4m of loose, sandy fill to act as an energy absorber for rock fragments, blocks, etc, which do fall. Alternatively, a narrower exclusion zone with an engineer designed catch fence should be installed.

4.2.4 Excavation Techniques and Vibrations

Rock excavations will need to be strictly controlled as there will probably be direct transmission of ground vibrations to nearby structures and buried services. We therefore recommend that continuous quantitative vibration monitoring be carried out during rock excavation on this site.

The monitoring must include the installation of vibration monitors (equipped with data loggers which provide graphical presentation of vibration velocity versus vibration frequency) which measure transverse, vertical and longitudinal ground vibrations and their vector sum. The monitors must be installed along the eastern site boundary.

By referencing the relevant German Standard DIN4150-3:1999-02, the vibrations on the closest nearby houses should be limited to a peak particle velocity of 5mm/sec, subject to review of the dilapidation survey reports. Should the vibration limits be exceeded, they should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then alternative techniques will need to be adopted. Alternative excavation techniques which will reduce vibrations include a smaller rock hammer, grid sawing in conjunction with ripping and hammering, or using a rotary grinder.

The following procedures are recommended to reduce vibrations during rock excavation:

- Rock saw the perimeter faces. The rock saw slot must be maintained at a lower level than the adjacent excavation level at all times. Rock sawing would also improve the aesthetics of the finished rock faces.
- Maintain rock hammer oriented towards the face and enlarge excavation by breaking small wedges off face.
- Operate one hammer at a time and in short bursts only, to reduce amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances.

We recommend that a copy of this report be provided to the prospective excavation contractors so that they can make their own assessment of excavation conditions.

4.2.5 Groundwater Seepage

Groundwater was not encountered during the investigation nor was any groundwater seepage noticed to be emanating from the steep slope or its toe. However, groundwater inflows into the excavation may occur as local seepage flows at the soil-rock interface or clay through joints and bedding partings within the cut rock face, particularly after heavy rain.

Further to the above, and given an upslope catchment of limited extent, groundwater inflow into the bulk excavation are expected to be localised, of limited extent, and easily controlled by conventional gravity drainage.

We recommend that toe drains be formed at the base of all cut rock faces to collect groundwater and lead it to a sump for disposal. Further, groundwater seepage monitoring should be carried out during excavation, so that any unexpected conditions can be timeously addressed.

4.2.6 Stress Relief

In Sydney, there is a relatively high insitu horizontal stress field. When excavations extend down into the sandstone bedrock, the horizontal stresses are relieved, resulting in movement of the excavated faces into the excavation. These movements occur along sub-vertical bedding partings and are generally in the order of about 0.5mm to 1mm for each metre depth of excavation into the sandstone bedrock. Therefore, a predicted lateral movement up to about 15mm may occur in the vicinity of the deepest portion of the excavation. However, as the site is located near the edge of a topographical spur, we consider that most of the stress relief has already occurred, and therefore lateral movements due to stress relief are to be expected to be minimal and can be ignored.

4.3 Excavation Support

4.3.1 Batter Slopes

Where space permits, excavations in the shallow soil profile may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1 Horizontal (H). On the basis of the provided architectural drawings and survey plan, it would appear that temporary batters can generally be accommodated within the site geometry. Care is required during excavation not to undermine the existing properties along the east, particularly at the northern end. Possible seepage at the soil-rock interface may cause localised instability of soil batters and allowance should be made for sand bagging. We also recommend that a minimum 0.5m wide berm be provided between the toe of soil batters and the crest of vertical cut rock faces.

We expect that good quality sandstone of low or higher strength may be cut vertically. However, localised stabilisation measures may be necessary if adverse defects (such as inclined joints or bedding) are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay seams occurring in permanently exposed sandstone slopes may also require ‘dental’ treatment. Although only a minimal number of defects have been encountered in the boreholes, defects and particularly adverse defects commonly only become apparent during excavation and a large excavated rock face is visible. We therefore recommend that the perimeter rock face be progressively inspected by a geotechnical engineer/engineering geologist at 1.5m depth intervals as excavation proceeds to identify adverse defects and propose appropriate stabilisation measures. We note that geotechnical inspection of the initial stages of excavation at the southern end of the site will be particularly important to confirm that the sandstone bedrock is suitable for cutting vertically. We note that this area was not accessible to our drilling rig at the time of the fieldwork for this investigation. We recommend that provision be made in the contract documents (budget and program) for the above inspections and for possible stabilisation measures.

4.3.2 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of the conventional retaining walls, landscape walls and any rock face stabilisation measures:

- For design of conventional walls that will be supported by the structure, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.55 for the soil profile and extremely weathered sandstone bedrock, assuming a horizontal backfill surface.

- Where some minor movements of retaining walls can be tolerated (eg. landscape walls), we recommend the use of a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient, K_a, of 0.3, for the soil and extremely weathered sandstone bedrock, assuming a horizontal backfill surface.

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

- Any surcharge affecting the walls (eg. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.

- The retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.

- Walls founded at the crest of rock cuttings or natural cliff faces should be designed for lateral restraint based on a concrete to rock friction angle of 35°. Alternatively, the wall footing should be secured to the bedrock using rock dowels which are grouted into holes drilled at an angle back or away from the rock face. An allowable bond stress of 200kPa may be adopted for low (Class IV) or better sandstone. The rock dowels will be permanent and must be designed with due regard for long term corrosion. It is particularly important that the cut rock face below the toe of the retaining wall be inspected by a geotechnical engineer to confirm that there are no adverse defects present which could destabilise the wall.

- Rock bolts should be designed for an allowable bond strength of 350kPa assuming they are installed into sandstone bedrock of at least low to medium strength (Class III). Rock bolts should be 'nipped' tight. Where rock bolts extend beyond the site boundaries, permission from the neighbouring property owners will be required. Permanent rock bolts will need to be designed with due regard for long term corrosion (ie. hot-dipped galvanised).

4.4 Footings

Over the footprint of the proposed buildings, sandstone bedrock will be exposed at design subgrade levels, based on the results of the investigation.

Conventional pad or strip footings are therefore appropriate and should be designed for an allowable bearing pressure of 3,500kPa, for Class III or better sandstone bedrock. Retaining wall and other footings founded in sandstone bedrock at the crest of a cut or cliff face should be designed for a reduced allowable bearing pressure of 1,000kPa.

Higher allowable bearing pressures to 6MPa are probably feasible, however, at least 30% of the footing excavations would need to be spoon tested to confirm the founding material. Irrespective, the initial footing excavations for each building will need to be inspected by a geotechnical engineer to confirm that adequate founding material has been exposed.

4.5 On-Grade Floor Slabs

The proposed floor slabs will directly overlie bedrock. We therefore recommend that underfloor drainage be provided The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as 'blue metal' gravel. The underfloor drainage should connect with the wall drainage (where appropriate) and lead to a sump for gravity disposal.

The proposed concrete floor slabs, unless suspended, should be separated from all walls, footings etc (ie designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys.

4.6 Further Geotechnical Input

The following summarises the further geotechnical input which is required and which has been presented in the preceding sections of this report:

- Review and, if appropriate, revision of this report once architectural drawings are finalised.
- Dilapidation surveys of neighbouring buildings and structures to the east.
- Continuous quantitative vibration monitoring during rock excavation.
- Geotechnical inspection of cut rock faces.
- Monitoring of groundwater seepage into bulk excavation.
- Geotechnical footing inspections and spoon testing, if appropriate.

5 GENERAL COMMENTS

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

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 TEST REPORT

Client: JK Geotechnics
Project: Redevelopment of Chatswood Golf Club
Location: Beaconsfield Road, Chatswood, NSW

Ref No: 37168Z
Report: A
Report Date: 6/02/2017

AS 1289	**TEST METHOD**	**2.1.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
101	1.20-1.30	4.0
103	1.20-1.27	2.0
104	2.20-2.60	5.1
104	3.00-4.00	5.7
105	0.70-1.00	3.0

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SOIL TEST SERVICES

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TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Redevelopment of Chatswood Golf Club
Location: Beaconsfield Road, Chatswood, NSW

Ref No: 27168Z
Report: B
Report Date: 1/02/2017

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	1.45-1.48	1.1	22
	1.92-1.96	0.9	18
	2.56-2.60	0.6	12
	3.00-3.04	1.1	22
	3.75-3.80	0.8	16
	4.26-4.31	1.3	26
	4.90-4.94	0.6	12
	5.56-5.60	0.7	14
	6.36-6.40	0.7	14
	6.84-6.88	0.9	18
	7.25-7.29	0.5	10
	7.80-7.85	0.8	16
	8.25-8.30	0.9	18
	8.79-8.83	0.8	16
	9.45-9.50	0.9	18
	10.10-10.14	0.9	18
	10.96-11.00	1.4	28
	11.42-11.47	0.6	12
102	0.70-0.74	1.1	22
	1.20-1.24	0.8	16
	2.20-2.24	1.3	26
	2.97-3.00	1.0	20
	3.20-3.23	0.6	12
	3.66-3.71	0.8	16
	4.10-4.14	0.9	18
	4.85-4.89	1.3	26
	5.26-5.29	1.6	32
	5.85-5.89	2.0	40

NOTES: See Page 2 of 2

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SOIL TEST SERVICES

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TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	27168Z
Project:	Redevelopment of Chatswood Golf Club	**Report:**	B
		Report Date:	1/02/2017
Location:	Beaconsfield Road, Chatswood, NSW	**Page 2 of 2**	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
102	6.17-6.20	1.6	32
	6.80-6.84	0.9	18
	7.49-7.53	0.8	16
	7.95-8.00	0.7	14
	8.65-8.69	0.8	16
103	1.34-1.39	0.6	12
	1.96-2.00	0.5	10
	2.81-2.85	0.2	4
	3.06-3.09	0.4	8
	3.64-3.68	0.4	8
	4.06-4.11	0.4	8
	4.47-4.50	0.3	6
	4.77-4.82	0.4	8
	5.20-5.24	0.6	12
	5.71-5.74	2.1	42
	6.10-6.13	1.4	28
	6.90-6.94	0.7	14
	7.36-7.40	0.9	18
	7.91-7.95	0.9	18
	8.60-8.64	0.9	18
	9.10-9.14	0.9	18
	9.80-9.85	0.8	16
	10.15-10.20	1.5	30
	10.88-10.92	1.0	20

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :

   U.C.S. = 20 $I_{S\,(50)}$

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101

BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Method:** SPIRAL AUGER **R.L. Surface:** ~43.9 m
Date: 30/1/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** D.A.F./A.Z.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING							FILL: Silty sand, fine to medium grained, brown, traces of roots, fine to coarse grained igneous gravel and concrete fragments.	M			GRASS COVER APPEARS POORLY TO MODERATELY COMPACTED
		N = 7 4,4,3	43	1			FILL: Clayey sand, fine to coarse grained, brown and light brown, traces of fine to medium grained igneous gravel and fine to coarse grained sandstone gravel.				
						-	SANDSTONE: fine to coarse grained, light brown.	DW	L M - H		LOW 'TC' BIT RESISTANCE MODERATE TO HIGH RESISTANCE
							REFER TO CORED BOREHOLE LOG				
			42	2							
			41	3							
			40	4							
			39	5							
			38	6							
			37								

CORED BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z
Core Size:
R.L. Surface: ~43.9 m
Date: 30/1/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: D.A.F./A.Z.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DETAILS DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
		43	1		START CORING AT 1.34m						
100% RETURN		42	2		SANDSTONE: fine to coarse grained, orange brown and light grey, cross bedded at 20-30°.	DW	M - H				
		41	3							(2.74m) XWS, 0°, 2 mm.t	
		40	4		SANDSTONE: fine to medium grained, light grey and light red brown, bedded at 10-15°.					(3.71m) XWS, 0°, 15 mm.t	
ON COMPLETION OF CORING		39	5		SANDSTONE: fine to coarse grained, red brown and orange brown, bedded at 10-15°.		M				
100% RETURN		38	6								
		37									

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
101

CORED BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Core Size:** **R.L. Surface:** ~43.9 m
Date: 30/1/17 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** D.A.F./A.Z.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DETAILS – DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		36, 35, 34, 33	8, 9, 10, 11		SANDSTONE: fine to medium grained, light grey and grey, bedded at 10-15°.	SW	M			(7.07m) CS, 0°, 10 mm.t (7.67m) XWS, 40°, 50 mm.t (8.18m) XWS, 40°, 30 mm.t (8.47m) XWS, 0°, 20 mm.t (11.27m) CS, 5°, 60 mm.t	
		32, 31, 30	12, 13		END OF BOREHOLE AT 11.75 m						


JK Geotechnics
27168Z BH101 START CORING AT 1.34m
1
2
3
4
5
6
7
8
9
10
11
END OF BH AT 11.75m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Method:** SPIRAL AUGER **R.L. Surface:** ~32.1 m
Date: 30/1/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** D.A.F./A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■					32			-	ASPHALTIC CONCRETE: 30mm.t				
										FILL: Clayey sand, fine to coarse grained, light brown, with silt and traces of fine to medium grained igneous and sandstone gravel.	M			
										SANDSTONE: fine to coarse grained, light brown and light grey.	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						31	1			REFER TO CORED BOREHOLE LOG				
						30	2							
						29	3							
						28	4							
						27	5							
						26	6							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 102

CORED BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z	**Core Size:**	**R.L. Surface:** ~32.1 m
Date: 30/1/17	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** D.A.F./A.Z.

Water Loss\Level	RL (m AHD)	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, structure, minor components.)	Weathering	Strength	DEFECT DETAILS – DESCRIPTION (Type, inclination, thickness, planarity, roughness, coating.) Specific
	32		START CORING AT 0.65m			
100% RETURN		0.65	SANDSTONE: fine to coarse grained, orange brown.	DW	M - H	
	31		SANDSTONE: fine to medium grained, light grey and grey, bedded at 0-10°.			(1.39m) Be, 0°, P, R (1.64m) Be, 0°, P, R, IS (1.65m) Be, 0°, P, R, IS (1.76m) Be, 10°, P, R, IS (1.84m) Be, 10°, P, R, IS (1.88m) XWS, 10°, 2 mm.t
	29	3			M	(3.15m) 0°, 30 mm.t, SHALE LENSE
ON COMPLETION OF CORING	28	3.5	SANDSTONE: fine to coarse grained, light orange brown and light grey bedded at 15-30°.			(3.85m) Be, 10°, P, R, IS (4.03m) XWS, 0°, 10 mm.t (4.06m) XWS, 0°, 5 mm.t (4.30m) XWS, 20°, 30 mm.t (4.40m) CS, 0°, 10 mm.t
100% RETURN	27	4.8	as above, but light red brown and light orange brown, bedded at 0-10°.		H	(5.11m) J, 20°, Un, R
	26	6.65	as above, but orange brown and light grey, bedded at 10-30°.		M	(6.62m) Be, 0°

POINT LOAD STRENGTH INDEX $I_s(50)$: EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH

DEFECT SPACING (mm): 500, 300, 100, 50, 30, 10

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
102

CORED BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z | **Core Size:** | **R.L. Surface:** ~32.1 m
Date: 30/1/17 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** D.A.F./A.Z.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		25, 24	8		SANDSTONE: fine to coarse grained, orange brown and light grey, bedded at 10-15°.	DW	M			(7.43m) Cr, 0°, 30 mm.t (7.84m) J, 35°, P, S, IS	
		23, 22, 21, 20, 19	9, 10, 11, 12, 13		END OF BOREHOLE AT 8.77 m						

JK_LIB_CURRENT - V8.00.GLB Log J & K CORED BOREHOLE - MASTER 27168Z CHATSWOOD.GPJ <<DrawingFile>> 03/03/2017 11:42 Produced by gINT Professional, Developed by Datgel


JK Geotechnics
27168Z BH102 START CORING AT 0.65m
0
1
2
3
4
5
6
7
8
END OF BH AT 8.77m

Borehole No.
103

BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Method:** SPIRAL AUGER **R.L. Surface:** ~35.7 m
Date: 31/1/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** D.A.F./A.Z.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■								-	FILL: Silty sand, fine to medium grained, light brown, traces of fine to medium grained igneous and sandstone gravel.	DW	M		MODERATE 'TC' BIT RESISTANCE
						35	1			SANDSTONE: fine to coarse grained, red brown and light red brown.	XW - DW	EL - VL		VERY LOW RESISTANCE
				■								M - H		MODERATE TO HIGH RESISTANCE
						34, 33, 32, 31, 30, 29	2, 3, 4, 5, 6			REFER TO CORED BOREHOLE LOG				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z | **Core Size:** | **R.L. Surface:** ~35.7 m
Date: 31/1/17 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** D.A.F./A.Z.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT SPACING (mm) (500, 300, 100, 50, 30, 10)	DEFECT DETAILS DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific / General
		35	1		START CORING AT 1.27m					
100% RETURN		34	2		SANDSTONE: fine to coarse grained, red brown, orange brown and light grey, bedded at 15-20°.	DW	M			(2.21m) CS, 0°, 40 mm.t
ON COMPLETION OF CORING		33			CORE LOSS 0.50m					
20% RETURN (BELOW 2.5M)		32	3, 4		SANDSTONE: fine to medium grained, red brown orange brown and light grey, bedded at 10-15°.	DW	L - M			
		31	5		as above, but fine to coarse grained.					
		30	6		SANDSTONE: fine to medium grained, light grey and grey, bedded at 0-10°.	SW - FR	H			(5.45m) XWS, 0°, 90 mm.t (6.19m) Be, 10°, P, S (6.22m) J, 10° (6.25m) Be, 0° (6.29m) XWS, 5°, 10 mm.t, IS (6.31m) Be, 5°, P, R, IS (6.32m) Be, 0°, P, S, IS
		29			as above, but fine to coarse grained.	SW	M			(6.82m) J, 50°, P, R, IS

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
103

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z | **Core Size:** | **R.L. Surface:** ~35.7 m
Date: 31/1/17 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** D.A.F./A.Z.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
20% RETURN (BELOW 2.5M)		28–25	7.43–11.50		SANDSTONE: fine to coarse grained, light grey and grey, bedded at 0-10°.	SW	M			(7.43m) XWS, 5°, 20 mm.t	
							H			(10.11m) XWS, 5°, 2 mm.t	
		24–22	11.50		END OF BOREHOLE AT 11.50 m						

JK_LIB_CURRENT - V8.00.GLB Log J & K CORED BOREHOLE - MASTER 27168Z CHATSWOOD.GPJ <<DrawingFile>> 03/03/2017 11:43 Produced by gINT Professional, Developed by Datgel

JK Geotechnics
27168Z BH103 START CORING AT 1.27m
1
2
CORE LOSS 0.50m
3
4
5
6
7
8
9
10
11
END OF BH AT 11.50m

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
104
1 / 1

BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Method:** SPIRAL AUGER **R.L. Surface:** ~35.5 m
Date: 31/1/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** D.A.F./A.Z.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			0		FILL: Silty sand, brown and light grey, trace of fine to medium grained sand.	D			APPEARS MODERATELY COMPACTED
	N = 13 9,6,7	35			FILL: Silty clay, low to medium plasticity, light grey, red brown and orange brown, trace of fine grained sand and fine to coarse grained sandstone gravel.	MC<PL			
	N = 3 1,2,1	34	1			MC>PL			APPEARS POORLY COMPACTED
		33	2	-	SANDSTONE: fine to coarse grained, light orange brown and light grey.	DW	M - H		MODERATE TO HIGH 'TC' BIT RESISTANCE
							M		MODERATE RESISTANCE
		32	3				M - H		MODERATE TO HIGH RESISTANCE
		31	4		END OF BOREHOLE AT 4.00 m				'TC' BIT REFUSAL
		30	5						
		29	6						

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
105
1 / 1

BOREHOLE LOG

Client: WATERMARK CHATSWOOD
Project: REDEVELOPMENT OF CHATSWOOD GOLF CLUB
Location: BEACONSFIELD ROAD, CHATSWOOD, NSW

Job No.: 27168Z **Method:** SPIRAL AUGER **R.L. Surface:** ~42.3 m
Date: 31/1/17 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** D.A.F./A.Z.

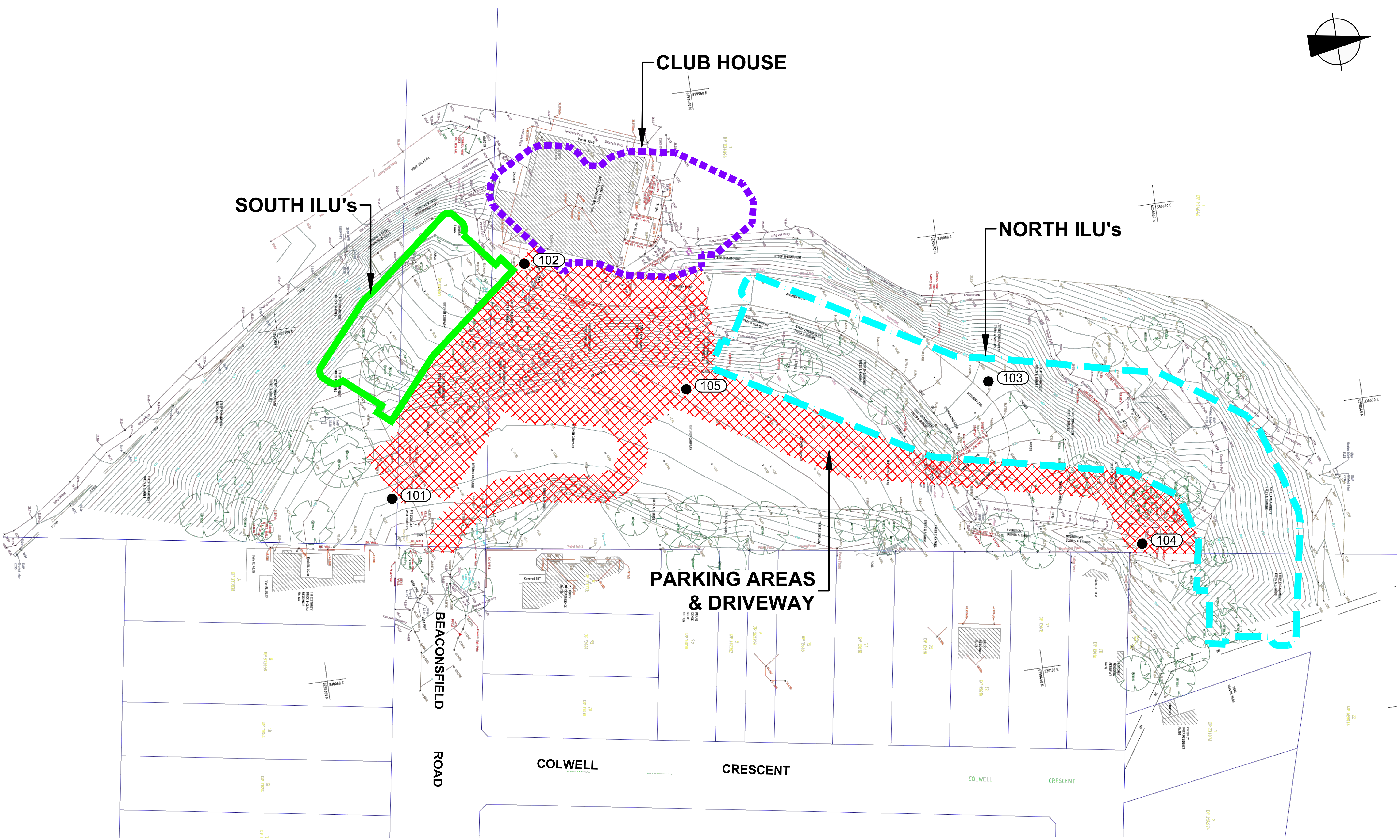
Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	ES, DS		42	0		-	ASPHALTIC CONCRETE: 30mm.t				
	ES	N=SPT 4/ 0mm REFUSAL					FILL: Silty sand, fine to coarse grained, light brown, trace of fine to coarse grained sandstone gravel.				
	DS						SANDSTONE: fine to coarse grained, light orange brown and light grey.	DW	L - M / M - H		LOW TO MODERATE 'TC' BIT RESISTANCE / MODERATE TO HIGH RESISTANCE
			41	1			END OF BOREHOLE AT 1.00 m				'TC' BIT REFUSAL
			40	2							
			39	3							
			38	4							
			37	5							
			36	6							

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

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

Title:	SITE LOCATION PLAN
Location:	BEACONSFIELD ROAD CHATSWOOD, NSW
Report No: 27168Z	Figure No: 1
	JK Geotechnics

PLOT DATE: 8/03/2017 11:04:11 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\27000'S\27168Z CHATSWOOD\CAD\27168Z.DWG



LEGEND

- BOREHOLE
- PARKING AREAS AND DRIVEWAY
- NORTH ILU's
- CLUB HOUSE
- SOUTH ILU's

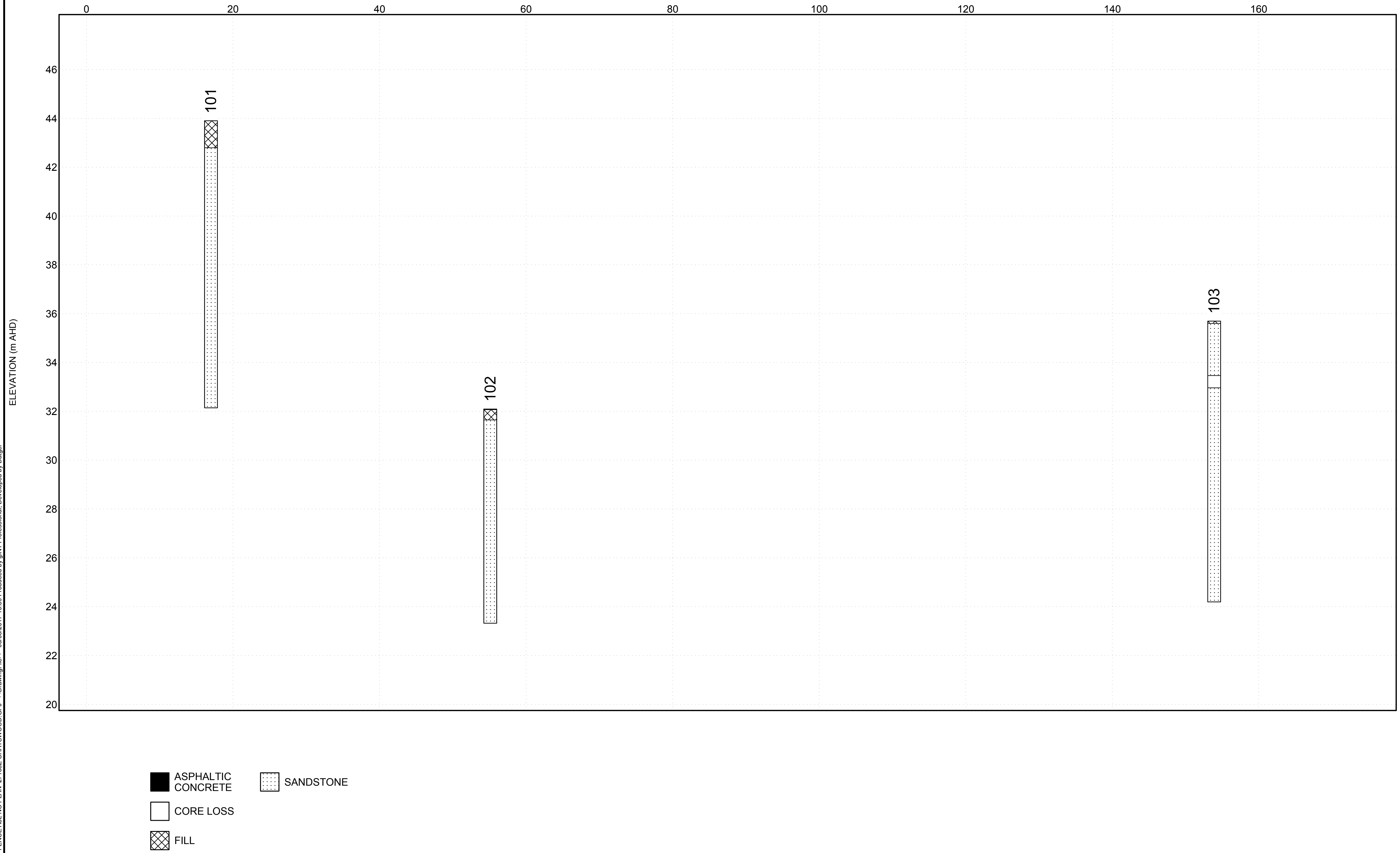
0 8 16 24 32 40
SCALE 1:800 @A3 METRES

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

Title:	**BOREHOLE LOCATION PLAN**
Location:	BEACONSFIELD ROAD CHATSWOOD, NSW
Report No:	27168Z
Figure No:	2

JK Geotechnics

PLOT DATE: 17/05/2017 10:54:55 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\27000'S\27168Z CHATSWOOD\CAD\27168Z.DWG



Scales (A3)	Title: GRAPHICAL BOREHOLE SUMMARY
H 1:500	Location: BEACONSFIELD ROAD, CHATSWOOD, NSW
V 1:150	Report No: 27168Z — Figure No: 3
	JK Geotechnics

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, 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 (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 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 a Cone Penetrometer Test (CPT). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

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

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

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

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 <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves 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

SOIL

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

ROCK

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

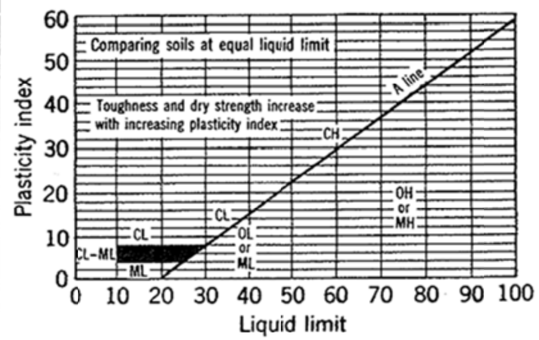
DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

Major division	Subdivision	Fines	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*)		
			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			



Plasticity chart
for laboratory classification of fine grained 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	