REPORT TO

CHRIS ACRET

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENCE AND POOL

AT

44 BOWER STREET, MANLY, NSW

Date: 28 February 2020

Ref: 29343SM2rpt Rev 2

JKGeotechnics

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
29343SM2rpt	Final Report	4 February 2020
29343SM2rpt Rev1	Minor amendments	6 February 2020
29343SM2rpt Rev2	Update to DA drawings reference. Minor amendments.	28 February 2020

Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical assessment for a proposed residence and pool. The location of the site is shown in Figure 1. The investigation commissioned by Mr Chris Acret on 20 December 2019 and was completed in accordance with our proposal Ref P50903SM dated 13 December 2019.

From review of the supplied the Development Application series drawings (No.s DA.301 to 304, 401 to 404, 501, 502, 901, all Rev A, dated 28 February 2020) prepared by Madeline Blanchfield Architects, we understand the proposed development includes demolition of the existing house and construction of a new 2 storey house with a double garage, a partial basement level and a pool to the rear. The garage will be at a similar level to the existing driveway. The basement will be at RL 23.49m requiring about 1.8m to 2.7m of excavation. The basement extent is limited to the rear portion of the main building footprint but a side path similar to the basement level is proposed along the eastern side of the basement extending to the boundary also requiring similar depths of excavation. The ground floor level is similar to existing levels. The development area is limited to the upper gently sloping portion of the hillside, away from the steeper lower slopes and outcrops. Existing stormwater and sewer pipes will be relocated from the overhand feature at mid-slope.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, retaining wall design/earth pressures, footing design, and subgrade preparation.

2 INVESTIGATION PROCEDURE

The geotechnical investigation was completed on 10 January 2020 and comprised a site walkover by our Associate Geotechnical Engineer (Matthew Pearce) to enable mapping of topographic, surface drainage and geological conditions of the site and its immediate environs. The observations were complemented by Dynamic Cone Penetration (DCP) tests in the front garden completed during a previous site visit in 2016.

The principal geotechnical features are presented on Figure 2, which also shows the outline of the development overlain on an existing survey base plan. Figure 3 presents a longitudinal sectional sketch through the site based on the survey data and augmented by our mapping observations and DCP/borehole information. Figure 4 presents a section across the eastern site boundary, mostly based on taped measurements and observations. The attached Report Explanation Notes define the terms and symbols used.

Due to very shallow refusal of the initial DCP test, two subsequent tests were completed within a 1.5m radius. All the tests (DCP1A to DCP1C) were completed to refusal to further penetration at depths of 0.2m and 0.25m below existing surface levels. The DCP Test Results are attached to this report. From DCP refusal we infer the presence of sandstone bedrock but it should be noted since the test does not return a sample, refusal can be due to cobbles, boulders (also known as 'floaters'), or other obstructions within fill.

The location of the DCP tests, as shown on Figures 2 and 3, was recorded by taped measurements from features shown on the supplied survey plan prepared by Hill and Blume Pty Ltd (Ref. 61235 dated 2 April 2019). The approximate reduced levels shown on the attached DCP Test results were interpolated from spot heights indicated on the survey. The survey datum is the Australian Height Datum (AHD).

Where readily assessable, the strength of the sandstone bedrock exposed was assessed by sounding with a geological pick while the degree of weathering and other characteristics were assessed visually.

3 RESULTS OF INVESTIGATION

3.1 Site Description

This site description should be read in conjunction with Figures 1, 2 and 3.

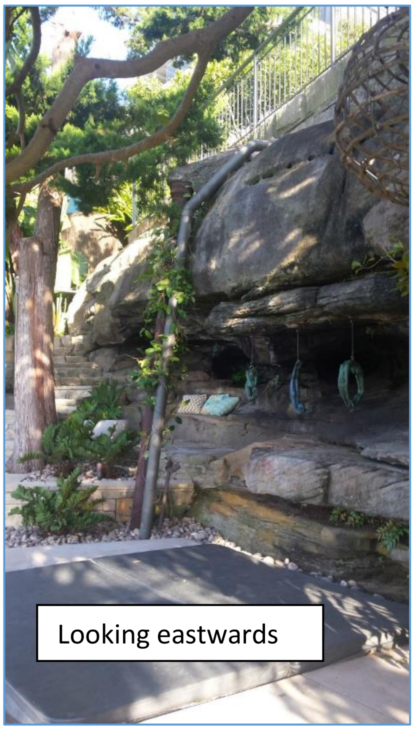
The site spans the stepping sandstone cliff lines from the seafront walkway (near Shelly Beach) to the start of gentle mid-slopes of the promontory which forms of Sydney's North Head National Park.

No 44 Bower Street is a rectangular property 15m wide by 45m long with three distinct cliff lines over its northern (lower) half and a slope of about 4° to 5° over its southern (upper) half. Surface reduced levels range from 27.7m at the southern street frontage to about RL9m at the northern property boundary, although the lower cliffs extend down to the walkway at RL2m.

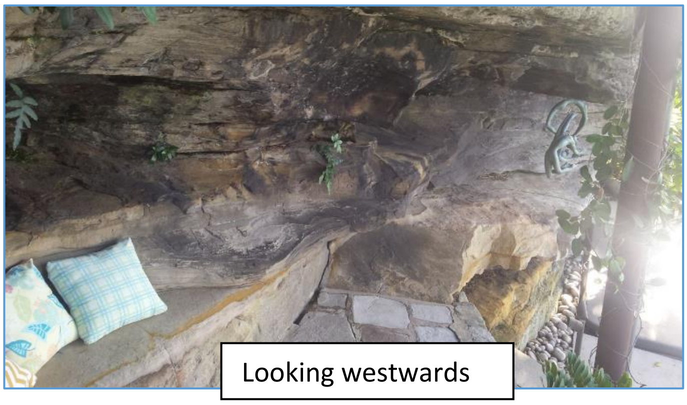
The property is currently occupied by a 2-storey brick house positioned on the upper slope with an elevated driveway off Bower Street which is supported by a low height stone retaining wall. There is an adjacent front garden with minor steps and stone paved pathways between a few small trees. There are narrow pathways down both sides of the house. Steps on the eastern side of the house have been cut into sandstone bedrock and lead down to a pebble surfaced pathway. These steps are the only rock exposure at the front of the property. The western side path is concrete paved with brick steps. The house has a lower ground floor storage 'room' at about RL25m over the northern portion only, but there is a subfloor space under the remainder of the house where bedrock was visible.

At the rear of the house is an elevated deck with storage access under, where sandstone bedrock is also visible. This exposure of rock extends out to the upper (rear) lawn, on the eastern side only. The lawn is supported by a low height stone masonry retaining wall founded on outcropping sandstone ("Cliff Line 1"). This sandstone unit is coarser grained than the more commonly encountered Hawkesbury Sandstone, and has frequent fine-grained quartz gravel inclusions. This rock unit features significant vertical jointing orientated parallel to the side boundaries, spaced about every 2m, which have weathered in a distinctly rounded manner. About 1m from the toe of this cliff line, is the more common medium grained sandstone (without gravel inclusions).

Steps linked with stone paved paths have been cut through the upper and middle cliff lines. There is another lawn on the 5m to 6m wide 'step' (between the Cliff Line 1 and Cliff Line 2) which is also retained by a low height stone masonry wall. This wall sits upon a sandstone cliff which has a substantial but irregular overhang, ranging from 1m to about 3m (into the hillside) and about central to the height of this cliff, as shown in the photos below. The rock is generally massive and continuous with no significant jointing observed. The overhang is also indicated on the survey plan and Figure 2.



Looking eastwards



Looking westwards

At the toe of Cliff Line 2 is a spa and stone paved area supported by a masonry retaining wall at the top of a very steep, densely vegetated slope, several meters high. Two pipes are exposed in front of Cliff 2 top to the toe. At the toe of that slope are the tops of small and medium sized trees and the lower 'Cliff Line 3' which is only visible from the Manly to Shelly Beach walkway. The eaxact property boundary is not clearly marked but is about at the top of the cliff line

Located beyond the site, the walkway is concrete surfaced and supported by a stone seawall. Between the walkway and the cliff line is a narrow lawn. Cut into the cliff is a stormwater (or sewer) pipe covered with stone masonry and some small "faux rock" (coloured shotcrete) sections. The sandstone is horizontally bedded with the lower 1m comprising several small (approx. 0.5m x 0.5m x 0.5m) slightly detached blocks. To the east of the pipeline are adversely inclined defects. There are large boulder sized blocks (1m tall x 2m x wide x 1m deep) with a vertical joint (potential release plane) at the rear, and root growth from small and medium sized trees above. Refer to photo below. Since our visit in 2016 the blocks have been stabilised by rock bolts.

Covered Pipe(s)

Large Blocks with Jointing at Rear (now stabilised with rock bolts)

The neighbouring properties have many similarities with the subject site, also having 2 storey houses with set-backs of about 1m from the common boundaries. The landscaping profiles to the north of the houses are similar to that of the subject site due to the cliff lines extending across the site boundaries.

The house to the east of the site, No 42, is of brick construction and set back about 1m from the common boundary. It has a swimming pool between Cliff Lines 1 and 2. Between the houses, existing surface levels are similar across the boundary which is marked by a combination of stone masonry walls, fencing, low height rendered walls and vegetation. The house has a partial basement level cut into the hillside. As shown on Section B (Figure 4) and the adjacent photograph, there is a 3.3m deep vertical cutting though sandstone bedrock located parallel to, and set back 0.8m from, the common boundary. The northern end of the ground floor level side path appears to be suspended from the main building spanning the narrow gap between the basement wall and the rock face. The path has adown turn supported on the top of the rock.

To the west of the site, at No 46, a new house is under construction. The building is set back about 1m from the common boundary. We understand from supplied photographs that bulk excavation for the lower ground floor extended to the common boundary is lower than at No 44. The common boundary is marked by a stone capped brick wall at the front and then a timber fence for the remainder of the upper portion of the sites.

3.2 Geology and Subsurface Conditions

The Sydney 1:100,000 geological map indicates the site to be underlain by Hawkesbury Sandstone.

Our observations confirmed the 'rock outcrops' indicated on the survey plan are outcrops of mostly intact massive sandstone bedrock. There were also other exposures under the house and at the steps to the south-west of the house. The majority of the sandstone was assessed as medium or high strength and slightly weathered.

The upper and lower units of sandstone bedrock were typical of Hawkesbury sandstone composition being medium to coarse grained. Most of the outcrops appeared to be massive and continuous except for the following features, described from top to bottom:

- Between RL23.5m to RL21m there is an outcrop of sandstone from with abundant fine-grained quartz gravel exhibiting regular vertical jointing roughly parallel to the side boundaries of the site (reference 'Cliff-Line 1' on Figure 3)
- There is an overhang in 'Cliff-line 2' as shown on Figure 3). Based on cursory observations during the walkover the rock appeared to be free of adverse jointing.
- There are two large boulder sized blocks of sandstone just to the north of the property boundary (reference 'Cliff-line 3' on Figure 3). The blocks are about 1.5m high x 1m or 2m wide x 1m deep with

a vertical joint behind them (potential release plane). They have recently had rock bolts installed. Based on the survey plan the blocks appear to be located within the council reserve or nature strip.

- Much of the remainder of this cliff line was covered with vegetation and could not be assessed for stability.

4 COMMENTS AND RECOMMENDATIONS

4.1 Dilapidation Surveys

As discussed in the following sections, hard rock excavation to within 1m to 2m of the neighbouring houses will be required for the proposed development. Detailed dilapidation surveys should be completed on both neighbouring properties, assuming No 46 is completed by commencement of development. Particular care is needed during percussive rock excavation if green mortar or render are present on the adjacent building site.

Ideally the dilapidation surveys should comprise a detailed inspection of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, orientation etc. The owners of the adjoining properties should be asked to confirm that the reports represent a fair record of actual conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.2 Demolition

There is potential for transmission of vibrations from demolition works to impact on the neighbouring structures. In particular, if the existing building's footings or floor slabs are to be demolished by use of rock breakers, a vibration monitor should be set up on the neighbouring buildings to determine whether vibrations emitted exceed the attached 'Vibration Emission Design Goals'. This is discussed in more detail below in Section 4.3. If the vibration limits are exceeded, the footings or slabs should be saw cut or otherwise broken into smaller manageable pieces. The impact of large masonry or concrete having been dropped can also cause potentially damaging vibrations.

4.3 Excavation

As described in Section 1, to achieve the proposed basement floor level and side path, excavation of up to about 2.7m depth will be required at the rear of the house. Localised excavation will also be required for the partially inground pool which extends over the crest of Cliff Line 1.

Following removal and disposal of what we expect will be a thin covering of sandy topsoil, the remainder of excavation will be through competent sandstone bedrock which is likely to be medium to high strength based on our assessment of the rock outcrops.

Soil and even extremely low strength rock will be readily removed by buckets of small excavators, however, excavation of sandstone of low or higher strength will require specialised rock excavating equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders and rock saws. Quartz gravel or iron indurated bands within the rock mass may result in higher than normal 'wear and tear' of excavation attachments.

Due to the close proximity of the neighbouring residences, hydraulic rock hammers must be used with care due to the risks of damage to nearby structures from vibrations generated by such equipment. We recommend that hydraulic rock hammers be limited in size, say no more than 500kg attached to a small excavator, provided vibrations emitted are tolerable. Initial hammering must commence from the point furthest from the neighbouring properties i.e. the middle of the site, to check the appropriateness of the hammer selection. Continuous monitoring of the vibrations transmitted to the adjoining buildings must be carried out during demolition of existing footings, and excavations using a rock hammer, with the monitors attached to flashing warning lights to warn the operator when acceptable vibration limits have been exceeded. Reference should be made to the attached Vibration Emission Design Goals sheet for typical acceptable limits of transmitted vibrations (5mm/s where frequency is less than 10MHz). If it is found that transmitted vibrations are excessive, then it would be necessary to change to alternative excavation equipment, such as a smaller rock hammer, ripping hooks, rotary grinders or rock saws. Using a rock saw to cut a slot along the excavation perimeter before breaking out the rock using a ripping tyne or rock hammer may reduce the transmitted vibrations, but the effectiveness of such an approach must be confirmed by vibration monitoring.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain the rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate hammer in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in such excavation work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

To relocate stormwater or sewer pipes, we recommend core holes are drilled through the rock. Provided the diameter is similar to the existing pipes, cored holes should not reduce the integrity of the overhang. Any additional localised excavation required in the rock to install the pipes should discrete, carried out using non-vibratory techniques such as hand held concrete saws/grinders, and subject to geotechnical engineers' approval.

4.4 Groundwater

Based on the investigation results, we do not expect significant groundwater seepage flows at this site. However, we would expect some groundwater seepage to occur at the soil/rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy or prolonged rainfall. Seepage, if any, during excavation is expected to be satisfactorily controlled by conventional sump and pumping techniques or gravity drainage.

We recommend that groundwater seepage into the excavation be monitored by site personnel and the results (quantity, location, source, etc.) be reported to the geotechnical and hydraulic engineers so that any unexpected conditions can be promptly addressed. In the long term, drainage should be provided behind all retaining walls, and below the lowest floor slabs.

Where habitable rooms are planned next to retaining walls, reinforced shotcrete or even unsupported rock faces, there must be provision for effective drainage and damp control for internal walls. Exposed rock faces in particular have the potential to fret and spall with seepage. Toe drains can then become blocked with this material so it is an important measure to include crawl space for maintenance, or hatches to facilitate flushing.

4.5 Excavation / Rock Face Support

From observation of sandstone outcrops along the eastern side of the property, the shallow depth to refusal of the DCP tests at the front (inferring just 0.15m of soil cover) and sandstone exposures under the house we expect that only a nominal depth of soil cover will be present and that shoring walls installed prior to excavation will not be required.

Rock of low strength which is relatively free of defects will be suitable to be cut vertically and remain unsupported in the short term only. Long term support must be provided by means of reinforced shotcrete horizontally braced by the structure of the new house or by retaining walls (typically reinforced block masonry or similar), also braced by the structure or ‘L-shaped’ cantilever footings. Support may also be required where a narrow ‘plinth’ of rock remains on the eastern boundary following excavation for the side path and is discussed further below.

Rock of medium or high strength, provided it is relatively free of adverse defects will be suitable to be cut vertically and remain unsupported in the long term. Note, unsupported rock faces have the potential to fret (usually where there is seepage) which could clog up toe drains in the long term and reference should be made to Section 4.4 for further advice.

Excavated rock faces should be inspected by a geotechnical engineer progressively, every 1.5m of vertical cut, to check for the presence of adverse defects and to enable access for any rectification works such as installation of rock bolts and to reduce the risk of instability for property within the zone of influence above and workers below. From the orientation of the site boundaries and predominant jointing of the sandstone in the Sydney basin, we expect that some rock bolting may be required and should be budgeted for. Similarly, seams, if encountered, will most likely need dental treatment such as grubbing out and dry packing with non-shrink grout and provision of ‘spitters’ to alleviate groundwater pressure. Provision must be made for long term support of all rock faces by retaining walls as permanent rock bolts will not be allowed to extend beyond site boundaries.

Eastern Side Path

As shown on Figure 4, a narrow stretch of rock (measuring about 0.8m wide, 2m high and 17m long) will be remain along the eastern boundary following bulk excavation. If, subject to geotechnical inspection, it proves to be a continuous massive block of sandstone bedrock then it may be left insitu. However, if vertical joints intersects the cut face, it may become unstable. Initial excavation must not extend beyond 1m from the boundary and not deeper than 1.5m from existing surface levels prior to geotechnical inspection. The top of the rock should be cleaned of all debris to enable thorough assessment. Horizontal cored holes may also be required to investigate the presence of such jointing or other adverse defects. If joints are encountered, potential stabilisation measures will be specified by the geotechnical engineer and may include a series of fully grouted bars (or rock bolts) to stitch the rock together. Alternatively, it may be assessed that the side path would need to be underpinned to bulk excavation level (BEL). Permission should be sought from the neighbour for the potential installation of temporary and permanent rock bolts. Another possibility would be a retaining wall perhaps tied to a thickened concrete path to form a ‘L’-shaped cantilever footing.

4.6 Footings

Due to the presence of shallow rock across the property, we consider the site can be classified as Class A in accordance with AS2870-2011 Residential Slabs and Footings.

All footings must be uniformly founded on sandstone bedrock. Pad and strip footings would be appropriate. Footings on sandstone of at least very low strength can generally be designed for an allowable bearing pressure (ABP) of up to 1,000kPa.

Any footings near the crest of an excavation or step down in natural bedrock, ie above a line of 1 vertical (V): 1 horizontal (H) drawn up from the toe of a cutting/cliff, should be designed for a reduced ABP of 500kPa provided the rock is free of adverse defects and must be specifically inspected by a geotechnical engineer.

Footings should be inspected by a geotechnical engineer. All footings must be cleaned of loose or softened material and be free of water prior to pouring concrete, without delay.

4.7 Slabs on Grade

Rock is expected to be uniformly exposed at bulk excavation level for the basement.

A de-bonding layer of sand or fine crushed rock should be placed between the slab and the bedrock. Subsoil drains should be provided along the perimeter of the slabs on grade, with inverts not less than 0.2m below subgrade level. The drainage trenches should be excavated with a longitudinal fall to appropriate discharge. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

The proposed garage floor slab will be at the existing driveway location and extend slightly beyond the existing low height stone retaining walls. Where the proposed slab extends beyond the existing footprint it could be supported on new backfill behind new retaining walls but there is potential for differential settlement between the new and old subgrade. Consideration could therefore be given to supporting the entire slab on short piles to rock.

Trafficable slabs on grade should be designed to transmit shear forces by dowelled or keyed joints.

4.8 Lower Cliff Stability

Cliff Line 3

Regarding the lower cliff visible from the Shelly to Manly walkway, two large boulder sized blocks (about 1.5m high x 1m or 2m wide x 1m deep) with a vertical joint (potential release plane) just to the north of the property boundary (reference 'Cliff-line 3' on Figure 3) were been identified during a previous walkover. Based on the survey plan they appear to be located within the council reserve or nature strip. We do not expect that excavation or construction activities will have any effect on the stability of these blocks but note that these blocks appear to have since been stabilised with rock bolts.

We understand a risk assessment was completed by JK Geotechnics for Manly Council Ref 28099ZRrpt rev 3 dated 13 April 2016, indicated a 'Risk to Life of 3×10^{-5} for persons below the rock but that stabilisation measures have since been implemented thus reducing the risk.

We do not consider any further action is warranted.

Cliff Line 2

A large overhang is present in Cliff Line 2 for which has no stabilisation measures have been recommended. While no bulk excavation or building works are planned directly above Cliff Line 2, we suggest the contractor use 'non' or low vibration emitting equipment for works in the vicinity of the proposed pool area to avoid potential for vibrations damaging the integrity of the overhang. Otherwise this feature should be specifically assessed for stability prior to excavation. Part of assessing a large overhang's stability requires inspection of the top of the rock above and behind/upslope for which the turf would need to be removed.

The only planned excavation on Cliff Line 2 is diversion of the existing stormwater and sewer pipes which currently run down from the crest of the overhang. To maintain integrity of the overhang, the relocation of the pipes should be completed with cored holes (rotary diamond core drilling with water flush) drilled, from the top of the rock, to exit at the rearmost of the cave ceiling. They can then be recessed into the base of the 'cave' to be hidden. Recessing into the base of the cave should only be carried out using tools emitting negligible magnitudes of vibration such as hand held concrete saws and small hand held demo hammers.

While it is expected that the densely vegetated steep slope between cliff lines 2 and 3, comprises shallow soil on a stepping rock profile, to reduce the risk of instability, the stormwater system must be checked that it discharges beyond the toe of the steep slope (to at least the top of Cliff Line 3).

4.9 Summary of Recommended Additional Geotechnical Work

- Vertical cored boreholes could be considered for a more detailed rock strength assessment and indication of the presence of defects, to reduce uncertainty in tenders relating to excavation.
- Test pits following demolition to profile the rock surface along the side boundaries.

- Geotechnical review of structural drawings and contractor's methodology prior to excavation and construction
- Further geotechnical assessment of the overhang for long-term stability, if percussive excavation techniques are proposed nearby.
- Vibration monitoring/advice at commencement of excavation
- Progressive geotechnical inspections of rock cuts every 1.5m of vertical excavation and when within 1m of the eastern boundary.
- Horizontal cored boreholes to check the integrity of the narrow block of rock along the eastern boundary which will remain following excavation.
- Footing Inspections, especially where adjacent to cuts/cliffs

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 or observed outcrops may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

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

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	Dr Craig Taylor		
Project:	Proposed Residence and Pool		
Location:	44 Bower Street, Manly, NSW		
Job No.	29343SM	Hammer Weight & Drop: 9kg/510mm	
Date:	15-4-16	Rod Diameter: 16mm	
Tested By:	MP	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL27.2	RL27.2	RL27.2				
Depth (mm)	**1A**	**1B**	**1C**				
0 - 100	1	1	1				
100 - 200	3R	2	1R				
200 - 300		3R/50mm					
300 - 400							
400 - 500							
500 - 600							
600 - 700							
700 - 800							
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2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal

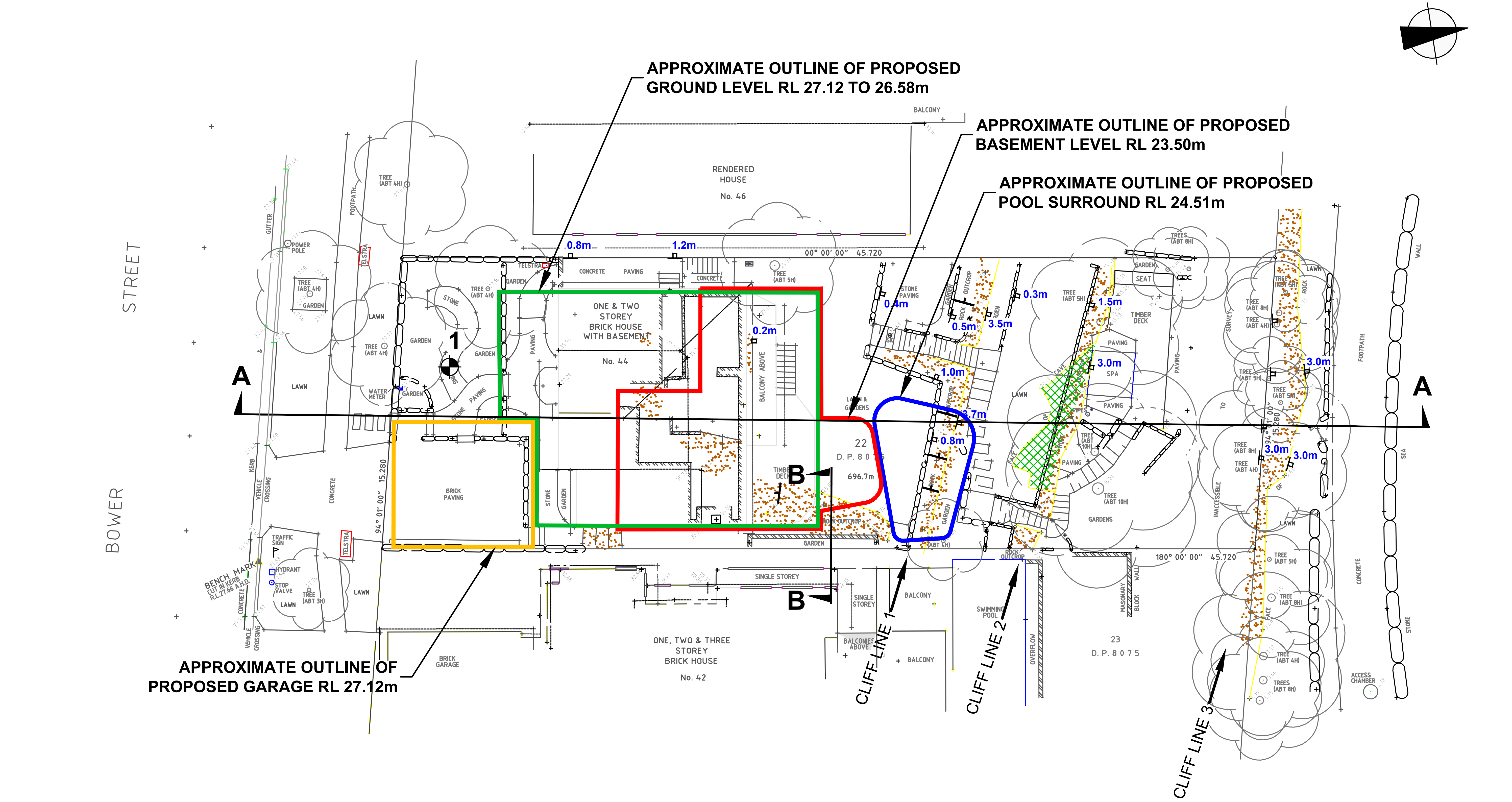
AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	SITE LOCATION PLAN
Location:	44 BOWER STREET MANLY, NSW
Report No: 29343SM2	Figure No: 1

JKGeotechnics

PLOT DATE: 4/02/2020 9:15:19 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\29000'S\29343SM MANLY\CAD\29343SM2.DWG



LEGEND

OVERHANG

0.2m
ROCK OUTCROP AND HEIGHT (m)

JOINT (INDICATIVE LOCATION ONLY)

DCP TEST

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

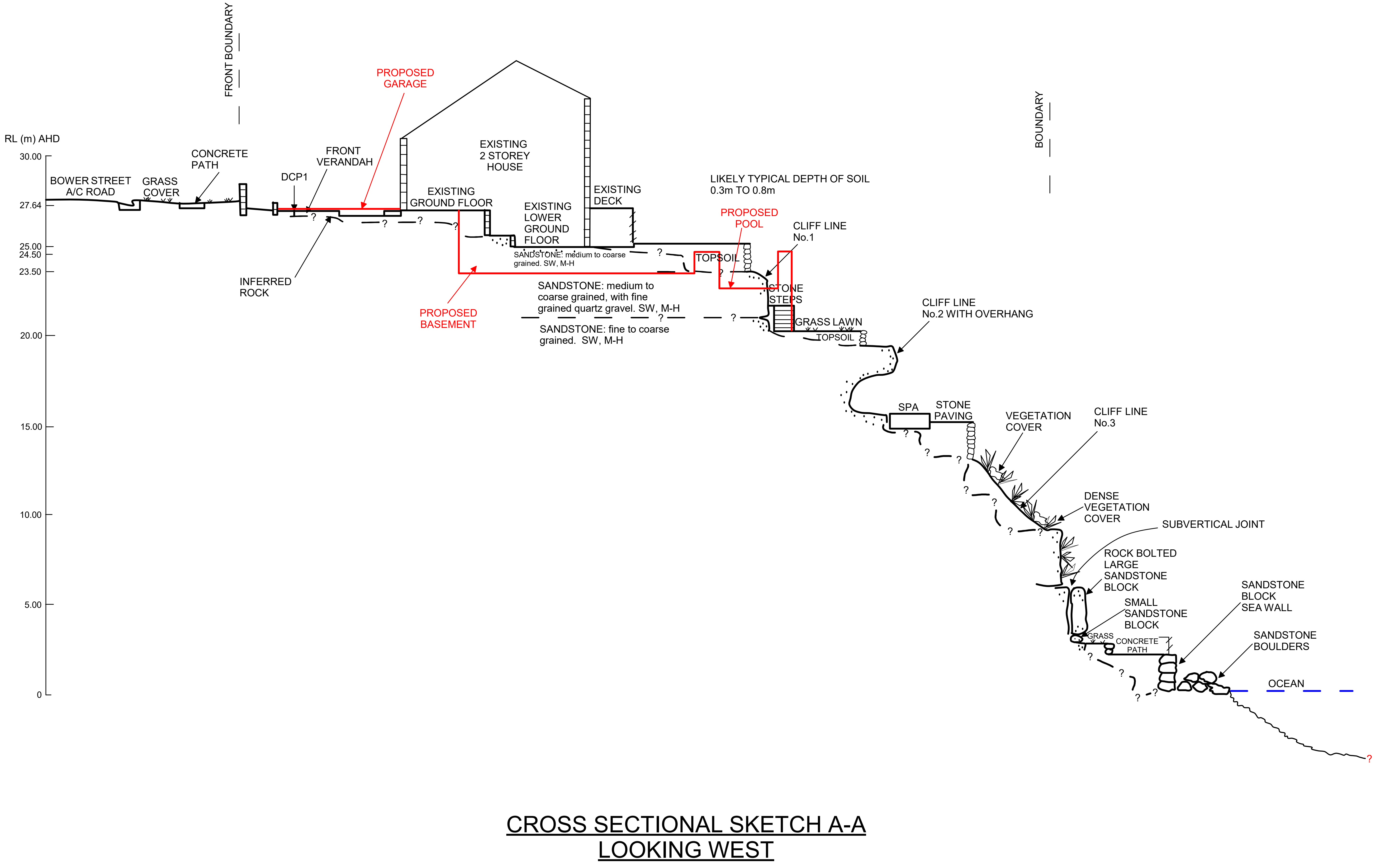
Title: **INVESTIGATION LOCATION PLAN**

Location: 44 BOWER STREET
MANLY, NSW

Report No: 29343SM2 Rev2

Figures No: 2

JKGeotechnics



CROSS SECTIONAL SKETCH A-A
LOOKING WEST

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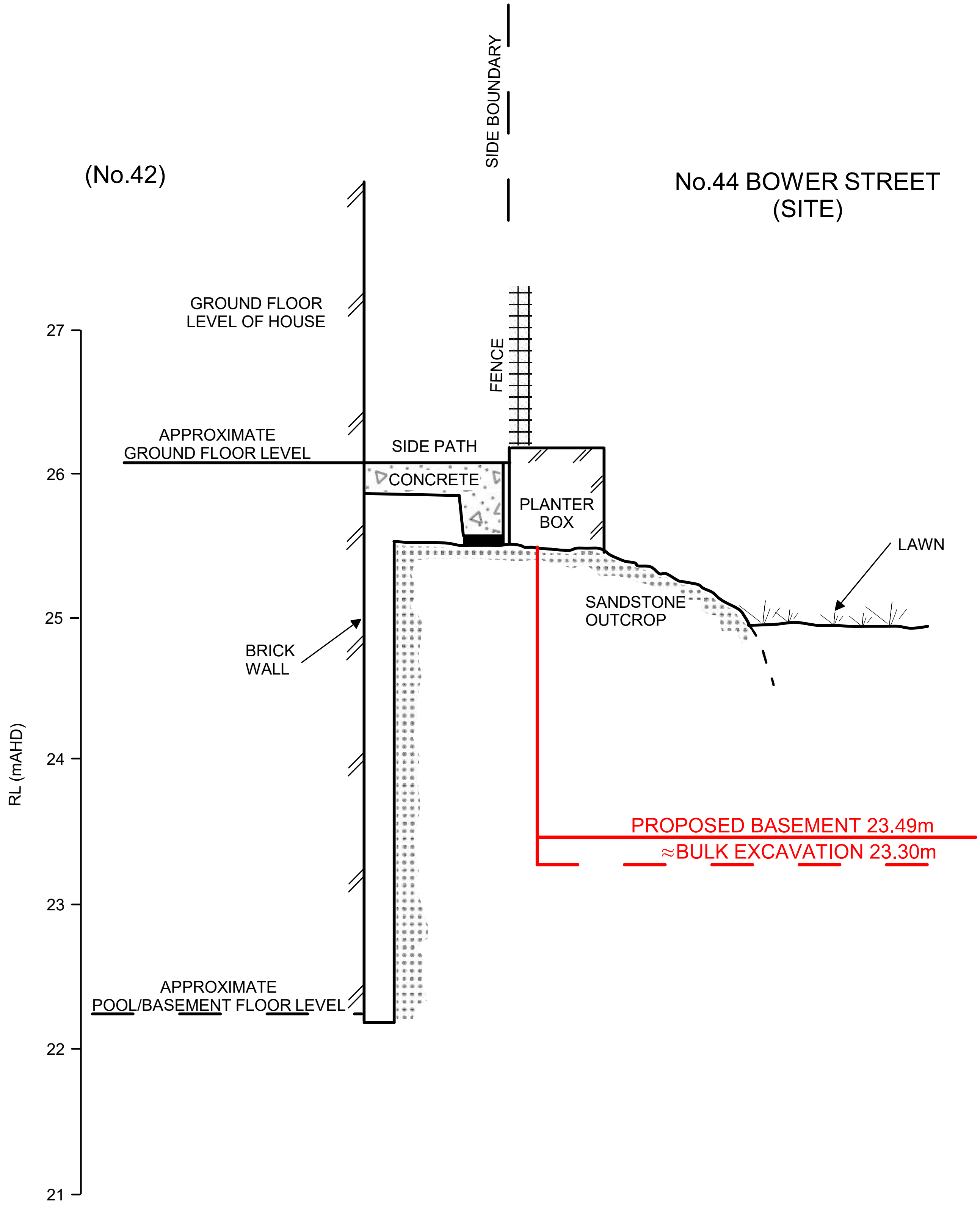
Report No. 29343SM2 Figure No. 3



SKETCH SECTION B-B
(LOOKING SOUTHWARDS)

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Report No. 293435SM2

Figure No. 4

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 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

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

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

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

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

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

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

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

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

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

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

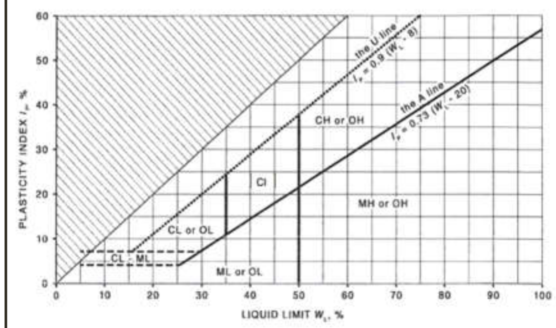
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



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LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.		
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.		
		Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	Sample taken over depth indicated, for environmental analysis.		
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.		
	DB	Bulk disturbed sample taken over depth indicated.		
	DS	Small disturbed bag sample taken over depth indicated.		
	ASB	Soil sample taken over depth indicated, for asbestos analysis.		
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.		
	SAL	Soil sample taken over depth indicated, for salinity analysis.		
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	VNS = 25	Vane shear reading in kPa of undrained shear strength.		
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).		
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained Soils)	D	DRY – runs freely through fingers.		
	M	MOIST – does not run freely but no free water visible on soil surface.		
	W	WET – free water visible on soil surface.		
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
Density Index/ Relative Density (Cohesionless Soils)			**Density Index (I_D) Range (%)**	**SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

Log Column	Symbol	Definition	
Remarks	'V' bit	Hardened steel 'V' shaped bit.	
	'TC' bit	Twin pronged tungsten carbide bit.	
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.	
	Soil Origin	The geological origin of the soil can generally be described as:	
		RESIDUAL	– soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED	– soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL	– soil deposited by creeks and rivers.
		ESTUARINE	– soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE	– soil deposited in a marine environment.
		AEOLIAN	– soil carried and deposited by wind.
		COLLUVIAL	– soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL	– beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $Is_{(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres