REPORT TO

WALTER PROJECTS PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED RESIDENTIAL PLANNING PROPOSAL

AT

1-31 WALTER STREET AND 452-462 WILLOUGHBY ROAD, WILLOUGHBY, NSW

Date: 23 August 2019
Ref: 32635PHrpt

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1 INTRODUCTION

This report presents the results of a geotechnical assessment for the proposed residential planning proposal at 1-31 Walter Street and 452-462 Willoughby Road, Willoughby, NSW. The location of the site is shown on Figure 1.

To assist with our assessment, we have been supplied with the following:

1. An unreferenced and undated site survey plan.
2. Another survey plan prepared by Axiom Surverying (Job No. 2616, Drawing No. 2616CO_KD.DWG, dated 7 April 2016). The survey datum is the Australian Height datum (AHD). This plan only shows the eastern portion of the site.
3. Architectural drawings by Architecture Urbaneia (Drawing Nos. A.050, A.051, A.100, A.101 to A.107, A.120, A.121, A.122, A.151 to A.155, A.155A, A.156, A.156A, A.157, A.158, A.158A, A.159, A.159A, A.160 to A.200, Issue A, dated 20 August 2019).
4. An email dated 20 August 2019 from Mr Mo Chehelnabi of Architecture Urbaneia confirming the lowest basement level for Buildings A and B.

With reference to the supplied architectural drawings, we understand that following demolition of the existing structures over the footprint of the proposed development, seven multi-storey residential apartment buildings (Buildings A to G) are proposed. The buildings will be underlain by either two or three basement levels, which will have finished floor levels between reduced level (RL) 48.25m (Buildings A and B) and RL52.63 (Building G), requiring excavation to a maximum depth of about 12m below existing grade. The outlines of the proposed buildings and basements are shown on the attached Figure 2.

The purpose of the geotechnical assessment was to complete a brief walkover inspection along the roads adjacent to the site, to review the results of our previous 2016 geotechnical investigation carried out over the south-eastern portion of the site and data from the published Sydney geological map, as a basis for providing preliminary comments and recommendations on excavation conditions, groundwater, shoring design, footing design, the lowest basement floor slabs and further geotechnical input.

A preliminary desktop environmental site assessment was carried out by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E30088KPrptRev2, for the results of the environmental site assessment.

2 ASSESSMENT PROCEDURE

2.1 Walkover Inspection

On 20 August 2019, our Senior Associate level Geotechnical Engineer (Adrian Hulskamp) carried out a walkover inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs from Walter Street, Willoughby Road and Scott Street (to the north of the site).

2.2 2016 Investigation Procedure

In 2016, an investigation comprising the drilling of five hand augered boreholes (BH1 to BH5) to depths between 0.7m and 2.2m below the existing ground surface was carried out over the south-eastern portion of the site. Dynamic Cone Penetration (DCP) tests were completed adjacent to each borehole to refusal depths between 0.7m (DCP2) and 2.8m (DCP4).

The borehole locations, as shown on Figure 2, were set out by tape measurements from existing surface features at the time of the original fieldwork. Figure 2 is based on a recent Nearmap image of the site. The surface RL's shown on the attached borehole logs were interpolated between spot level heights indicated on the supplied survey plan by Axiom, and are therefore approximate. The survey datum is AHD.

The relative compaction of the fill and strength/relative density of the subsoil profile were assessed from the DCP test results, augmented by hand penetrometer readings taken on cohesive soil samples recovered from the hand auger. Groundwater observations were made during and on completion of drilling of each borehole.

The borehole logs and DCP test results are attached, together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Our geotechnical engineer was present on a full-time basis during the fieldwork to set out the borehole locations, nominate the testing and sampling, prepare the borehole logs and record the DCP test results.

Laboratory geotechnical testing was not carried out as it was not deemed appropriate.

3 SITE OBSERVATIONS

The site is located towards the toe of a south-east facing hillside and mostly within an east-west trending gully feature. Flat Rock Creek is located along the base of the gully about 50m to the south of the site. Ground levels slope typically less than about 10°, apart from where the site 'steps' down to the south-east over the western portion of the site. Walter Street and Willoughby Road bound the site to the south and east, respectively.

At the time of our inspection, the site was occupied by several one and two storey houses, apart from 462 Willoughby Road which was vacant. The ground surface between the houses was generally covered by grass, driveways, landscaping and medium to tall trees.

Distinctly weathered sandstone bedrock of at least medium strength outcropped along the northern side of the footpath adjacent to 7 Walter Street. A sandstone cliff face up to about 5m high was visible in the rear yard of 27 Walter Street. The supplied survey plan also shows sandstone cliff lines in the rear yards of 19 to 25 Walker Street. Sandstone bedrock also appeared to be exposed in the rear yard of 21 Walter Street, when observed from Scott Street to the north of the site. Sandstone bedrock was also visible along the base of the gully about 30m to the south of the site, and along the northern side of Artarmon Road about 140m to the north of the site. No groundwater seepage was observed over the rock surfaces that were observed.

The neighbouring property to west of the site was vacant and occupied by dense bushland. The neighbouring properties to the north-east were occupied by several unit buildings and two in-ground swimming pools. The Channel 9 premises occupied the neighbouring property to the north-west of the site and comprised several buildings, open on-grade car parking areas, a telecommunications tower and ancillary equipment. Residential houses were located on the southern side of Walter Street opposite the site.

4 SUBSURFACE CONDITONS

The 1:100,000 Geological Map of the Sydney indicates the site to be underlain by Hawkesbury Sandstone, but close to the contact with man-made fill immediately to the east of Willoughby Road.

Our site observations have confirmed that sandstone bedrock outcrops on the site and within some of the neighbouring properties to the north and south of the site.

The previous 2016 investigation disclosed a subsurface profile comprising relatively shallow fill overlying natural clayey sands and sandy clays. The clayey sands were assessed to be of very loose and loose relative density and the sandy clays were assessed to be of firm to stiff strength. Refusal of the DCP tests have inferred sandstone bedrock surface at the test locations ranges between 0.7m to 2.8m depth. We note that since the DCP tests do not provide sample recovery, the material on which refusal has occurred has not been confirmed.

Groundwater seepage was encountered in some of the previous boreholes at depths between about 0.5m and 1.2m. Our previous investigation was carried out following a period of wet weather, and so the seepage does not necessarily represent a permanent condition.

Based on our site observations and noting the results of our previous 2016 investigation, we expect the majority of the site to be underlain by relatively shallow soil in areas where the sandstone does not outcrop, apart from areas towards the eastern end of the site, where the bedrock could be encountered at depths in the order of about 3m. Groundwater seepage should be expected within the excavations, particularly during and after heavy or prolonged rainfall.

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

5.1 Geotechnical Investigation

The comments and recommendations provided in this report are based on a walkover inspection along the adjoining roads to the south, east and north and reference to the results of our limited scope 2016 investigation completed over the south-eastern corner of the site, and therefore must be considered to be generalised.

For the purpose of this report, we have assumed that sandstone bedrock of at least low strength will generally be encountered within about 3m depth.

Prior to finalising the structural design, a comprehensive geotechnical and hydrogeological investigation comprising the drilling of cored boreholes must be completed to confirm the subsurface profile, including the depth and quality of the underlying sandstone bedrock and presence of groundwater by installation of monitoring wells. The recommendations provided in this report will then need to be reviewed and updated accordingly. For adequate site coverage, we anticipate that the full scope of the investigation can only be completed after demolition of the existing houses when access is available for a drill rig. We can provide a fee proposal for this additional work, if requested to do so.

5.2 Suitability of the Site For Development

In our opinion, we consider that the proposed development is geotechnical feasible for this site, provided the comments and recommendations below are adopted in their entirety and a comprehensive geotechnical investigation is completed.

The proposed development will incorporate common construction techniques and methodologies carried out on many nearby sites, as well as on sites throughout Sydney and surrounds.

5.3 Roads and Maritime Services (RMS)

Willoughby Road bounds the site to the east, which we understand is an RMS asset. In our experience and for an excavation of this size, depth and proximity to the road, RMS may require numerical analyses of the proposed excavation sequencing and shoring system to assess the potential impact of the proposed works on Willoughby Road.

The RMS may also require the installation and subsequent monitoring of borehole inclinometers.

Reference should be made to the RMS Technical Direction document, Reference: GTD20012/001, dated 27 April 2012 for further information.

Once any RMS requirements have been confirmed, we can prepare a proposal to assist with the geotechnical requirements, if requested.

5.4 Dilapidation Surveys

Prior to the commencement of demolition, we recommend that dilapidation surveys be completed on the buildings over the southern half of the neighbouring properties to the north, and on the houses located along the southern side of Walter Street. The dilapidation surveys should also include any boundary walls and surrounding pavements.

The dilapidation survey reports can be used as a benchmark against which to set vibration limits for rock excavation and for assessing possible future claims for damage arising from the works.

The respective owners of the adjoining buildings should be asked to confirm in writing that the dilapidation report on their property presents a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc) and photographed where possible.

5.5 Excavation

Prior to any excavation commencing, reference should be made to the Safe Work Australia 'Excavation Work Code of Practice' dated July 2015.

Following demolition of the existing buildings and pavements and removal of vegetation within the development footprint, any deleterious fill should be stripped and disposed appropriately off-site. Reference should be made to the JKE report for guidance on the off-site disposal of soil.

Excavation of fill, natural soils and sandstone bedrock up to very low strength may be carried out using a bucket attached to a hydraulic excavator, with assistance using a ripping tyne to break any low or medium strength bands that are no thicker than about 0.3m. Sandstone bedrock of low or higher strength will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, rock saws or ripping tynes. Such equipment would also be required for trimming rock faces and for detailed rock excavations, such as for footings, trenches, lift pits, etc.

For the larger common basement below proposed buildings A, B, C and D, it may also be feasible to rip the sandstone using a large dozer, but to increase excavation productivity particular if the rock is of at least high strength, a generous allowance should be made for hydraulic rock hammer assistance in conjunction with the ripping. As a guide, ripping of Class II and III sandstone bedrock will be possible with a Caterpillar D10 dozer. Confirmation on the dozer size should be made following completion of the geotechnical investigation.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to nearby structures and buried services. We recommend that quantitative vibration monitoring be carried out whenever hydraulic rock hammers are used during demolition and excavation on this site, as a precaution against possible vibration induced damage. The vibration limits should be set by the structural engineer following their review of the dilapidation reports so that any particular sensitivities of the nearby structures can be accounted for. However, unless any of the structures are particularly sensitive, we expect that a peak particle velocity of 5mm/sec would be applied. It should be noted that when vibration limits are 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, it would be necessary to change to alternative equipment, such as a smaller rock hammer, rotary grinders, rock saws or ripping tynes.

We recommend the 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 and should be provided with a full copy of this report and the subsequent geotechnical investigation report.

5.6 Groundwater

During our current site visit, we did not observe any obvious groundwater seepage through the sandstone cliff line on the western side of the site where it was observed from the street, nor did we observe any seepage over the other visible bedrock exposures on and near to the site as described above in Section 3.

However, groundwater seepage was encountered in the soil profile in the boreholes we drilled as part of our 2016 investigation which were located over the lower eastern portion of the site.

As the site is within a gully, groundwater inflows should be expected into the excavation as local seepage flows within fill, at the fill/natural soil and soil/bedrock interfaces, and through joints and bedding partings within the bedrock, particularly during and after or prolonged heavy rain.

Further advice on groundwater inflows and managing those inflows should be provided as part of the detailed geotechnical investigation yet to be completed. However, assuming seepage does occur, it is likely that in most areas it will be of a small flowrate and controlled during construction by sump and pump methods or gravity drainage to divert it to the stormwater system for disposal.

In the long term, drainage should be provided behind the basement retaining walls and below the basement floor slabs. The completed excavation should be inspected by the hydraulic consultant to assess if the designed drainage system is adequate for actual seepage flows.

Following completion of the detailed investigation, it may be necessary to undertake seepage analysis and obtain a dewatering licence from Water NSW. If such a licence cannot be obtained, it may be necessary to 'tank' the basements.

5.7 Shoring Design

Where excavations through soil are less than about 3m deep and sufficient space is available within the site, temporary batters may be used provided they are no steeper than 1 Vertical (V) in 1 Horizontal (H) for clayey soils and sandstone bedrock up to very low strength and 1V in 1.5H for sandy soils. All surcharge loads, including construction loads, must be kept well clear of the crest of these temporary batters. All stormwater runoff should be directed away from the temporary batters to reduce erosion.

Where excavations through soil are deeper than 3m, or there is insufficient space within the site for battering, or where batters are not preferred, a shoring system to support the soils and upper more weathered bedrock to very low strength should be installed prior to the start of excavation. Soldier pile walls are envisaged, with possible timber lagging between the piles for more sandy soil areas. The piles could then be terminated above bulk excavation level in at least low strength rock, provided there is sufficient temporary toe support, such as by the installation of another lower row of temporary anchors or toe bolts.

Sandstone bedrock of low and higher strength may be cut vertically, subject to geotechnical inspections of the cut faces at not more than 1.5m depth intervals. However, localised stabilisation measures may be necessary if adverse defects such as inclined joints are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, etc. Clay seams in any permanently exposed sandstone cut faces may also require treatment by excavating into and then dry packing the seams with cementitious mortar, with regular weep holes to allow the dissipation of any water pressures in the seams.

Walls of no more than 3m in height may be designed as cantilevered walls using a conventional triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient, K_0, of 0.6 and a bulk unit weight of 20kN/m^3. For excavations in soil or rock up to very low strength which are deeper than 3m, the walls may need to be anchored or internally propped as the excavation proceeds to reduce deflections. If anchors are to be installed, then permission must be sought from the respective property owners, prior to installation.

Propped retaining walls may be designed based on a trapezoidal lateral earth pressure distribution of 6H kPa for the retained profile, where 'H' is the retained height of soil and weathered rock in metres. This pressure should be assumed to be uniform over the central 50% of the pressure distribution, tapering to zero at the crest and toe of the wall.

The above coefficient and pressure assume horizontal backfill behind the wall and inclined backfill must be taken as a surcharge load. Any surcharge affecting the walls (eg. inclined backfill, construction plant, etc) must be allowed for in the design. The retaining walls should be designed as drained and measures taken to induce complete and permanent drainage of the ground behind the walls.

For piles embedded into sandstone bedrock below bulk excavation level, an allowable lateral toe resistance of 250kPa may be adopted, assuming the bedrock is of at least low strength. All localised excavations in front of the wall, such as for buried services, footings, lift pit etc, must be taken into account in the embedment depth design, together with an appropriate allowance for over excavation of the greater of 10% of the retained height or 0.5m, whichever is higher.

Temporary anchors bonded into at least low strength sandstone bedrock may be designed for an allowable bond stress of 200kPa. The anchors should have a bond length of at least 3m and free length of at least 4m, with the bond located beyond a 45° line inclined up from the toe of the shoring system. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer independent of the anchor contractor, with anchors 'locked off' at about 85% of the design working load. Lift-off tests should be carried out on at least 10% of the anchors 3-4 days following locking off to confirm that the anchors are holding their load. We recommend that only experienced contractors be considered for the anchor installation.

5.8 Footing Design

Based on our expectation that sandstone bedrock of at least low strength will be encountered within about 3m of existing grade, sandstone bedrock is expected at bulk excavation level within the basement excavations. Therefore, for uniformity of support, we recommend that all footings be founded within the sandstone.

Pad and strip footings and any perimeter shoring piles founded in sandstone bedrock may be provisionally designed for an allowable end bearing pressure of 1,000kPa, provided a selected number of footing excavations are inspected by a geotechnical engineer prior to pouring.

It is more likely that sandstone bedrock of about Class II or Class III quality would be encountered at bulk excavation level. Therefore, following the detailed investigation with cored boreholes, and depending on the amount of site proving during construction, it is likely that allowable (serviceability) bearing pressures of about 3,500kPa to 6,000kPa could be adopted.

If any of the above ground portions of the proposed building extend outside the footprint of the proposed basement, these must be supported by suspended slabs on footings founded within sandstone bedrock below a 45° line drawn up at from the base of the adjacent excavation.

All piles/footings should be drilled/excavated, cleaned out, inspected and poured with minimal delay.

For minor structures, such as landscape walls, fences etc, could be founded on footings in the soil profile for a provision maximum allowable bearing pressure of 40kPa, due to the presence of firm clays. Following the detailed investigation and assessment of the soils across the site, this bearing pressure may be able to increase to a maximum of 100kPa.

5.9 Basement Floor Slabs

Based on the expectation that the basement floor slabs will directly overlie sandstone bedrock, a gravel sub-base layer should be placed below the floor slab to act as a separation layer between the slab and sandstone, and also an underfloor drainage layer. This should comprise a strong, durable, single-sized aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead any groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the basement concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

5.10 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Comprehensive geotechnical investigation, including cored boreholes and monitoring of groundwater wells to update this report.
- Review of the dilapidation survey reports.
- Witnessing of proof testing of temporary anchors.
- Progressive cut face inspections as excavations proceeds.
- Groundwater seepage monitoring.
- Footing/pile inspections.

6 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the investigation, design and construction phases 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.

This report provides preliminary advice on geotechnical aspects for the proposed civil and structural design and are to be confirmed following a geotechnical investigation at the site. Contract Documents and Specifications should only be prepared based on our report following completion of a geotechnical investigation at the site.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

PLOT DATE: 20/08/2019 1:33:25 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32635PH WILLOUGHBY\CAD\32635PH.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 20 40 60 80 100

SCALE 1:2000 @A4 METRES

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

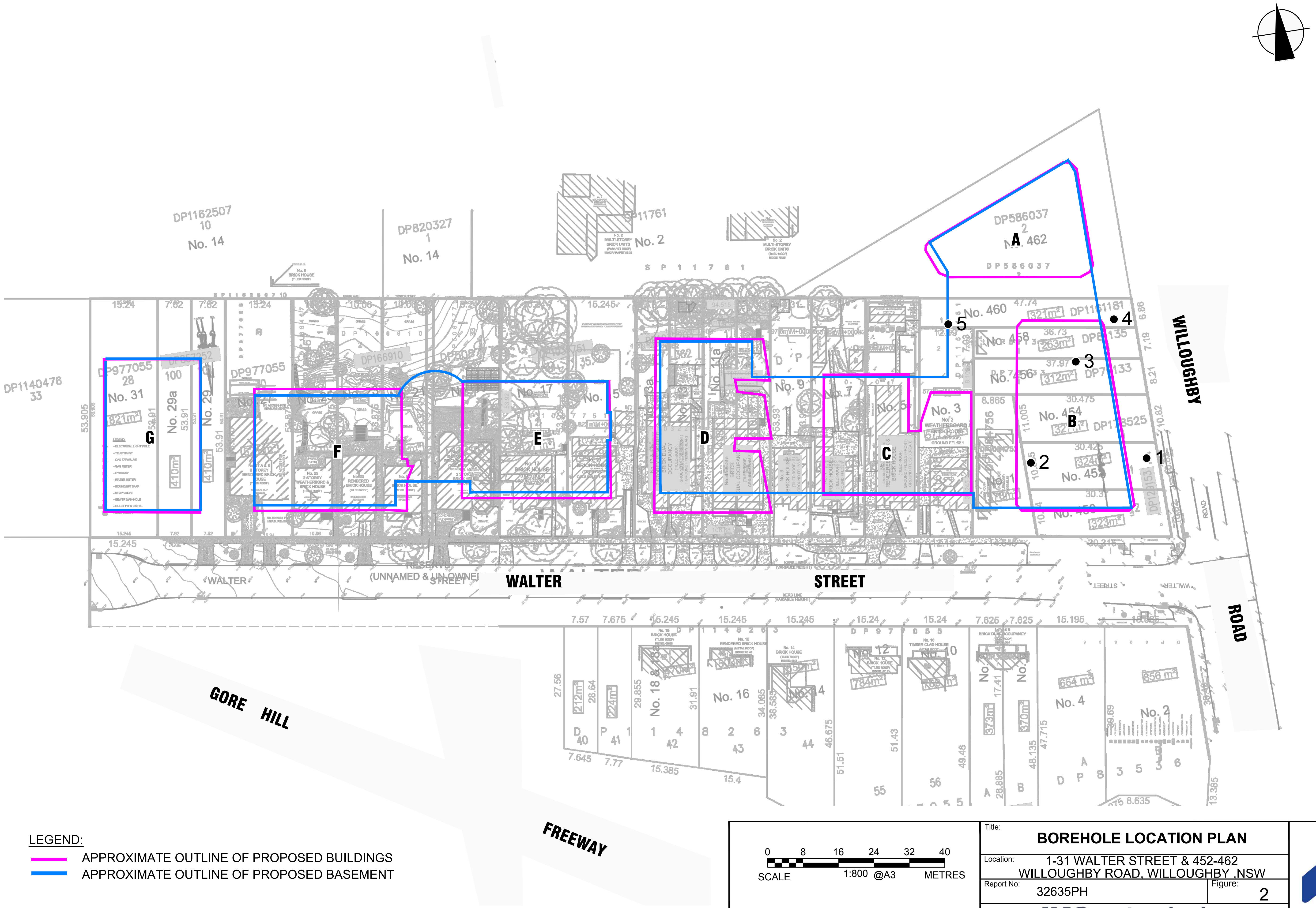
Title: **SITE LOCATION PLAN**

Location: 1-31 WALTER STREET & 452-462 WILLOUGHBY ROAD, WILLOUGHBY ,NSW

Report No: 32635PH

Figure: 1

JKGeotechnics



LEGEND:

APPROXIMATE OUTLINE OF PROPOSED BUILDINGS

APPROXIMATE OUTLINE OF PROPOSED BASEMENT

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

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

Title: **BOREHOLE LOCATION PLAN**

Location: 1-31 WALTER STREET & 452-462 WILLOUGHBY ROAD, WILLOUGHBY ,NSW

Report No: 32635PH

Figure: 2

JKGeotechnics

PLOT DATE: 20/08/2019 3:43:45 PM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32635PH WILLOUGHBY\CAD\32635PH.DWG

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

<table>
<tr><th rowspan="3">Group</th><th rowspan="3">Type of Structure</th><th colspan="4">Peak Vibration Velocity in mm/s</th></tr>
<tr><th colspan="3">At Foundation Level
at a Frequency of:</th><th>Plane of Floor of Uppermost Storey</th></tr>
<tr><th>Less than 10Hz</th><th>10Hz to 50Hz</th><th>50Hz to 100Hz</th><th>All Frequencies</th></tr>
<tr><td>1</td><td>Buildings used for commercial purposes, industrial buildings and buildings of similar design.</td><td>20</td><td>20 to 40</td><td>40 to 50</td><td>40</td></tr>
<tr><td>2</td><td>Dwellings and buildings of similar design and/or use.</td><td>5</td><td>5 to 15</td><td>15 to 20</td><td>15</td></tr>
<tr><td>3</td><td>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).</td><td>3</td><td>3 to 8</td><td>8 to 10</td><td>8</td></tr>
</table>

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 or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

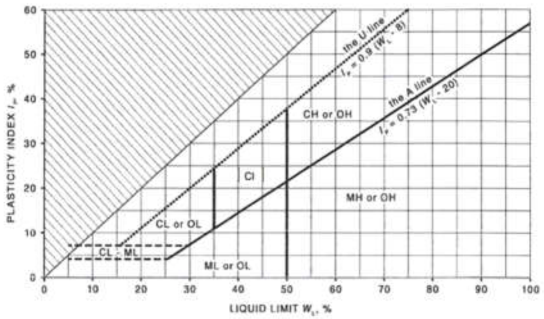
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		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.
Extremely Weathered		XW		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.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	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.
Moderately Weathered		MW		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.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

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 $I_{s(50)}$ (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

APPENDIX A

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client:
Project:
Location: 1 WALTER STREET, WILLOUGHBY, NSW

Job No. 29507S **Method:** HAND AUGER **R.L. Surface:** ≈ 50.1m
Date: 17-6-16 **Datum:** AHD
Logged/Checked by: T.C./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark grey and dark brown, with roots and root fibres.	M			
						FILL: Sandy clay, low plasticity, orange brown, yellow brown and dark grey, with fine to coarse grained ironstone gravel.				APPEARS POORLY COMPACTED
			0.5							
▶			1							
					SC	CLAYEY SAND: fine to medium grained, grey and brown.	W	VL-L		
			1.5							
					CL	SANDY CLAY: low plasticity, orange brown and grey.	MC≈PL	F-St	100	
			2			END OF BOREHOLE AT 2.0m				
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No. 2
1/1

Client:
Project:
Location: 1 WALTER STREET, WILLOUGHBY, NSW

Job No. 29507S **Method:** HAND AUGER **R.L. Surface:** ≈ 50.6m
Date: 17-6-16 **Datum:** AHD
Logged/Checked by: T.C./P.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Clayey sand topsoil, fine to coarse grained, dark brown, with roots.	M			GRASS COVER
								SC	CLAYEY SAND: fine to coarse grained, dark brown.	M	VL-L		
						0.5		CL	SANDY CLAY: medium plasticity, light grey.	MC≈PL		100 110	
									END OF BOREHOLE AT 0.7m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No. 3

1/1

Client:

Project:

Location: 1 WALTER STREET, WILLOUGHBY, NSW

Job No. 29507S **Method:** HAND AUGER **R.L. Surface:** ≈ 50.0m

Date: 17-6-16 **Datum:** AHD

Logged/Checked by: T.C./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		REFER TO DCP TEST RESULTS	0			FILL: Clayey silty sand, fine to course grained, dark grey, with root fibres.	M			CONCRETE PAVER
						FILL: Gravelly sand, fine to coarse grained, orange brown and grey, with fine to coarse grained sandstone gravel and sandstone cobbles.				APPEARS POORLY COMPACTED
			0.5							
			1		SC	CLAYEY SAND: fine to medium grained, dark brown.	M	VL-L		
			1.5			CLAYEY SAND: fine to medium grained, orange brown and light grey.				
			2			END OF BOREHOLE AT 2.0m				
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No. 4

1/1

Client:

Project:

Location: 1 WALTER STREET, WILLOUGHBY, NSW

Job No. 29507S | **Method:** HAND AUGER | **R.L. Surface:** ≈ 51.2m

Date: 17-6-16 | **Datum:** AHD

Logged/Checked by: T.C./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		REFER TO DCP TEST RESULTS	0			FILL: Sandy clay topsoil, low plasticity, dark grey and brown, with root fibres.	MC≈PL			APPEARS POORLY TO MODERATELY COMPACTED
			0.5			FILL: Clayey gravelly sand, fine to coarse grained, orange brown, with fine to coarse grained sandstone gravel and sandstone cobbles.	M			
▶			1		SC	CLAYEY SAND: fine to medium grained, brown.	W	VL-L		
			1.5			CLAYEY SAND: fine to medium grained, orange brown and grey.				
			2							
			2.5			END OF BOREHOLE AT 2.2m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
			3							
			3.5							

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BOREHOLE LOG

Borehole No.
5
1/1

Client:
Project:
Location: 1 WALTER STREET, WILLOUGHBY, NSW

Job No. 29507S **Method:** HAND AUGER **R.L. Surface:** ≈ 51.6m
Date: 17-6-16 **Datum:** AHD
Logged/Checked by: T.C./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to coarse grained, dark brown, with roots fibres.	M			GRASS COVER
					SC	CLAYEY SAND: fine to coarse grained, brown, with root fibres.	M	VL		
			0.5		CL	SANDY CLAY: low to medium plasticity, orange brown and grey.	MC>PL	F	70	
			1			END OF BOREHOLE AT 0.9m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
			1.5							
			2							
			2.5							
			3							
			3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:			
Project:			
Location:	1 WALTER STREET, WILLOUGHBY, NSW		
Job No.	29507S	Hammer Weight & Drop: 9kg/510mm	
Date:	17-6-16	Rod Diameter: 16mm	
Tested By:	T.C.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL ~50,1m	RL ~50.6m	RL ~50.0m	RL ~51.2m	RL ~51.6m		
Depth (mm)	1	2	3	4	5		
0 - 100			5	2			
100 - 200		1	1	5	2		
200 - 300	1	2	2	4	4		
300 - 400		1	1	1	3		
400 - 500	1	1	1	1	1		
500 - 600	1	3	2	3	3		
600 - 700	1	1	3	2	1		
700 - 800	2	REFUSAL	1	3	2		
800 - 900	1		1	4			
900 - 1000	1		1	2	1		
1000 - 1100	1		1	4	4		
1100 - 1200	12		2	3	4		
1200 - 1300	6		3	1	REFUSAL		
1300 - 1400	4		1	1			
1400 - 1500	4		1	2			
1500 - 1600	5		2	1			
1600 - 1700	5		1	3			
1700 - 1800	5			1			
1800 - 1900	2		1	2			
1900 - 2000	3		1	1			
2000 - 2100	9		3	3			
2100 - 2200	11		4	4			
2200 - 2300	10		REFUSAL	4			
2300 - 2400	14			4			
2400 - 2500	REFUSAL			2			
2500 - 2600				1			
2600 - 2700				6			
2700 - 2800				4			
2800 - 2900				REFUSAL			
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
3. Survey datum is AHD.