REPORT

TO

DAP WOODLAND PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED COMMERCIAL AND RESIDENTIAL DEVELOPMENT

AT

26 WHISTLER STREET, MANLY, NSW

13 June 2019

Ref: 32250SMrpt Rev2

JK Geotechnics
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1 INTRODUCTION

This report presents the results of a geotechnical desktop assessment for a proposed commercial and residential development. A site location plan is presented as Figure 1. The assessment was commissioned by Mr Michael Stanton of Lighthouse Project Group on 25 February 2019 on behalf of DAP Woodland Pty Ltd and was carried out in accordance with our proposal, Ref. P48937S dated 13 February 2019.

From review of the architectural drawings (Proj. 21813 No's. DA 01 to 13, dated 7 June 2019, prepared by Wolski and Coppin), we understand the development includes the following:

- Demolition of existing structures (no known basements)
- Construction of a single basement parking level with finished floor level ranging from RL2.8m to RL3.30m (AHD). The basement will extend to the full extent of the site boundaries. We note the rear boundary has an irregular alignment. Excavation to about 3.3m depth below existing surface levels will required.
- Construction of a ground floor (mixed use) level plus 7 residential stories above.

The purpose of the assessment was to obtain geotechnical information on likely subsurface conditions as a basis for comments and recommendations on excavation, retention, groundwater, footings, slabs on grade and site specific geotechnical investigation which will be required for detailed design following the DA stage.

2 ASSESSMENT PROCEDURE

The assessment involved the following procedure:

- A desk top study of our nearby geotechnical investigations,
- Review of the published information including geological maps
- A walkover of the site and surrounds by our Associate Geotechnical Engineer on 27 February 2019.

The proposed basement outline is overlain on an extract of the supplied Bee and Lethbridge survey plan (Ref. 21156 dated January 2019) and is presented as Figure 2. The datum for reduced levels (RLs) is the Australian Height Datum.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is in a relatively flat low lying area about 150m east of the toe of the hillside (down which Sydney Road runs) and has a similar set back from Manly Beach.

The site measures approximately 30m by 36m with ground surface levels of about RL5.8m to RL5.9m. It is currently occupied by a 3 storey apartment building over a ground floor parking level. The carpark is concrete surfaced. There is a narrow vegetation strip along the site's frontage with Whistler Street that contains small and medium sized trees. On the eastern side of the property is a paved 'right of way' path linking through to Short Street.

To the south of the site is a 6 storey cement rendered building. It does not appear to have any basement levels.

To the north of the site is a 2 storey brick building (an Energy Australia sub-station) adjacent to the boundary. This does not appear to have any basement levels.

To the rear of the site adjoining the walkway are garden beds near the corners and Short Street most of which has been 'pedestrianised' with stone paving and a lawn. Beneath Short Street is a basement car park accessible via a ramp and garage-like entrance from the Raglan Street end of the 'street'). The basement appeared to be single level and supported by contiguous concrete piles approximately along the common boundary, which was somewhat irregular (stepping eastwards from its initial alignment). The number of basement levels and their plan layout should be confirmed by survey or further investigation.)

3.2 Likely Subsurface Conditions

The 1:100,000 Sydney geological map indicates the site to be underlain by a channel of Quaternary period medium to fine marine sand. The hillside to the west of the site is underlain by Hawkesbury Sandstone.

We have completed several deep geotechnical investigations at sites within the same geology and within an area stretching about 500m to the north and 150m west and south-west of the site. Investigation techniques included cone penetrometer tests (CPTs), dilatometer testing, boreholes with standard penetration tests (SPTs) and coring of bedrock, and long term groundwater level monitoring. We have also completed shallow investigations with augered boreholes with SPT closer to the site including to the north and south.

In summary, a deep sandy soil profile was encountered comprising mostly sands and silty sands over sandstone bedrock, with groundwater a meter or two above ‘sea level’.

Beneath a limited depth of fill, silty sandy soils were initially very loose to loose. The relative density from about 3m was variable, often increasing with depth to medium dense or denser but at some locations very loose sand extended to greater depth (to 9m at one test location on Manly Oval). Some silty clay and clayey silt bands were interpreted to be present.

West of the site, sandstone bedrock was inferred to be present from CPT tests at depths ranging from 20m to 34m below ground surface levels. To the north rock was also inferred to be present from depths of 30m to 32m. To the south-west of the site (Cnr of West Promenade and Sydney Road), rock was in the range of 12m to 22m depth. Where it was encountered in boreholes, the upper few metres of sandstone was highly variably weathered with strengths ranging from extremely low to medium strength. Some units of typically extremely weathered interbedded sandstone and shale were also encountered.

Groundwater was encountered at 4.7m depth at the corner of Raglan and Whistler Streets (approximately RL1.3m) and was more accurately recorded over a longer period of time in the range between RL1.1m and RL1.3m under the eastern side of Manly Oval.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Considerations

All comments and recommendations are based on an assumed subsurface profile from information beyond the site and therefore should be reviewed by JK Geotechnics once geotechnical investigations are completed. Further details on geotechnical investigation for detailed design are discussed below.

About 2.8m to 3.2m of excavation is required for the proposed basement which is within the zone of influence of existing buildings of various scale, construction type and period. Some of them may be founded in very loose sands, and their footings may protrude onto the existing site. The principal geotechnical considerations will be how to maintain stability to neighbouring structures and infrastructure during demolition of existing structures and excavation. Careful demolition, completion of dilapidation surveys, consideration to underpinning and installation of suitable shoring prior to excavation will be required.

A working platform for large tracked plant will also be required.

Groundwater is expected to be about 1.5m below the bulk excavation level but long term monitoring is advised from as early a stage as possible to determine the magnitude of fluctuations with changes of rainfall.

Given the expected very deep sandy profile, the assumed high column loads will have to be transferred to a suitable bearing stratum by grout injected continuous flight auger (CFA) piles or perhaps CSM barrette footings. Detailed geotechnical investigation will be required to identify such a stratum which is likely to be a medium dense or dense unit of sand.

4.1 Dilapidation Surveys

Dilapidation surveys should be completed on the adjacent properties, and perhaps infrastructure, prior to commencement of excavation or even demolition.

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 and Working Platforms

Demolition should be carefully planned and executed in accordance with a sequenced methodology prepared by the structural engineer and with consideration to keeping the concrete pavement which may provide a good base for a working platform (or perhaps prevent the need for one being constructed at all). A working platform assessment should be completed once the preferred tracked plant for footings and shoring are known.

Working platforms for large tracked plant are required where the subgrade is of insufficient bearing capacity. Very loose upper sands such as is expected on this site often have insufficient bearing capacity. Contractors often assume (in their contracts) that working platforms will be provided for them and this can be a significant cost and time item for developers. Geotechnical investigation for a working platform assessment will often require a number of DCP tests and shallow boreholes. Any test pits, holes from removal of pad footings, or trenches should be backfilled with cement stabilised sand or well compacted granular material to avoid soft spots which would present a serious instability hazard.

There is potential for transmission of vibrations from demolition works to impact on the neighbouring structures some of which may be on shallow footings on very loose to loose sand.

Vibrations emitted during excavation should be minimised to prevent potential settlement of loose sands beneath footings. We therefore recommend that existing site building footings and floor slabs are saw cut or otherwise broken into smaller manageable pieces rather than to be demolished by use of rock breakers, particularly where in close proximity to buildings on shallow footings.

Monitoring should be completed on the neighbouring buildings targeting ‘as low as reasonably practical’ vibrations, say not greater than 3mm/s peak particle velocity (PPV). If this vibration limit is repeatedly reached, lower impact techniques should be adopted. The impact of large masonry or concrete having been dropped to the ground, or even into trucks, can cause damaging vibrations.

4.3 Underpinning/Soil Improvement

As discussed above structures on shallow footings founded on very loose to loose sands are susceptible to settlement from vibrations during some demolition activities, movement of large plant and trucks, and soil decompression from shoring and pile installation. We therefore recommend that test pits are completed to investigate the footing system of the adjacent structures, the brick substation building in particular. If such shallow footing conditions are confirmed then consideration should be given to monitoring and ‘underpinning’. Monitoring could be in the form of high accuracy surveying of prisms etc. Underpinning could be in the form of permeation grouting or chemical grouting to control settlement. Further advice should be sought in this regard once founding conditions are determined. Test pits should be inspected by a geotechnical engineer who may also recommend testing of the soil density by means of Dynamic Cone Penetration tests, or similar.

4.4 Shoring

Prior to excavation a shoring system must be installed to retain the soils and support the adjacent buildings. For a maximum 3m depth of excavation and without surcharges, a shoring wall sufficiently embedded to act in cantilever is feasible. Where surcharges are present such as buildings founded at shallow depth or live loads on roads, the wall may need to be anchored or propped. Props other than across corners would require internal piles to resist lateral movements. Anchors may not be feasible where there are adjacent basements such as to the east of the site or below the substation where underground cables/easements may be present. Cantilever piles are normally of greater diameter than anchored piles and architectural design should allow sufficient space for the shoring required. Permission will be needed from property owners where anchors extend onto their property. It can be a lengthy process to achieve the permission so we encourage this be started without delay, if required.

Top down construction is also feasible, given the assumed sandy material will be easily excavated. Obviously footing piles would have to be drilled from the surface prior to the slab being constructed. Top down construction has the advantages of reducing the risk of shoring wall deflection and therefore reduces the risk of damaging neighbouring buildings, but also allows construction of above ground levels to commence at an earlier stage.

Assuming a sandy profile with groundwater about 1.8m below bulk excavation level, the following shoring systems would be suitable.

- A contiguous shoring pile wall drilled using cased cement grout injected CFA piles. Without the casing, there is a greater risk of soil decompression occurring thus potentially damaging neighbouring buildings. To prevent soil loss, gaps between piles should either be packed with grout, or shotcreted.
- Cutter Soil Mix (CSM) wall. This system mixes cement with the existing sand and water to form 'concrete' panels insitu, into which steel reinforcement (usually 'I' beams) is added. The site is relatively small compared to the space normally required for this equipment so contractors should be consulted regarding the feasibility prior to committing to design. This technique also has the potential for soil decompression so further consideration should be given to underpinning the adjacent structures, prior to shoring works. CSM walls may not necessarily have the same lifespans as CFA piled walls. Internal reinforced shotcrete finishes can be added, or perhaps since the basement is expected to be above the groundwater level, contractors may provide sufficient design life warranties.

Only experienced contractors with appropriate experience and insurances should be engaged.

Any surcharge loads affecting the walls (e.g. buildings, traffic loading, construction loads etc) should be taken into account in the wall design, and these are additional to the earth pressures. We assume that permanent lateral support of the retaining walls will be provided by the new structure.

Design parameters can be provided following detailed geotechnical investigation, but for preliminary concept design a ‘worst case’ of the typical conditions could be assumed and would comprise very loose sands and a groundwater level say just below bulk excavation level.

Localised shoring may also be required for construction of the lift pit which is in the centre of the site and depending on its depth may protrude below the groundwater level requiring dewatering. Interlocking driven sheet piles may be appropriate given the 9m to 16m offsets from the boundaries. If sheet pies are adopted we recommend vibration monitoring be carried out during the installation. If vibrations are notable then lower vibration emitting shoring systems should be installed such as CFA secant pile walls.

If dewatering is anticipated the wall toe level must be designed following detailed seepage analysis to avoid a broader draw down profile which potentially may affect neighbouring structures.

4.5 Excavation Techniques

Excavation to about 3m in an assumed very loose to loose sandy profile should be readily achieved using buckets of hydraulic excavators and bobcats. Groundwater is expected to be about 1.8m below bulk excavation level.

All excavation should be in accordance with the NSW WorkCover ‘Excavation Code of Practice’ July 2015.

Excavation for the lift pit may encounter groundwater which would require localised dewatering. Any dewatering should be carried out in accordance with a detailed methodology designed by an engineer to prevent ‘boiling’, and other issues (discussed in Section 4.4) and approved by a geotechnical engineer independent of the contractor.

4.6 Footings

Detailed geotechnical investigation is critical to the design for the footings. We expect that there may be a medium dense or dense layer within the expected deep soil profile that may be suitable for embedment of piles. We recommend a minimum of 5 Cone Penetration Tests be carried out within the site to reduce the risk of unidentified soil conditions. Dilapidation testing may assist optimising pile design.

Footings will have to be cement grout injected CFA piles or perhaps, if CSM is used for shoring, then a CSM panel could be constructed (also known as a barrette) to save establishing a second large rig.

One advantage of CSM over CFA piles is that minimal spoil is generated for disposal, particularly if acid sulphate soils are present.

If loose soils are present and rock is not excessively deep (i.e. less than say 15m), then piled footings on rock could be an option. Cored boreholes would be required to provide detailed information on the rock strength and defects in order to optimise design bearing pressures.

4.7 Groundwater and Permeability

About 150m inland from the subject site, groundwater levels were at about RL1.1m to RL1.3m. The level reduced at sites closer to Manly beach. We therefore expect similar groundwater levels will be encountered at the subject site i.e. 1.8m to 2.0m below bulk proposed excavation level (RL3.1m), or 4.5 to 4.8m below ground surface levels.

Continuous groundwater level monitoring should be carried out to determine the groundwater levels on site and also fluctuations following long periods of rainfall. There may also be tidal fluctuations being 200m from the ocean. Piezometers with electronic data loggers should be installed without delay since the assumptions made regarding groundwater will significantly change geotechnical design concepts if the groundwater level is above or close to bulk excavation level.

Until the site is accessible for site specific infiltration testing, preliminary design of stormwater infiltration systems could be based on the typical hydraulic conductivity (permeability), K, for the expected natural soils. Based on past experience and published literature, permeability of sand to silty sands would typically be in the order of 10^{-4}m/s to 10^{-5}m/s but could range by a further one to two orders of magnitude depending on the silt fines content. Infiltration may also be affected by possible layers of clay and a varying groundwater level. Infiltration systems should also consider possible effects on adjacent basements. We recommend preliminary design values be revised following site specific testing when site access becomes available.

4.8 Subgrade Preparation and Slabs-on-Grade

We assume sandy soils will be present but layers of silt and clay may be present within the alluvial soil profile.

Slabs-on-grade are feasible above the groundwater level and would effectively be 'floating' independent of the superstructure. To confine the assumed sandy soils, a 100mm layer of crushed rock to RMS QA Specification 3051 (2013) unbound base material (or similar good quality and durable fine crushed rock) should be placed. The subgrade should then be prepared by rolling with a minimum 8 passes of a static smooth drum roller of not less than 7 tonnes to densify the near surface soils. No vibrations should be used due to the potential damage that could be caused to nearby structures. The final pass should be completed in the presence of a geotechnical engineer to check for the presence of any soft spots which usually indicates unsuitable soils. Should any soft spots be identified they should be excavated and replaced with good quality granular material compacted in thin layers until no noticeable deflection is observed.

The subbase layer should be compacted to at least 100% of its Standard Maximum Dry Density.

Trafficable concrete pavements should be designed with effective shear transmission at all joints by way of either dowelled or keyed joints.

Drainage must be provided below the basement slab and may comprise a grid of subsoil drains being strips of free draining gravel (single sized durable washed material) cut and placed within the subbase 100mm gravel layer. Alternatively the entire subbase layer may comprise free draining gravel (also known as a 'gravel blanket').

4.9 Detailed Geotechnical Investigation and Other Geotechnical Input

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

- Drilling of boreholes, installation of piezometers, groundwater level monitoring and infiltration testing
- CPT testing of the site soils and perhaps subsequent dilatometer testing
- Cored boreholes in the rock if the soil profile is of insufficient strength for piles, or if rock is shallower than anticipated
- Investigation/survey of adjacent basements (by others)
- Test pits for adjacent building footings
- Working platform assessment

- Consideration of underpinning or completing ‘ground improvement’ under any adjacent shallow footings, prior to shoring works
- Dilapidation surveys
- Review of shoring and footing design
- Inspection of initial shoring and footings
- Proof roll inspection of subgrade

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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the nearby boreholes and CPT tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

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

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018.

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

Title:	SITE LOCATION PLAN
Location:	26 WHISTLER STREET MANLY, NSW
Report No: 32250SM	Figure No: 1

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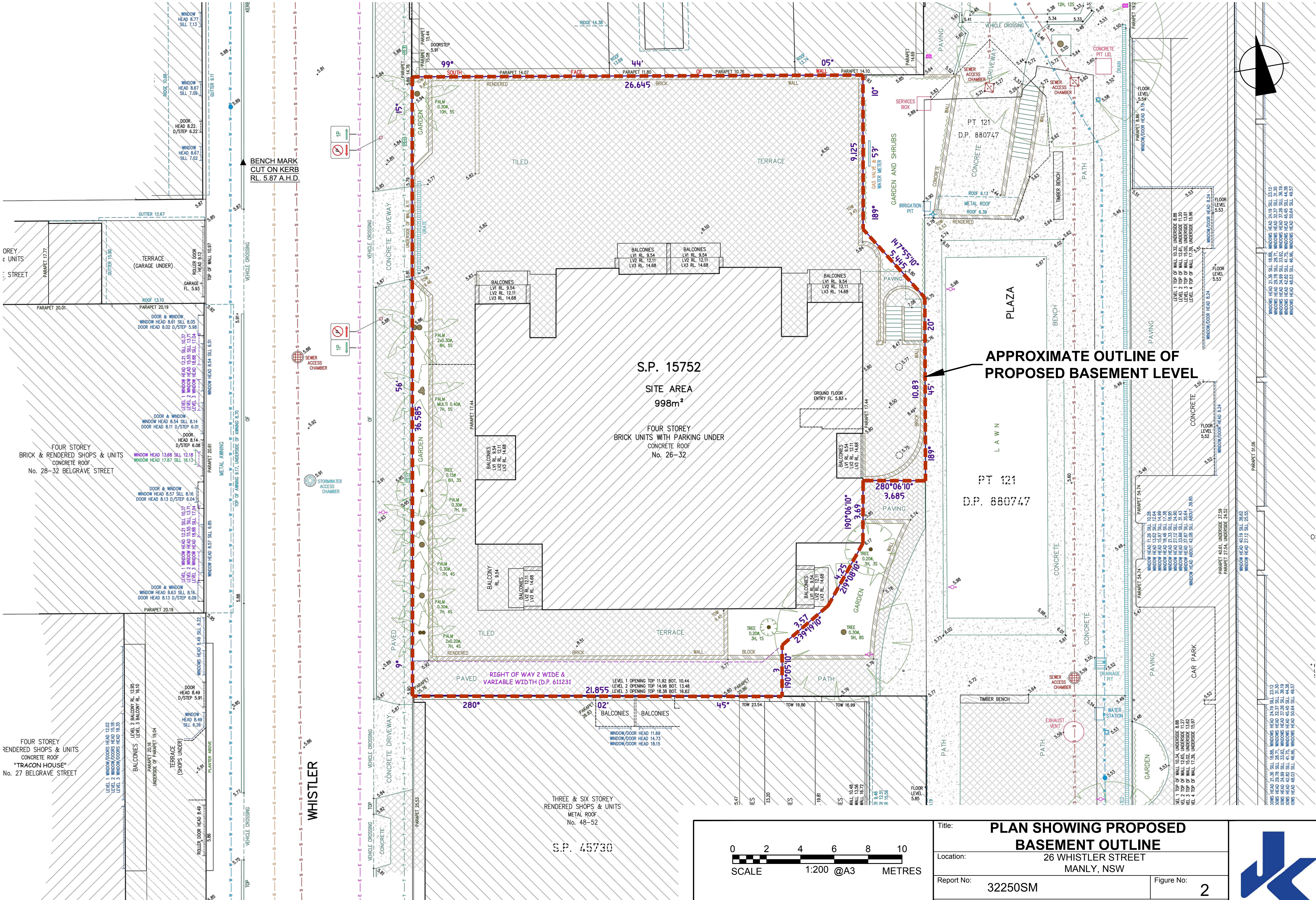

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This plan should be read in conjunction with the JK Geotechnics report.

Title: **PLAN SHOWING PROPOSED BASEMENT OUTLINE**

Location: 26 WHISTLER STREET MANLY, NSW

Report No: 32250SM

Figure No: 2

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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:

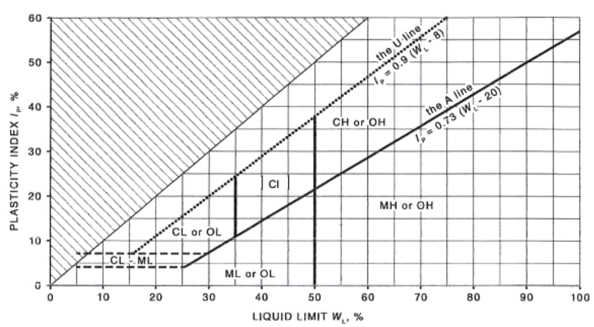
$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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.		

Log Column	Symbol	Definition
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)	Guide to Strength: 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 $\leq$ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres