REPORT TO

FRASERS PROPERTY

TELOPEA DEVELOPER LIMITED

ON

PRELIMINARY GEOTECHNICAL ASSESSMENT

FOR

PROPOSED HOUSING DEVELOPMENT

AT

TELOPEA, NSW

Date: 20 May 2020

Ref: 33079SCrpt2

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1 INTRODUCTION

This report presents the results of a desktop geotechnical assessment for the proposed masterplan housing renewal project at Telopea, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by Cameron Jackson of Frasers Property Australia and was carried out in accordance with our proposal dated 21 February 2020, Ref: P51260S.

In order to prepare our geotechnical assessment, we have been provided with the following documentation supplied by Frasers Property Australia and the architects Bates Smart;

- Preliminary masterplan basement car parking plan (20181205, no date)
- Annexure N-Staging Plan
- Survey plan by Craig & Rhodes (Ref:191-19, Amend No.01, dated 20 February 2020)

We have also been provided with the following documentation by the architects, Bates Smart (Project No. S12226) DA01.MP.000^{revB}, DA02.MP.101^{revB}, DA02.MP.110^{revA}, DA02.MP.120^{revB}, DA02.MP.200^{revA}, DA02.MP.211^{revB} DA02.MP.220^{revB}, DA02.MP.310^{revB}, DA02.MP.320^{revB} and massing core and potential visualisation wireframe drawings.

Based on the above information we understand that the proposed overall development will include construction of residential tower blocks at the various stages, as shown on the attached Figure 2. The tower blocks will range from about 8 storeys to 20 storeys high. At this stage we understand the residential tower blocks will have basement car parks of either 1-2 levels or 2-3 levels below existing ground levels. The proposed basement car parks are likely to require excavation to depths ranging from 3m to 9m depth below existing ground surface levels.

The purpose of our assessment is to review the available geotechnical information for the site and for nearby sites to assess the likely subsurface conditions and provide our comments and recommendations on geotechnical issues for the proposed development to assist with planning and preliminary concept design.

2 ASSESSMENT PROCEDURE

The assessment comprised an inspection of the site and its immediate surrounds by our Senior Engineering Geologist, Mr Thomas Clent, on 25 March 2020. Observations made during the inspection are summarised in Section 3.1.

A search of our project database was also carried out to find previous geotechnical investigations carried out on sites within the proposed housing renewal masterplan to assess the likely subsurface conditions. The results of our previous investigations are summarised in Section 3.2.

We have also used information from the Stage 1A geotechnical investigation dated 7 May 2020 (33079SC Stage 1A).

3 RESULTS OF ASSESSMENT

3.1 Site Observations

The site lies within undulating topography with overall southern to south-eastern facing slopes which fall at between 5° and 10° in some areas, though the upper western areas were relatively flat. The western parts of the site are more elevated with slopes falling to the lower eastern and southern areas. The development area is shown on the attached Figure 2.

At the time of the site walkover, the western (upper) portion of the site contained three walk-up tower blocks with 10 above ground levels. On the eastern side of the tower blocks was an above ground car park which was elevated by filling along its eastern side which was supported by a crib retaining wall. The crib retaining wall was approximately 2m in height and appeared in fair condition. Further to the east were trees along the Wade Street frontage.

Within the central and south-western portions of the site the residential buildings were predominantly three storeys high and constructed from brick with gently sloping grass areas interspersed between the buildings. Small to medium sized trees, concrete footpaths and asphalt surfaced car parks were also located along the various street frontages. The buildings, footpaths and car parks all appeared to be in fair condition, based on a cursory inspection. The Dundas Community Centre building was also located centrally within the site and comprised a two-storey blockwork building with an asphalt car park located on the north-western side of the building. The building had been cut into the sloping hillside on the western side and was suspended on the eastern side. Both the building and car park appeared to be in fair condition, based on a cursory inspection.

The residences located in the southern and northern margins of the site were predominantly single storey brick and weatherboard buildings with front and rear yards. However, some more recently renovated dwellings of up to two storeys were also observed. Some relatively recent residential developments up to six storeys constructed from brick/blockwork were located within the northern and southern parts of the site. At least one level of basement car parking was observed below the buildings.

A skate park, School and commercial shop properties were located in the central and eastern parts of the site. The commercial shop buildings were one and two storey structures and constructed from brick. The skate park was part of a larger park (Sturt Park) which sloped gently to the south-east. Medium to large sized trees were located around the perimeter. Concrete footpaths and slabs for the skate board equipment were located on the level areas. The Telopea Public School contained various buildings which were a mixture of brick and weatherboard construction; the building platforms appear to have been formed by cut and fill earthworks, a grassed fill batter was present on the south-eastern and eastern side of the sports field.

To the west of the site was the cutting for the Carlingford rail corridor. To the north of the site the topography climbed gently and the area was covered with predominantly single-storey residential lots. The lower lying areas on the southern and eastern sides of the site were predominantly parkland within which Seconds Ponds Creek flows north-eastwards to Rapanea Community Forest.

Each development stage will have individual boundary conditions which will have to be inspected and considered during the detailed geotechnical investigations.

3.2 Available Subsurface Information

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the majority of the site is mapped to be underlain by Ashfield Shale of the Wianamatta Group, but immediately to the east is the boundary with the underlying Hawkesbury Sandstone which underlies the parkland around Second Ponds Creek.

We have completed geotechnical investigations close to some of the proposed development areas and the results of these previous investigations are summarised below. The attached Figure 3 presents a graphical cross section showing the anticipated ground conditions.

Sturt Park Skate Board Facility

An investigation was carried out in 2006 within the northern portion of Sturt Park for a proposed skateboard ramp. The auger drilled boreholes encountered fill to depths ranging from 0.3m to 0.4m covering residual silty clay assessed to be of high plasticity and of hard strength. Sandstone bedrock was encountered below the residual clays at depths ranging from 1.2m to 2m and was assessed to be distinctly weathered and of low or low to medium strength.

7 Sturt Street

An investigation was carried out in December 2009 at 7 Sturt Street which is located over the central and southern eastern parts of the site. The cored boreholes drilled encountered fill to depths of up to 1.7m covering silty clay, sandy clay and clayey sand, with sandstone bedrock at depths ranging from 0.5m to 4.2m. The clayey soils were assessed to be of stiff to very stiff strength whilst the sandy soils were of medium dense relative density. The sandstone was locally capped by a thin layer of siltstone of low strength. In general, the sandstone was initially distinctly weathered and of low to medium strength improving to medium to high strength below depths ranging from 2m to 5m.

Groundwater was not encountered during or on completion of auger drilling. No longer term ground water monitoring was carried out.

1 to 5 Shortland Street

An investigation was carried out in December 2009 at 1 to 5 Shortland Street, which is located within the north-western portion of the site. The cored boreholes drilled encountered clayey fill to depths ranging from 0.5m to 0.8m covering residual silty clay of medium to high plasticity and of very stiff to hard strength. Weathered siltstone and sandstone bedrock were encountered at depths ranging from 1.6m to 2.8m, which were initially distinctly weathered and very low to low strength improving to medium to high strength below depths ranging from 2m to 4.7m. Some of the boreholes revealed a thin siltstone bedrock (Ashfield Shale) capping over the sandstone bedrock (Hawkesbury Sandstone).

Groundwater was not encountered during or on completion of auger drilling. No longer term ground water monitoring was carried out.

Polding Place-Telopea Stage 1A

The boreholes encountered fill in all boreholes to depths ranging from 0.2m to 1.2m over residual silty clay, which was assessed to be of medium to high plasticity and generally of hard strength. Below the residual soils, weathered siltstone was encountered at depths ranging from 1.7m to 4.2m, with the level of the surface of the rock falling towards the south and west from about RL58.5m to RL52.6m. The siltstone was initially assessed to be extremely weathered to distinctly weathered and of hard (soil strength) to very low strength, increasing to low to medium strength and then generally medium to high strength shortly thereafter.

Groundwater seepage was encountered at depths of 1.2m and 4.4m, during auger drilling. The groundwater levels were measured within the monitoring wells installed within BH1 and BH4 on the 29 April 2020 when levels of 2.6m and 3m respectively were recorded.

4 COMMENTS AND RECOMMENDATIONS

4.1 Inferred Subsurface Conditions

Based on the results of previous geotechnical investigations close to the subject site and within the development masterplan area as summarised above, we expect that the subsurface conditions below the site will comprise predominantly shallow fill covering predominantly residual silty clay soils which in turn overlie siltstone and sandstone bedrock. The attached Figure 3 shows a preliminary cross-section indicating the anticipated ground conditions.

From the results of the Telopea Stage 1A investigation and the 1 to 5 Shortland Street investigation which were located over the upper (western) portions of the site we expect weathered siltstone to be encountered at depths ranging from about 2m to 4m over sandstone bedrock at depths ranging from 8m to 10m.

Over the lower (southern and eastern) portions of the site the previous investigation (No. 7 Sturt Street) revealed a thin layer of siltstone over sandstone bedrock which we expect to be typical of these portions of the site. However, towards the Second Ponds Creek line, the surface of the bedrock may be deeper due to previous erosion of the weathered bedrock.

The above inferred subsurface profile may be used for planning purposes, but will need to be confirmed to allow detailed design. A detailed geotechnical investigation of each development area must be carried out to determine the actual subsurface conditions. The final scope of the geotechnical investigation should be determined once the final layout of the proposed buildings are known so the borehole locations can be targeted to suit the building layout.

Due to the expected size of the buildings all boreholes should involve the core drilling of the bedrock in order to optimise bearing pressures for the design of footings.

Information on groundwater levels should also be obtained and as part of the geotechnical investigation wells should be installed within boreholes and the groundwater levels monitored. Information on the groundwater levels will be particularly important as it is likely that the basements will extend to the groundwater table as well as encountering some seepage from the soil/bedrock interface.

4.2 Geotechnical Issues

Based on the above inferred subsurface profile the main geotechnical issues for the proposed development as outlined in Section 1 are presented below. Overall, we consider that the site is geotechnically suitable for the proposed developments and will be comparable to other similar developments constructed within nearby sites.

The comments and recommendations provided herein are preliminary and should only be used for planning and preliminary concept design purposes. The comments and recommendations will need to be confirmed by detailed geotechnical investigations of the various sites.

Dilapidation Surveys

Prior to the start of excavation, dilapidation surveys should be carried out on the adjoining properties which lie within a distance equal to twice the depth of excavation. Council may also require dilapidation surveys of their assets within the adjacent footpaths and roadways. The dilapidation surveys should comprise a detailed inspection of the adjoining properties and existing buildings, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The owners of the adjoining properties should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The preparation of the dilapidation report will help to guard against opportunistic claims for damage that was present prior to the start of excavation.

Excavation

Excavation to the proposed depths of up to 9m is expected to encounter clayey fill, residual silty clay and weathered siltstone and sandstone bedrock.

Excavation of the soils and upper rock of up to very low strength should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Some ripping of higher strength bands may be necessary if they are encountered within weaker rock.

Excavation of rock of low strength or higher strength, will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. Hydraulic rock hammers must be used with care due to the risk of damage to any adjoining properties from vibrations generated by such equipment. If hydraulic rock hammers are to be used the vibrations transmitted to the adjoining properties should be quantitatively monitored at all times during rock hammer operation. Preferably the monitors should be attached to the existing buildings, but if access for this is not possible then the monitors should be set up on the site boundaries. The monitors should be attached to flashing lights, or other suitable warning systems to advise the operator when acceptable limits have been reached so that excavation can cease. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are excessive it would be necessary to change to less vibration emitting equipment, such as smaller rock hammers, ripping hooks, rotary grinder or rock saws.

The excavated material will need to be classified for disposal purposes, which will require environmental testing of all materials, if this has not already been completed.

Groundwater

Only a few of the previously drilled boreholes encountered groundwater seepage during auger drilling and groundwater was measured within monitoring wells at between RL53.5m and RL51.8m. Due to the variable groundwater levels measured within the wells, we infer that the measured groundwater levels are likely to comprise seepage flowing through the rock rather than a standing groundwater level, at least in the shallower basements and in the more elevated parts of the site. However, information on groundwater levels should be obtained as part of the detailed geotechnical investigation and wells should be installed within boreholes and the groundwater levels monitored.

In the long term, drainage should be provided behind all retaining walls to collect and control any seepage that does occur. The completed excavation should be inspected by the hydraulic consultant to assess if the designed drainage system is adequate for the actual seepage flows.

Retention

Suitable retention systems will depend on the proposed basement depth and set-back distances from adjoining properties. For basements which extend up to or close to the site boundaries, full depth retention systems will need to be installed prior to the start of excavation.

Where space permits, temporary batters through the clayey soils and poor-quality siltstone and sandstone bedrock may be formed at no steeper than 1 Vertical (V): 1 Horizontal (H). Where adopted all surcharge loads such as stockpiles, traffic loads etc must be kept well clear of the crest of the batters. Where permanent batters are adopted, they should be formed at no steeper than 1Vertical (V): 2 Horizontal (H) and should be protected from erosion by vegetation, shotcrete and mesh or similar. For maintenance purposes it may be more practical to from permanent batters at no stepper than 1V:3H or 4H.

Where space does not allow for the formation of batters and excavation would extend into adjoining properties, a retention system will need to be installed prior to the commencement of excavation. Such a retention system may comprise soldier pile walls with shotcrete infill panels, or contiguous pile systems for areas which contain movement sensitive structures. From experience the construction of such shoring systems has become very cost effective and we do not expect that creation of temporary batters, stockpiling of materials for use as back fill, export of surplus materials, import of expensive drainage gravel and construction of “conventional” retaining walls to be an economical option.

Bored piers would be appropriate for the piled walls, but some groundwater seepage may be encountered requiring the use of pumps and tremie concreting techniques. The piers should be founded at least 1m below the base of the excavations, including excavations for footings and services, but more as required for stability design. Temporary lateral restraint of the retention systems would be required in the form of external anchors or internal props where excavation depths are 3m or more, with each restraining point progressively installed as it is exposed during excavation.

Detailed shoring wall design parameters can be provided following site-specific geotechnical investigations.

Footings

Following bulk excavations, we expect that weathered sandstone or siltstone will generally be encountered at bulk excavation level. Therefore, we recommend that the buildings are supported on the underlying siltstone and sandstone bedrock to provide uniform support and reduce the risk of differential movements.

We expect that pad/strip footings founded within the siltstone and sandstone would be appropriate. Where above ground portions of the buildings extend outside the basement excavation the use of piles may be required so that the footings are founded within bedrock below the zone of influence of the basement excavation.

The allowable bearing pressure for footings founded within siltstone and sandstone would commence at 700kPa for siltstone and sandstone of at least very low strength, but higher bearing pressures are expected to be possible if medium or high strength siltstone and sandstone is encountered, which will depend partly on the depth of excavation.

Basement Floor Slabs

The basement slabs are likely be cast on weathered siltstone and sandstone bedrock. Following completion of the bulk excavation, we recommend that the subgrade be inspected by a geotechnical engineer to assess the suitability of the subgrade to support the basement floor slabs. The design of the basement floor slabs should incorporate a subbase layer of DGB20, or other approved durable granular material compacted to at least 100% Standard Dry Density (SMDD). This will act as a separate/debonding layer from the weathered rock subgrade and will also reduce the risk of pumping of fines at slab joints. Sand layers should not be used below trafficable slabs.

Drainage may be required below the basement slab and the subbase layer may be used as a drainage layer if free draining and durable gravel is used. Alternatively, a grid of subsoil drains could be constructed below the slab. The drainage system should divert the collected water into sumps containing automatic pumps to remove the collected seepage to the stormwater system. The hydraulic consultant should inspect the completed excavation to confirm that the designed drainage system is adequate for the actual seepage flows.

Nearby Railway Line

As shown on the both the Annexure N-Staging Plan and survey plan by Craig & Rhodes (Ref:191-19, Amendment No.01, dated 20 February 2020) the railway line (Carlingford Line) is located on the north-western and western sides of the site. We understand the current works being carried out on the rail line are part of the upgrade to the Telopea Light Rail scheme.

Application will need to be given to the asset owners (Sydney Trains) for any development which is in proximity to the rail corridor. Sydney Trains may require finite element analysis of the possible movements affecting the rail infrastructure where parts of the development may be positioned within 25m of the rail corridor. Sydney Trains may also require monitoring to be carried out during construction, bur the extent of this will be dependant on the results of the modelling.

These issues can be investigated further during the detailed geotechnical investigation stages once details of basement setbacks and levels are known.

Other Services

At an early stage of planning we recommend that all major utilities are located as the presence of pipelines, tunnels etc are likely to have a significant impact on future developments.

5 GENERAL COMMENTS

The recommendations presented in this report are based on an inferred subsurface profile based on previous geotechnical investigations carried out on nearby sites. A site-specific geotechnical investigation will be required for each development. The comments and recommendations provided herein must be confirmed and amplified as part of the detailed geotechnical investigation.

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. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. 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

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

Title:	**SITE LOCATION PLAN**
Location:	TELOPEA, NSW
Report No: 33079SCrpt2	Figure: 1

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PLOT DATE: 20/05/2020 10:53:58 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33079SC TELOPEA\CAD\33079SCRPT2.DWG

JK GEOTECHNICS PREVIOUS INVESTIGATION, MAY 2020

JK GEOTECHNICS PREVIOUS INVESTIGATION, DECEMBER 2009

JK GEOTECHNICS PREVIOUS INVESTIGATION, DECEMBER 2009

0 200

Scale 1:200 @A3 Metres

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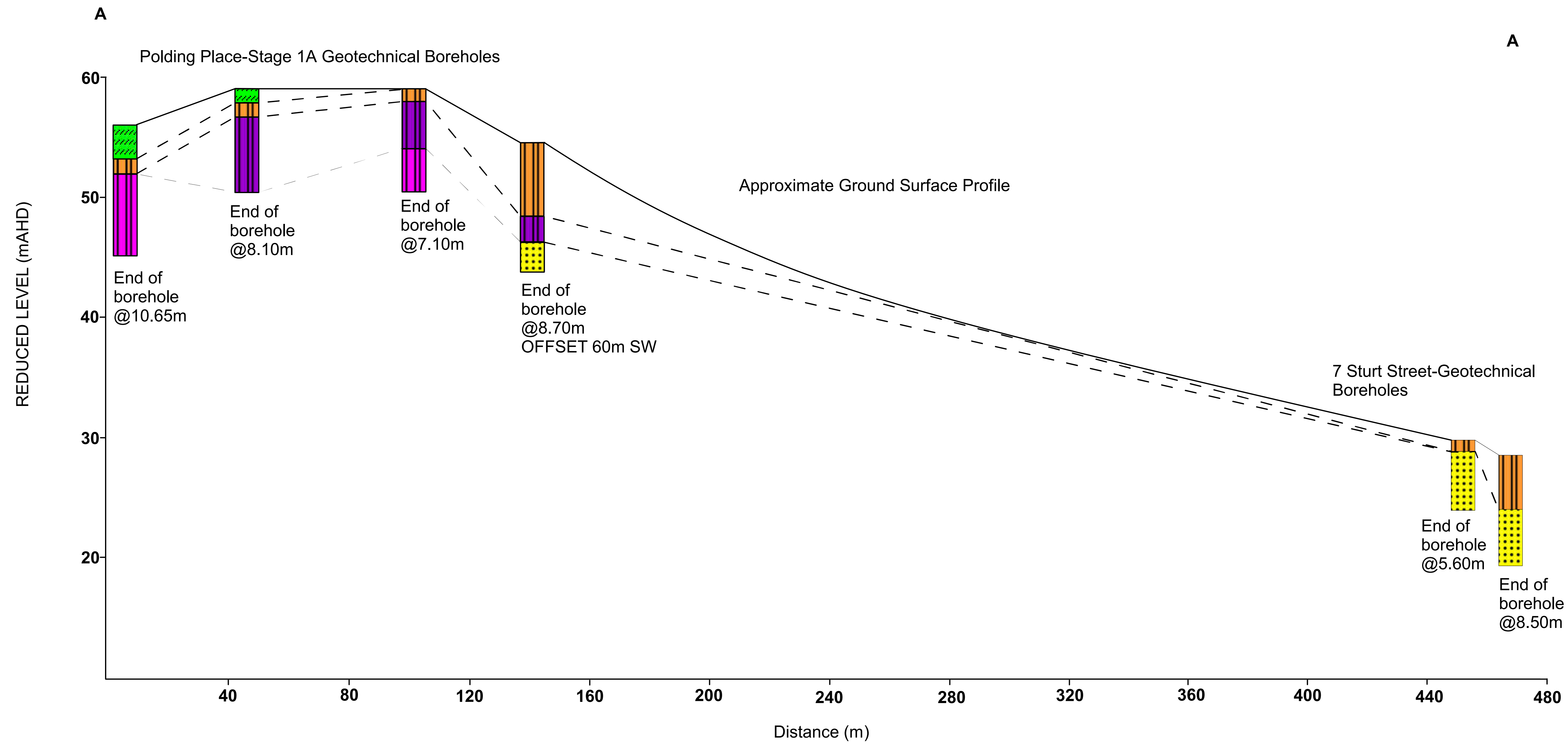
Report No. 33079SCrpt2 Figure No. 2



RESIDUAL SOIL

EXTREMELY WEATHERED SILTSTONE

SILTSTONE-LOW TO MEDIUM STRENGTH

SILTSTONE- MEDIUM STRENGTH OR BETTER

SANDSTONE- MEDIUM STRENGTH OR BETTER

GRAPHICAL SUMMARY & PRELIMINARY GEOTECHNICAL MODEL SECTION A-A LOOKING NORTHWARDS

0 20
Scale 1:400 @A3 Vertical Metres

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Report No. 33079SC Figure No. 3

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed 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:

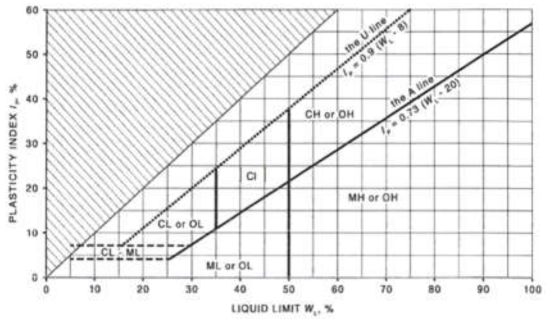
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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