REPORT TO

WINTEN PROPERTY GROUP

ON

GEOTECHNICAL DESKTOP ASSESSMENT

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

177-183 GREENWICH ROAD, GREENWICH, NSW

Date: 10 December 2024

Ref: 36742Lrptrev1

JKGeotechnics

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
36742Lrpt	Final Report	12 July 2024
36742Lrptrev1	Revision 1 – change in design	10 December 2024

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<u>**ATTACHMENTS**</u>

1 INTRODUCTION

This report presents the results of an updated geotechnical walkover and desktop assessment for the proposed redevelopment at 177 to 183 Greenwich Road, Greenwich, NSW. The location of the site is shown in Figure 1. The initial assessment was commissioned by Winten Property Group and it was carried out in general accordance with our fee proposal, Ref: P60470L, dated: 17 April 2024.

We understand that the proposed development is in the early stages of approval and a planning proposal is currently being prepared. From the revised concept architectural drawings prepared by PDB Architects (dated: 4 November 2024), the proposed development will now include an eight-level residential structure, comprising eight residential levels and a communal rooftop. The residential floors are distributed across four levels above the current ground level, while an additional four levels are built into the slope at the western end of the structure. Three basement car parking levels are included at the eastern end of the structure. Excavation for the basement carpark will be in the order of 10m at the eastern (Greenwich Road) end of the site.

The purpose of the assessment was to obtain geotechnical information on the likely subsurface conditions, and to use this as a basis for providing preliminary comments and recommendations on excavation conditions, shoring, groundwater, slope stability, acid sulfate soils and footings.

2 ASSESSMENT PROCEDURE

The geotechnical desktop assessment comprised the following:

- Review of our extensive database of previous projects.
- Review of the Sydney 1:250,000 Geological Series Sheet S1 56-5 and the Sydney 1:100,000 Geological Series Sheet 9130.
- Review of acid sulfate soils map.
- A review of aerial photography and digital street view (Google Earth).
- A brief site walkover inspection on 14 June 2024 by our Senior Engineering Geologist, Mr Charles Watkinson.

3 RESULTS OF ASSESSMENT

3.1 Site Description

The site is located within hilly topography that can be observed sloping down to the Lane Cove River to the west. The site itself is located near the crest of a ridge line, the topography across the site appears to step down via a series of vertical and subvertical rock outcrops, the site also gently slopes to the north. Based on the detailed survey prepared by Norton Survey Partners (Ref: 22195, Dated: 08.11.23), the surface levels within the site range from about 34mAHD within the south east corner of the site, 30mAHD within the north east corner and 0.70mAHD in the western portion of the site.

The site itself contains four residential properties. In general the front or eastern portions of the properties are gently sloping while the rear western portions are steeply sloping and heavily vegetated. A general description of the front portion of each of the properties is discussed below. It should be noted that at the time of the walkover, access to No. 177 and No. 179 was not possible due to the presence of existing tenants.

- **No. 177 Greenwich Road** – This is the most northern property within the site, and it contains a one and two storey brick and clad house.
- **No. 179 Greenwich Road** – This site contains a two-storey brick rendered house. Within the rear of the property there is a suspended swimming pool which is elevated approximately 3.5m above ground level on the lower west side, the pool coping level is approximately 2m lower than the adjacent ground level within No. 181.
- **No. 181 Greenwich Road** – This site contains a two-storey brick rendered house, no visible signs of distress were observed based on our external inspection. The front of the property is delineated by a sandstone block retaining wall with a retention height of approximately 0.7m. The boundary between No. 181 and No. 183 contains a sandstone block retaining wall which is approximately 0.6m in height. The sandstone block wall appears to be in good condition. The front of the property gently slopes down to the west at less than 1°.
- **No. 183 Greenwich Road** – This is the most southern property within the site and it contains a two-storey brick rendered house with a sandstone block subfloor level, no visible signs of distress were observed based on our external inspection. This property has a sandstone block retaining wall along the eastern (or Greenwich Road) boundary that has an approximate retention height of 2m and is showing significant bulging within the upper section of the wall, see Plates 1 and 2 below. The front of the property gently slopes west at less than 1°, whereas the rear garden of the property slopes west at approximately 2° to 3°. The boundary between the rear garden and bushland to the west is separated by a set of concrete steps and a sandstone block retaining wall which has a retention height of approximately 1m. The steps appear to be in poor condition with significant cracking up to 30mm. No visible signs of distress were noted in the retaining wall. There is a sandstone outcrop at the base of the retaining wall and the exposed sandstone was assessed to be moderately weathered and low to medium strength.

 

Plates 1 and 2 - View looking south showing bulge within No. 183 Greenwich Road front boundary retaining wall.

The rear portions of the properties generally comprise heavily vegetated bushland that slopes down to the west at average gradients ranging from about 14^{o} to 20^{o}. Some more specific site observations from within this rear portion of the site are described below. We note that there are no features that delineate property boundaries in this rear portion of the site, therefore this area has been described as a whole.

- **Steep slopes** – The rear of the subject site (western portion) comprises a heavily vegetated area, with surface levels generally sloping down to the west at gradients ranging from 14° to 20°. The topography appears to follow the natural hillslope with sandstone outcrops identified at the crest and base of the slope. During our site walkover, we did not observe any signs of recent slope instability (such as rock falls or soil slumps). However, we note that the rear of the site was quite heavily vegetated and it is possible that some slope features may have been hidden.

- **Sandstone outcrops** – Sandstone bedrock outcrops were identified near the crest and the base of the slope, with approximate surface levels of 18 mAHD and 1 mAHD respectively. These outcrops consist of moderately weathered, low to medium strength sandstone, with the bedding dipping towards the east at an angle of approximately 10° to 15°. During walkover, it was noted that at the upper-level outcrop, there is minor groundwater seepage occurring along defects and at the interface between soil and rock.

- **Cliff face overhangs/undercuts** – A low to medium strength sandstone cliff face was identified near the southern boundary of the site, immediately beyond the back garden of No 183. A photograph of the cliff face is shown in Plate 3, and its approximate location is indicated on Figure 2. The cliff face features an overhang that measures approximately 3.5 meters in height and 5 meters in width, the bedding was observed to be oriented perpendicular to the face and dipping east at 10° to 20°. Additionally, a semi-mature tree was observed growing on the cliff face.

Plate 3 - Cliff face overhang/undercut – looking east

- **Retaining walls** – Towards the top of the slope near the rear gardens of No. 183 and No. 181 there are three sandstone block and brick retaining walls which vary in height from 2m to 3m. Overall, the retaining walls appear to be in fair to poor condition with cracking and bulging observed.

The subject site is bounded by:

- No. 175 Greenwich Road, which is to the north of the subject site, contains a multi-level brick apartment building, the apartment building is offset approximately 5m from the common boundary. Between the apartment building and the boundary with No. 177 there is a set of concrete stairs, footpath and small garden bed. Site levels near the boundary are the same as those on the subject site.
- Greenwich Road, is to the east, and it comprises a footpath with grassed areas and mature trees, with a single lane road which appeared to be in fair condition with some minor cracking observed. The road gently slopes down to the north west at approximately 1° to 2°.
- No. 4 Landenburg Place, is to the south of the subject site, and it contains a five-storey brick apartment building. The apartment building is offset approximately 3m from the boundary with the subject site. The boundary is delineated by a sandstone block retaining wall that has a retention height of approximately 2.1m. The wall appears to be in poor condition as the lower section has moved eastward by approximately 60mm to 70mm as shown in Plate 4 below. The retaining wall was

observed, in parts, to be directly founded on sandstone bedrock, the bedrock was assessed to be moderately weathered and of low to medium strength.

- No. 3 Landenburg Place, lies to the southwest of the subject site, and it contains a four-storey brick apartment building. The apartment building is offset approximately 9m from the boundary with the subject site. Although due to site access we were unable to inspect the condition of the wall, we assume it is in poor condition, based on the conditions observed at No. 4 Landenburg Place.
- Lane Cove River, lies to the west.

Plate 4 - Photograph looking East of failed retaining wall between No. 183 Greenwich Road and No. 4 Landenburg Place.

3.2 Subsurface Conditions

Reference to the Sydney 1:250,000 Geological Series Sheet S1 56-5 and the Sydney 1:100,000 Geological Series Sheet 9130, indicates that the site is mapped to be underlain by Hawkesbury Sandstone comprising "medium to coarse-grained quartz sandstone, very minor shale and laminate lenses". The geological maps do not take into consideration past development at the site. Localised topsoil and fill are likely to be present at the site and are associated with the current residential development.

Based on our available information, we expect to encounter fill/topsoil directly overlying sandstone bedrock at shallow depths, although a thin layer of residual or slopewash soils overlying the bedrock may be present in some areas. Based on the exposed bedrock, the underlying sandstone is expected to be of low to medium strength on first contact.

Groundwater seepage through existing defects was observed in the existing cliff face within the western portion of the site. Groundwater seepage is expected along the soil/rock interface and through defects in the bedrock, particularly during rainfall periods. Groundwater is expected to be encountered during the proposed development.

3.3 Acid Sulfate Soils

From reference to the Atlas of Australian Acid Sulphate Soils (AAASS) map and the Acid Sulfate Soils Risk map (2nd edition, 1998) the site itself is not within an area that is subject to acid sulfate soils or acid sulfate risk.

The Lane Cove River, which is located immediately beyond the site's western boundary, is mapped to have a high probability of acid sulfate soils in the alluvial sediments below the water level.

Therefore, we do not expect that acid sulfate soils will be problematic for the current proposed development.

4 COMMENTS AND RECOMMENDATIONS

Based on our initial walkover assessment of the site, we consider that geotechnically the site is suitable for the proposed development. The comments and recommendations provided below are based on an assumed subsurface profile as detailed above. The subsurface profile must be confirmed at the very least by subsurface investigations (including but not limited to cored boreholes) during the detailed design phases of the project and prior to construction. Adjustments may need to be made to the recommendations below once the subsurface conditions have been confirmed.

We consider that the primary geotechnical issues for the proposed development will be;

1. Excavation for the lower Basement 2 will be to substantial depths in the order of 10m to along the Greenwich Road (or eastern) side of the site and in the order of about 10m along the seaward (or western) side of the site for the lower Basement 3. Excavation is expected to encounter sandstone bedrock at relatively shallow depth and as such the majority of the excavation will likely be through sandstone bedrock. Careful control of excavation stability and excavation vibrations will be required during the excavation process.
2. The lower portion of the site is moderately sloping. It is currently proposed to excavate into the slope to construct the building. Detailed investigation within this area will be required to confirm that footings are uniformly founded on sandstone bedrock below and sandstone floaters.
3. There are a number of retaining walls within the site and along site boundaries that are in poor condition. To reduce the risk of retaining wall failure these walls will need to be replaced or stabilised as part of the early works.

4.1 Dilapidation Reports

Prior to commencement of any site works, including demolition of the existing buildings, we recommend that a detailed internal and external dilapidation report be completed on the adjoining structures to the north and south. Dilapidation reports should also be carried out on all boundary retaining walls and other retaining walls within adjoining sites. Dilapidation reports provide a record of existing conditions prior to commencement of any site works. The dilapidation report would therefore be used as a benchmark against which to set vibration limits during excavation, and for assessing possible future claims for damage arising from the works.

As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly by reputable companies with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc). The dilapidation report should be reviewed by JK Geotechnics and the structural engineers prior to commencement of the works.

We forewarn that Council may also require a dilapidation survey on the abutting roadways and footpaths, and Sydney Water may require a detailed assessment of the impact of site works on any of their assets located in the footpaths or roadways. Our recent experience is that if Sydney Water requires an assessment of its assets, then this can take substantial time to obtain approval. Therefore, we recommend the requirement for this assessment be investigated early in the development stages.

4.2 Excavation Conditions

Prior to any excavation commencing we recommend that reference be made to the NSW Government "Code of Practice Excavation Work" dated January 2020 or the most recent version at the time of works commencing.

Excavation to achieve the lowest level (basement 3) is expected to extend through some surface fill, residual and/or slopewash soils and then predominantly through sandstone bedrock. The soil materials should be readily excavated using the buckets of conventional earthmoving equipment, such as hydraulic excavators. Excavation of the bedrock is expected to require rock excavation techniques. Rock excavation techniques include rock hammers, rock saws (possibly in combination with some ripping with a ripping tyne fitted to a large excavator) or rock grinders.

Hydraulic impact hammers would only be feasible, if continuous vibration monitoring on adjoining structures to the north and south is adopted, and the vibrations from the vibration monitors is shown to less than tolerable limits for the adjoining structures. Particular care will be required when excavations are close to site boundaries (in particular near existing retaining walls). In such areas we do not recommend hydraulic impact hammers be adopted unless the retaining walls have been replaced by properly engineered retaining walls, adequately stabilised or otherwise supported to maintain their stability.

Groundwater seepage into the excavation is expected at the soil/rock interface and through joints/defects within the bedrock. Due to the location of the site in the regional topography we consider that any groundwater seepage would be controllable by conventional sump and pump techniques.

4.2.1 Excavation Vibrations

Caution must be taken during all demolition, excavation, shoring and footing construction on this site as there will likely be direct transmission of ground vibrations to the neighbouring structures, retaining walls and any nearby buried services. Excavation procedures and the dilapidation reports should be carefully reviewed by the geotechnical and structural engineers prior to the commencement of demolition and excavation, so that appropriate equipment is used.

Where excavation of any rock using hydraulic impact hammers is being considered, then it should commence away from likely critical areas (i.e. commence away from the northern and southern site boundaries) employing a moderately sized excavator fitted with a relatively low energy hydraulic impact rock hammer. To reduce the transmission of vibrations, consideration may also need to be given to vertical saw cutting around the perimeter of the excavation, with the base of the saw cut slot maintained at a lower level than the adjoining rock excavation at all times.

We recommend that continuous quantitative vibration monitoring be carried out during all site works (including demolition), but particularly where rock excavation using hydraulic impact hammers is to be used. Vibration monitors should be set up on adjoining structures to the north and south and they should be fitted with flashing warning lights and sirens which would warn if vibrations exceed the pre-set limits.

Subject to review of the dilapidation reports, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec on the neighbouring structures. This limit takes both human comfort and potential structural damage into account and assumes that the structural engineers inspect the adjoining structures and confirm that these adjoining structure are not particularly sensitive to vibrations.

If during any site works (including demolition and excavation) it is found that transmitted vibrations are excessive, then it would be necessary to use a smaller rock hammer or alternative excavation techniques. The use of a rotary grinder or grid sawing in conjunction with ripping and hammering present alternative lower vibration excavation techniques. We expect that at least some of the rock excavation will require the use of these lower vibration emitting techniques, particularly where excavation is required close to the northern and southern site boundaries. Therefore, allowance in both budget and time should be made for such equipment.

4.3 Excavation Conditions and Support

Based on the supplied concept architectural drawings prepared by PDB Architects, we expect excavations in the order of 10m deep in the eastern end of the site for the proposed basement car park and 10m at the western end of the site for the proposed lower residential levels. The excavation will extend through the overlaying soils and sandstone bedrock.

Where soils are encountered and space allows, we recommend temporary batters are formed through the soils at no steeper than 1 Vertical (V) to 1.5 Horizontal (H), provided the soils are not greater than 3m deep. Given the expected limited depth of the soil, we expect temporary batters within the soils and vertical excavations within the sandstone should be feasible for the majority of the excavation. Again care will be required where temporary batters extend close to boundary retaining walls, and specific advice will be required if temporary baters are being considered.

Where the sandstone is of at least medium strength and has few adverse defects, then the sandstone will be able to be excavated vertically. However, this must be confirmed by further site specific subsurface investigations and then inspection of the sandstone bedrock at not greater than 1.5m depth intervals to check for any weak seams or inclined defects that require additional support. Any additional support recommended by the geotechnical engineer, such as rock bolts, shotcrete and mesh or dental treatment of thin seams, must be installed prior to further construction.

For rock excavation within the south western corner of the site where overhangs/undercuts are present, we do not recommend the use of percussive excavation equipment due to the risk of damage to the cliff that could affect the overhangs. Any rock excavation should be carried out using non-percussive excavation equipment, such as rock saws and grinders. In addition, excavation and other construction equipment should not traffic over the undercut portions of the cliff, with an exclusion zone enforced located at least 1m from the rear of any undercut sections.

All excavations exposing soil materials will require permanent support. At this stage assuming that the underlying bedrock is of at least medium strength with no adverse defects shoring walls can be founded on the top of the bedrock. Lateral support in the form of propping or anchors will be required. The following characteristic soil parameters are recommended for preliminary design of the shoring walls. In all cases specific shoring wall analysis must be undertaken, including an assessment of the likely ground movements beyond the shoring walls. The shoring wall design engineers should then be requested to provide comment on whether such movements will be problematic to any adjoining structures or services.

- Adjoining structures/retaining walls and roadways are likely to be movement sensitive, and they will also be within a horizontal distance of 'H' from shoring walls (where 'H' is the depth of soil in metres). Therefore, for preliminary design, we recommend propped or anchored shoring walls be designed for an apparent rectangular lateral earth pressure distribution of 8H kPa, to reduce deflections.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads, traffic, etc) are additional to the earth pressure recommendations above and should be included in the design.
- The retaining walls should be designed for hydrostatic pressures. This is also additional to the earth pressure recommendations above. However further groundwater monitoring should be carried out to assess potential groundwater fluctuations further for inclusion in detailed design and analysis.
- Anchors (if adopted for lateral support) should be bonded a minimum of 3m into sandstone bedrock of at least medium strength with an allowable bond stress of 300kPa. The anchor bond length should commence within rock and beyond a line projected up at an angle of 45° from the base of the excavation.
- All anchors should be proof-tested to 1.3 times their working load under the direction of an experienced engineer or construction superintendent, independent of the anchor contractor. Anchors should be locked off at about 80% of their design working load.

4.4 Groundwater

Groundwater seepage through existing defects was observed in the existing cliff face within the western portion of the site. During excavation seepage along the soil/rock interface and through defects in the bedrock is expected into the excavation, particularly during and following rainfall periods. The hillside generally slopes down to the west and any seepage flow would be expected to be in a similar direction.

Groundwater seepage encountered during construction will be able to be controlled by conventional sump and pump techniques or by conventional gravity drainage. Over the long term the proposed provision of effective drainage of all sub-structures will, in our opinion, allow 'through-flow' of groundwater with no build-up of uphill groundwater levels to the extent that neighbouring properties will be adversely affected. Furthermore, given the presence of the existing cliff face, we do not expect the proposed additions to have any impact on the current groundwater regime. If groundwater seepage collected from the basement excavation is to be redirected downslope then consideration will need to be given to disposal of the seepage in a controlled manner such that it does not cause instability or erosion downslope.

In view of the above, the proposed development should have negligible effects on the groundwater regime above and below the site and the neighbouring buildings and structures. However, we recommend that the proposed excavations be monitored to confirm groundwater conditions. We also note that effective control of surface run-off will be required both during and after construction.

4.5 Footings

Given the expected relatively shallow bedrock depth, and that following bulk excavation sandstone bedrock will almost certainly be exposed over the majority of the bulk excavation level, we recommend all new structures be supported on footings uniformly founded within the underlying sandstone bedrock. Where sandstone is exposed or is at shallow depths pad or strip footings may be used. However, where bedrock is at a depth of greater than about 1.5m, then piles will be necessary. Pile footings may be required at the western end of the site within the more steeply sloping portion of the site. As discussed above further investigations, including cored boreholes will be required to obtain a reasonable assessment of bedrock levels, to reduce the risk of piled footings being founded on sandstone floaters. We expect that bored piles will probably be feasible, although some seepage at the soil rock interface and through joints and bedding may occur and this may lead to the necessity to pour concrete via tremie methods or to pump water from the base of piles prior to pouring concrete.

Footings founded on the sandstone bedrock may be designed based upon a preliminary Allowable Bearing Pressure (ABP) of 1000kPa. Higher bearing pressures are likely to be appropriate within the sandstone, but boreholes would need to be drilled to assess the rock quality.

Allowance should be made for all pad/strip footings and piled footings to be inspected by a geotechnical engineer prior to pouring concrete to confirm that a suitable bearing pressure is being achieved.

4.6 Slope Stability

Based on our site observations, Instability of boundary retaining walls and instability within the lower western portion of the site generally present the highest risks of instability at this site. Geotechnical risks during construction (such as instability of rock cuts) can usually be well managed by incorporating appropriately timed geotechnical inspections.

Instability in the lower portion of the site may occur as a result of dislodgment of sandstone floaters or boulders, instability of cliff lines and/or local soil slumping.

To address the risk from geotechnical hazards on this site, a detailed stability and risk assessment of the site will need to be carried out. The stability assessment would be designed to identify the geotechnical hazards and to provide recommendations for stabilisation.

We consider that the risks from geotechnical hazards can be kept within acceptable levels provided there is appropriate and detailed site inspection and assessment, combined with engineer designed stabilisation and site management/controls.

4.7 Further Geotechnical Input

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

- Detailed site-specific subsurface investigations (including but not limited to cored boreholes) to depths greater than the maximum bulk excavation level.
- Investigation and assessment of existing boundary retaining walls which are showing signs of instability
- Detailed rock mapping / slope stability assessment of cliff faces and overhangs/undercuts.
- Quantitative vibration monitoring once rock excavation commences.
- Inspection of rock cut faces.
- Inspection of footing excavations.

5 GENERAL COMMENTS

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

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

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

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

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	177-183 GREENWICH ROAD, GREENWICH, NSW
Report No: 36742L	Figure No: 1

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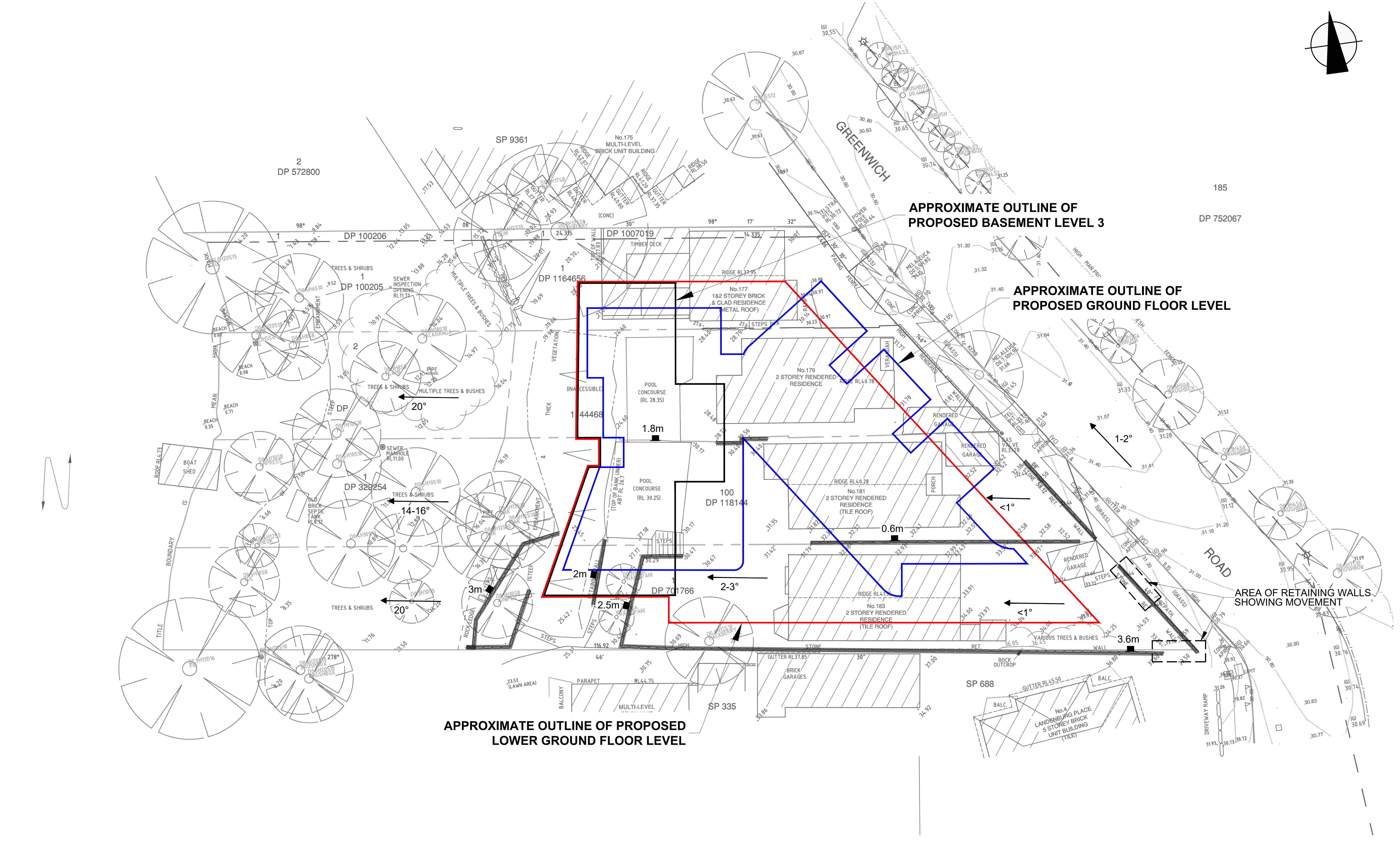
PLOT DATE: 26/06/2024 2:06:22 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36742L GREENWICH\CAD\36742L.DWG



NOTE:
REFER TO FIGURE 3 FOR GEOTECHNICAL MAPPING SYMBOLS.

Title:	GEOTECHNICAL SITE PLAN
Location:	177-183 GREENWICH ROAD, GREENWICH, NSW
Report No:	36742L
Figure No:	2

SCALE 1:400 @A3 METRES (0 4 8 12 16 20)

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

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PLOT DATE: 10/12/2024 10:30:27 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36742L GREENWICH\CAD\36742L.DWG

TOPOGRAPHY

Symbol	*Ground Profile*		
		convex concave	well defined or angular break of slope
		convex concave	poorly defined or smooth change of slope

breaks of slope
changes of slope
— convex and concave too close together to allow the use of separate symbols

sharp
rounded
— ridge crest

3 Cliff or escarpment or sharp break 40° or more (estimated height in metres)

15 Uniform Slope
10 Concave Slope
8 Convex Slope
— Slope direction and angle (Degrees)

Top
Bottom
— Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

OTHER FEATURES

Boulder

Seepage/spring

Swallow hole for runoff

Natural water course

Open drain, unlined

Open drain, lined

Fenceline

Property boundary

Dry Stone Wall

J —— J 200 Major joint in rock face (opening in millimetres)

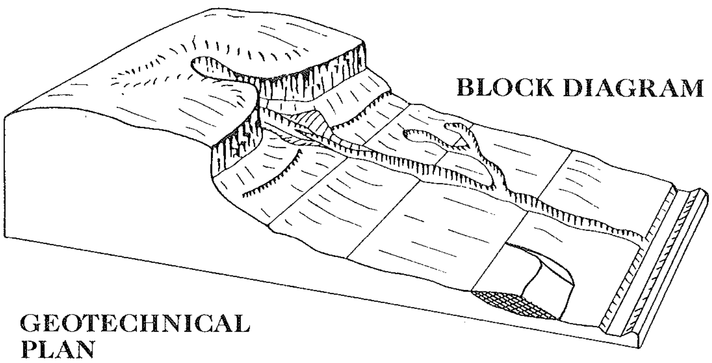
– T – T – 10 Tension crack (opening in millimetres)

Masonry or concrete wall

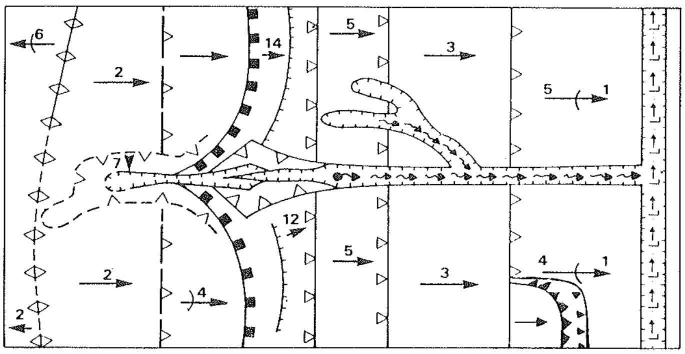
Ponding water

Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



GEOTECHNICAL PLAN



(After Gardiner, V & Dackombe, R.V. (1983), Geomorphological Field Manual; George Allen & Unwin).

GEOTECHNICAL MAPPING SYMBOLS

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. Figure No.

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:

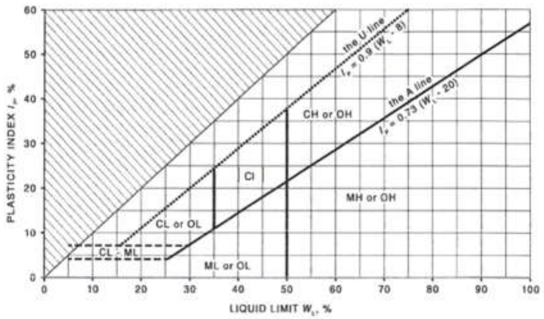
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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.

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

Abbreviations Used in Defect Description

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