# REPORT

TO
WELKIN CONSTRUCTION GROUP PTY LTD

ON
GEOTECHNICAL SLOPE STABILITY ASSESSMENT

FOR
PROPOSED ALTERATIONS AND ADDITIONS

AT
65 SEAFORTH CRESCENT, SEAFORTH, NSW

4 February 2019
Ref: 32084SNrpt

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Date: 4 February 2019
Report No: 32084SNrpt
Revision No: 0

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## 1 INTRODUCTION

This report presents the results of a geotechnical slope stability assessment for the proposed alterations and additions at 65 Seaforth Crescent, Seaforth, NSW. A site location plan is presented as Figure 1. The assessment was commissioned by returned Acceptance of Proposal form, signed by Ms Rebecca Zhou of Welkin Construction group Pty Ltd. The commission was on the basis of our proposal Ref: P48554PN dated 29 November 2018.

From the supplied Armada Pre-DA architectural drawings (Dwg Nos A-100, 101, 102, 107 to 122 & 125 all Revision 1 dated 26 November 2018), we understand the proposed alterations and additions will comprise the following:

- A new garage and entry pavilion above the existing building at the northern end, along with a new driveway from street level to the new garage.
- Excavation beneath the existing house to accommodate a new lift and access passage.
- New balconies off the southern end of the existing building at ground floor level, level 1, level 2 and level 3.
- A new pool at level 4 with entertainment room beyond and below the southern end of the existing building.
- Alterations to the existing lower terrace.
- A boat shed, jetty and mooring at the southern end of the property.
- An inclinator from the lower terrace down to the boat shed.
- Internal alterations to the existing building.

Excavation to a maximum depth of about 4m below existing levels, and stepping down the hillside, is expected to be required for the proposed works.

As no structural loads have been supplied, typical loads have been assumed.

The purpose of the assessment was to complete a walkover inspection of the site and its surrounds as a basis for completing a landslide risk assessment and providing comments and recommendations on slope stability excavation conditions, retention design, footings, and on-grade floor slabs.

## 2 ASSESSMENT PROCEDURE

This stability assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 5 following our geotechnical assessment.

The attached Figure 2 presents a geotechnical site plan showing our mapping of relevant geotechnical and surface features both on the site and in its immediate surrounds. Figure 2 is based on the supplied Hill & Blume survey drawing (Dwg no. 60909001A dated 9 11 2018) and available Nearmap imagery. In this regard, we note that a number of areas on site were heavily overgrown, and the area downslope of the existing terrace (i.e. the southernmost approximately 15m of the site) was not accessible at the time of our walkover inspection. Figure 3 presents our geotechnical mapping symbols. Features on Figure 2 have been measured by tape measure and hand held clinometer techniques where accessible, and estimated otherwise, and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques.

## 3 RESULTS OF INVESTIGATION

### 3.1 Site Description

We recommend that the summary of observations which follow, be complemented by reference to the attached Figures 2 and 3.

The site is located on a south facing hillside, which grades steeply to Middle Harbour, at about $30^0$ overall. Middle Harbour forms the southern site boundary and Seaforth Road the northern site boundary. The site is roughly rectangular in plan, being approximately 61m deep (north to south) and approximately 12m wide (east to west). The elevation relief between the northern and southern property boundaries is approximately 40m.

At the time of the inspection, a three storey concrete framed residence occupied the northern portion of the site. A paved driveway, which sloped down to the south at about $15^0$ provided access from Seaforth Road to a garage at the northern end of the house. To the north of the house, adjacent to the driveway, was a landscaped area with several masonry retaining walls to a maximum height of about 2.5m. Along the western side of the house, concrete stairs provided access to the southern half of the property. Below the lowest level of the house, was a 2.5m high sandstone masonry retaining wall.

To the south of the house, a series of concrete stairs meandered down to a lower, near level, paved and landscaped terrace. Supporting the stairs were a number of concrete masonry retaining walls. Between the meandering stairs were heavily vegetated garden beds which appeared to slope down to the south at about $35^0$. At the north-west corner of the lower terrace was a sandstone masonry wall, with a maximum height of about 2.5m, which supported the garden bed areas. In the north-east corner of the lower terrace was a sandstone bedrock outcrop which was assessed to be of medium strength based on striking with a geological hammer. The southern side of the terrace was supported by a 2.3m high concrete retaining wall, below which was a 2m wide garden bed, which was in turn supported by a 3.0m high concrete retaining wall. Beyond the lower retaining wall, the site graded down to the southern site boundary at about $35^0$. The area to the south of the lower retaining wall was not accessible and due to moderate vegetation cover, it was not possible to observe the southern boundary.

The existing house and the majority of the retaining walls appeared to be in good condition based on a cursory inspection, however, the sandstone masonry retaining wall at the north-west corner of the lower terrace was in fair to poor condition.

To the east of the site was a 3 storey rendered house, over the northern half of the site, set back about 1m from the site boundary. The southern half of the site to the east was heavily vegetated and sloped down to the south to Middle Harbour. Whilst not visible from within the subject site, review of available Nearmap imagery and the provided survey drawing indicates the presence of a jetty at the southern end of the site to the east, along with several sandstone outcrops or boulders.

To the west of the site was a 4 storey concrete framed house, over the northern half of the site, set back about 1m from the site boundary. Along the eastern side of the neighbouring house was a sandstone bedrock outcrop, which included an approximately 2m deep and 2m high overhang. The sandstone bedrock was assessed as being of medium strength based on striking with a geological hammer. Immediately to the south of the neighbouring house was a large sandstone bedrock outcrop, the surface of which graded to the south at about $60^0$. To the south of the bedrock outcrop, the neighbouring site was heavily vegetated and sloped down to the south to Middle Harbour.

### 3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site is underlain by Hawkesbury Sandstone.

The walkover inspection disclosed several sandstone bedrock outcrops both on the subject site and in the neighbouring site to west. The exposed sandstone was generally assessed to be of medium strength. From our experience in the vicinity of the site and our observations, we infer that where not exposed, sandstone bedrock will be at shallow depth below a sill and granular soil profile.

In addition, review of available Dial Before You Dig Drawings indicates a 450mm diameter reinforced concrete sewer main running in an east west direction blow the centre of the site. From discussions with Mr Jarrod Lamshed of Armarda, we understand this sewer is at a depth of about 11m below existing levels.

## 4 GEOTECHNICAL ASSESSMENT

### 4.1 Potential Landslide Hazards

We consider that the potential landslide hazards associated with the proposed alterations and additions to be the following:

A Temporary and permanent stability of sandstone bedrock cut faces

B Stability of new retaining walls.

C Stability of the natural hillside slope uphill of the site.

D Stability of the natural hillside on the site.

## 4.2 Risk Assessment

The Table below summarises our qualitative assessment of each potential landslide hazard, and of the consequences to property, should the landslide hazard occur. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A.

| POTENTIAL LANDSLIDE HAZARD | A: Stability of Bedrock Cut Faces * | B: Stability of Existing and New Retaining Walls ** | C: Stability of the Natural Hillside Uphill of the Site *** | D: Stability of the Natural Hillside on the Site |
|---|---|---|---|---|
| Assessed Likelihood | Unlikely (Temporary) or Rare (Permanent) | Rare | Unlikely | Rare |
| Assessed Consequences | Minor (Temporary) or Medium (Permanent) | Minor | Medium | Medium |
| Risk | Low | Very Low | Very Low | Low |

Notes:

* Assumes inspections of all bedrock cut faces, and adoption of all stabilisation measures recommended as detailed in Section 5 below.

** Assumes that the existing retaining walls to be maintained and any new walls are engineer designed to appropriate standards, and have been constructed in accordance with such designs.

*** Assumes adequate maintenance of the upslope area, in particular the Council road easement.

The above assessment assumes that the comments and recommendations in Section 5 below are adopted in full, and that the existing house is uniformly supported on footings founded on Hawkesbury Sandstone of at least medium strength.

Our assessed risk to property is Very Low or Low, which would be considered ‘acceptable’ in accordance with the criteria given in Reference 1.

Assuming typical occupancy levels, temporal, and spatial considerations, we also consider the risk to life for the person most at risk would also be 'acceptable' in accordance with the criteria given in Reference 1.

## 5 COMMENTS AND RECOMMENDATIONS

### 5.1 Caution

The comments and recommendations presented in the following sections of the report are based on the inferred subsurface conditions from our walkover inspection, which must be confirmed by a series of inspections during construction. Should unexpected subsurface conditions be encountered at any time, additional geotechnical advice must be sought without delay.

At this stage, it is unclear how the excavation for the proposed extension to the southern side of the existing house is to be completed. Mobilisation of a moderate sized excavator would require either a large mobile crane, or a barge along with extensive alterations to the hillside over the southern half of the site.

Further, demolition of the existing retaining structures and excavation to the south of the existing house must be completed with extreme caution as there is the potential for the works to undermine or destabilise some existing footings.

Prior to any demolition or excavation commencing, we strongly recommend a meeting be attended by the builder, excavation contractor, ourselves and the project structural engineer to discuss the excavation methodology as well as all inspections required for the proposed works.

### 5.2 Dilapidation Surveys

Prior to any demolition or excavation commencing, we recommend that detailed dilapidation reports be prepared for the adjoining buildings to the east and west of the site. The dilapidation surveys should comprise a detailed inspection both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective building owners should be provided with copies of the dilapidation reports and be asked to confirm that they present a fair representation of existing conditions.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works.

### 5.3 Sydney Water Sewer

The existing 450mm diameter Sydney Water Sewer is located at significant depth below existing levels, and based on the proposed works, will be no shallower than about 7m depth following the completion of excavation. Given the natural slope of the hillside, and relatively shallow nature of the proposed excavations, we consider the proposed works will not adversely impact the existing sewer. We note that Sydney Water must be consulted regarding any specific considerations with regards to the project structural design and construction.

## 5.4 Demolition and Excavation

The existing retaining walls to the south of the existing house may be providing support to some of the footings of the existing house. Initial demolition and excavation must be undertaken with caution such that all footings are progressively located, and their foundation material confirmed, prior to demolition or excavation progressing below the base of each footing. After the top of each footing is exposed, subsequent excavation to the base of the footing, to confirm the foundation material, must be completed in the presence of an experienced geotechnical engineer. Based on the inferred subsurface conditions, we anticipate that all of the existing house footings are founded on competent sandstone bedrock. Should the exposed footings be confirmed to be founded on sandstone bedrock of at least medium strength, it is likely that no particular additional support will be required unless adversely orientated defects are present; however, if footings are founded on a soil profile or weak bedrock, additional support would most likely be required, and would be detailed at the time of the inspection.

For the proposed lift and access passage beneath the existing house, the same procedure to that above would also apply as excavation progresses.

To achieve design surface levels for the proposed works, excavation through the expected limited soil thickness, and into the underlying sandstone bedrock profile is expected to be required.

Excavation of the soil profile, as well as any extremely weathered bedrock is expected to be readily achievable using conventional techniques such as the buckets of hydraulic excavators or hand tools where required.

Excavation through very low and greater strength bedrock will be expected to be slower, and we recommend ripping/hammering using an excavator mounted rock hammer, however, excavation using hand held jack hammer could also be considered where required. Rock sawing in conjunction with ripping and/or hammering will facilitate excavation of medium and greater strength bedrock.

We recommend that caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to adjoining buildings and services.

Excavation using hydraulic rock hammers should commence away from likely critical areas (i.e. towards the southern end of the site). We recommend that continuous vibration monitoring be carried out on the structures to the east and west during all demolition and excavation works if excavator mounted hammers are utilised. Monitoring of the existing structure on suite should also be considered. Vibrations, measured as Peak Particle Velocity (PPV), can be limited to no higher than 5mm/sec for the neighbouring houses. However, we recommend that this limit be reviewed following completion of the dilapidation reports. If higher vibrations are recorded, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use a smaller plant or alternative techniques, e.g. grid sawing in conjunction with ripping. If hand held jack hammers are used, we consider no vibration minoring would be required due to the limited energy of such equipment.

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

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Use rock hammers in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil on the hammer.

We recommend use of excavation contractors with experience in such work with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

### 5.5 Seepage

Whilst groundwater was not observed during our assessment, seepage will probably occur at the soil/bedrock interface and through defects in the sandstone during and following heavy or prolonged rainfall.

During construction, any such seepage should be able to be controlled by gravity drainage.

In the long term, we recommend that subsoil drainage be incorporated behind all retaining walls and below any on-grade floor slabs, to collect and direct seepage to the stormwater system.

## 5.6 Batter Slopes and Bedrock Cut Faces

Where space permits, the soil profile, which is expected to predominantly comprise granular soils, can be temporarily battered at no steeper than 1 vertical to 1.5 horizontal, provided that no surcharge loads, including existing footings, construction loads, etc. are positioned at least H back from the crest of such batter slopes, where H is the height of the batter slope. Given the expected limited thickness of the soil profile, such batter slopes are expected to be able to be accommodated across the site. If such batter slopes cannot be accommodated in any areas, additional geotechnical advice must be sought.

Sandstone bedrock of consistent medium or greater strength is expected to be suitable to stand unsupported, at least in the short term, and possibly for the long term. However, the adoption of such unsupported bedrock cut faces in either the short or long term will be subject to the completion of regular inspections of all bedrock cut faces, initially at no greater than 1.0m depth intervals, to check for the presence of any adversely orientated defects, e.g. inclined joints, or weathered seams. If inclined joints are encountered, rock bolting would be expected to be required. For weathered seams, treatment by shotcreting or dry packing with non-shrink mortar would be required.

## 5.7 Retaining Walls

For design and construction of new retaining walls, the following are recommended:

- The walls should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, $k_0$, of 0.6, for the soil profile, assuming a horizontal retained surface.
- A bulk unit weight of 20kN/m$^3$ should be adopted for the retained soil profile.
- Any surcharge loads affecting the walls (e.g. construction loads, adjacent footing loads etc.) should be allowed for in the design using the above earth pressure coefficient.
- Complete and permanent drainage of the ground behind the walls should be provided. Subsurface drains must incorporate a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion.
- For retaining walls constructed immediately in-front of bedrock cut faces which have been assessed as suitable to stand unsupported, or with a void no greater than 0.2m thick which is backfilled with single size aggregate (blue metal), may be designed based on a uniform lateral earth pressure coefficient of 15kPa. Where a backfilled void increases beyond 0.2m thick, a reduced triangular lateral earth pressure coefficient of 0.35 can be adopted for such walls.
- Lateral toe restraint of retaining walls may be achieved by embedding the walls into the bedrock below adjacent excavation levels. An allowable lateral resistance of 200kPa can be adopted

for such design for sandstone bedrock of medium or greater strength. Care is required not to over-excavate in front of such walls, and all excavations in front of the walls, such as for footings etc. must be taken into account in the wall design.

## 5.8 Footings

Sandstone bedrock is expected to be at shallow depth or exposed over the entire footprint of the proposed alterations and additions and both ends of the existing house, and the existing house is also expected to be uniformly supported on footings founded in the sandstone bedrock profile. For this reason, and t maintain the stability of the slope and structures, it is essential that all new footings be uniformly founded in the sandstone bedrock profile.

For conventional pad or strip footings founded in sandstone bedrock of at least medium strength, an allowable bearing pressure of 1,000kPa can be adopted, based on serviceability criteria. All footings must be inspected by a geotechnical engineer to confirm that the appropriate foundation material is being achieved and that footings are not founded on boulders or detached blocks, which may also be present on the subject site. If doubt exists, that exposed sandstone may comprise a detached block, the geotechnical engineer may require that holes be drilled through the sandstone to confirm that sandstone bedrock is present. Further, for all footings located within 3m of the crest of a bedrock cut face, the stability of the bedrock cut face must also be taken into account, and additional support, or deepening of selected footings may be required. Additional advice would be provided at the time of the relevant inspection if such additional work is required.

All footings must be clean of any loose or water softened material and free of standing water prior to pouring concrete.

## 5.9 Proposed Boat Shed, Jetty and Mooring

Access to the southern end of the site, where these structures are proposed, was not possible at the time of our inspection. For the boat shed, assuming the subsurface conditions are consistent with those anticipated further to the north, the comments and recommendations in the above sections of the report can be adopted. If a deeper soil profile is encountered, or the bedrock is of lesser quality, additional geotechnical advice must be sought.

For the proposed jetty and mooring, advice must be sought from a specialist designer and/or contractor. We could complete a geotechnical investigation to confirm founding conditions for piles to support these structures if one is required, and we are commissioned to do so.

## 5.10 Further Geotechnical Input

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

- Attendance at a meeting by the builder, excavation contractor, ourselves and the project structural engineer to discuss the proposed works and outline the responsibilities of all parties.
- Geotechnical inspections during initial demolition and excavation to confirm the founding material of all footings in the vicinity of any excavations.
- Geotechnical inspection of bedrock cut faces.
- Geotechnical footing inspections.
- Geotechnical investigation for the jetty and mooring, if required.

# 6 GENERAL COMMENTS

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

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

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

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

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


| | | | |
|---|---|---|---|
| AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 27 DEC 2018. | Title: **SITE LOCATION PLAN** | | |
| | Location: 65 SEAFORTH CRESCENT SEAFORTH, NSW | | |
| | Report No: 32084PN | Figure No: 1 | |
| This plan should be read in conjunction with the JK Geotechnics report. | **JK Geotechnics** | | |

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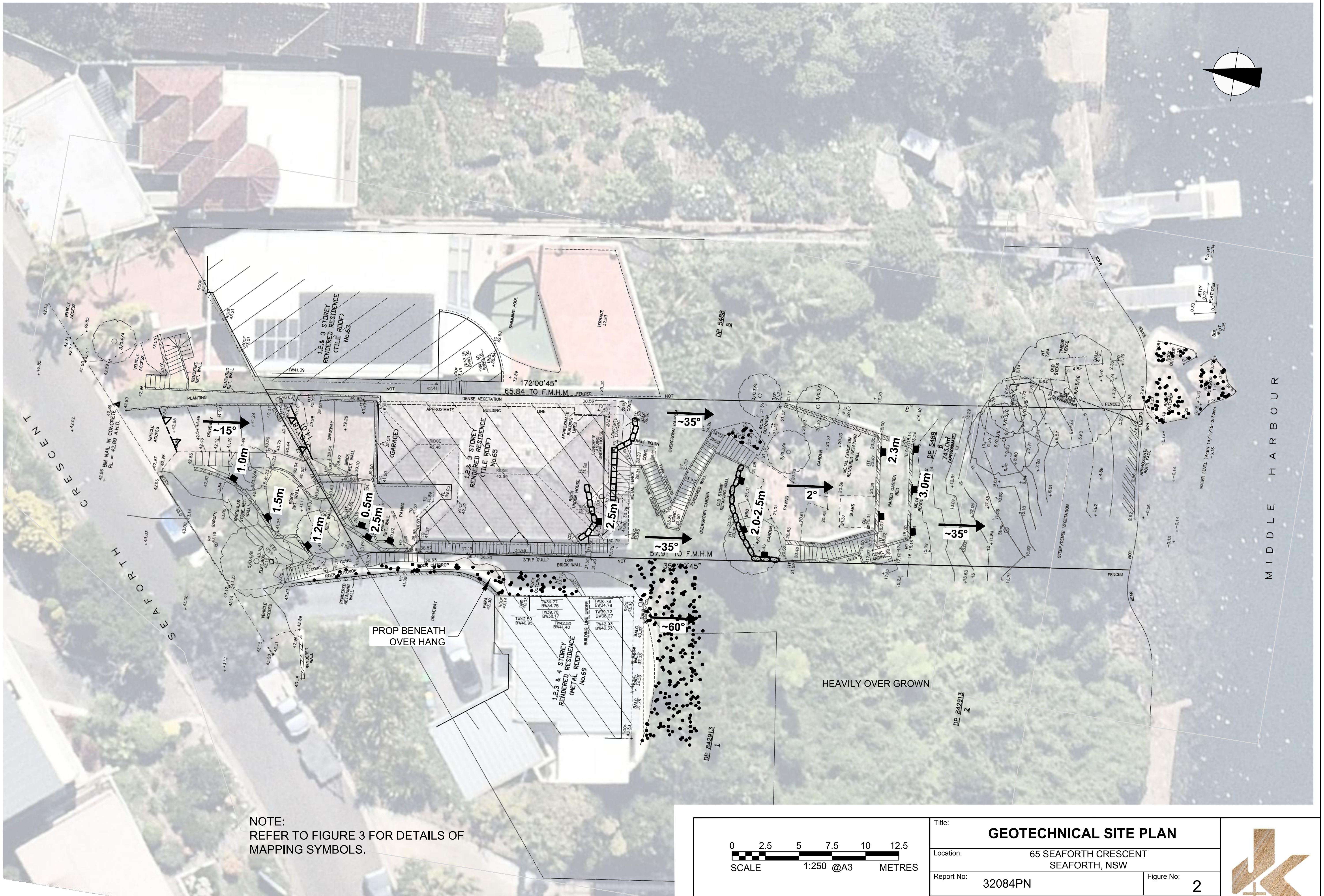


NOTE:
REFER TO FIGURE 3 FOR DETAILS OF MAPPING SYMBOLS.

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

0 2.5 5 7.5 10 12.5
SCALE 1:250 @A3 METRES

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

| | |
|---|---|
| Title: | GEOTECHNICAL SITE PLAN |
| Location: | 65 SEAFORTH CRESCENT SEAFORTH, NSW |
| Report No: 32084PN | Figure No: 2 |

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## TOPOGRAPHY

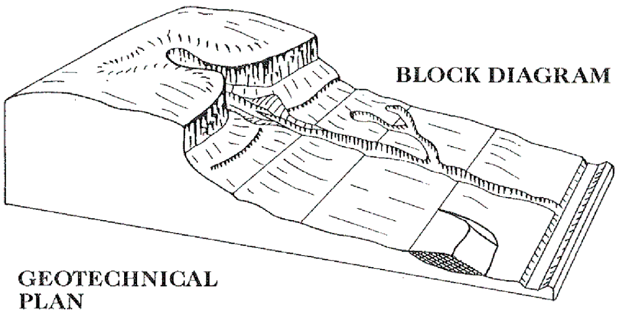
| *Symbol* | *Ground Profile* | | |
|---|---|---|---|
| | | convex | well defined or angular break of slope |
| | | concave | |
| | | convex | poorly defined or smooth change of slope |
| | | concave | |
| breaks of slope | | | convex and concave too close together to allow the use of separate symbols |
| changes of slope | | | |
| sharp | | | ridge crest |
| rounded | | | |
| 3 | Cliff or escarpment or sharp break 40° or more (estimated height in metres) | | |
| 15 | Uniform Slope | | Slope direction and angle (Degrees) |
| 10 | Concave Slope | | |
| 8 | Convex Slope | | |
| | Top | | Cut or fill slope, arrows pointing down slope |
| | Bottom | | |
| | Hummocky or irregular ground | | |

## OTHER FEATURES

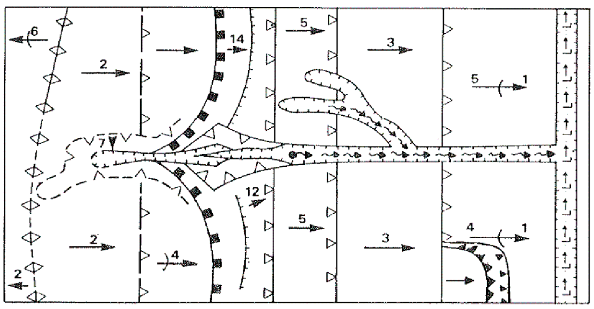
- Boulder
- Seepage/spring
- Swallow hole for runoff
- Natural water course
- Open drain, unlined
- Open drain, lined
- Fenceline
- Property boundary
- Dry Stone Wall
- J —— J 200 Major joint in rock face (opening in millimetres)
- – T – T – 10 Tension crack (opening in millimetres)
- Masonry or concrete wall
- Ponding water
- Boggy or swampy area

## EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

BLOCK DIAGRAM



GEOTECHNICAL PLAN




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

| | | | |
|---|---|---|---|
| | Title: **GEOTECHNICAL MAPPING SYMBOLS** | | |
| | Location: 65 SEAFORTH CRESCENT SEAFORTH, NSW | | |
| | Report No: 32084PN | Figure No: 3 | |
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## 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 ($\phi$), coefficient of consolidation ($C_h$), coefficient of permeability ($K_h$), unit weight ($\gamma$), 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 | $C_u > 4$<br>$1 < C_c < 3$ |
| | | 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 | $C_u > 6$<br>$1 < C_c < 3$ |
| | | 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:

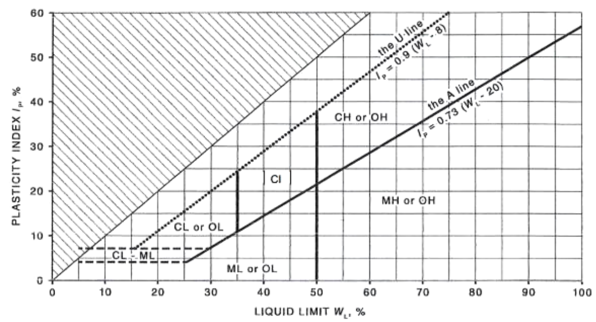
$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10}\ D_{60}}$$

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<br>U50<br>DB<br>DS<br>ASB<br>ASS<br>SAL | Sample taken over depth indicated, for environmental analysis.<br>Undisturbed 50mm diameter tube sample taken over depth indicated.<br>Bulk disturbed sample taken over depth indicated.<br>Small disturbed bag sample taken over depth indicated.<br>Soil sample taken over depth indicated, for asbestos analysis.<br>Soil sample taken over depth indicated, for acid sulfate soil analysis.<br>Soil sample taken over depth indicated, for salinity analysis. |
| Field Tests | N = 17<br>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<br>7<br>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<br>PID = 100 | Vane shear reading in kPa of undrained shear strength.<br>Photoionisation detector reading in ppm (soil sample headspace test). |
| Moisture Condition<br>(Fine Grained Soils) | $w$ > PL<br>$w \approx$ PL<br>$w$ < PL<br>$w \approx$ LL<br>$w$ > LL | Moisture content estimated to be greater than plastic limit.<br>Moisture content estimated to be approximately equal to plastic limit.<br>Moisture content estimated to be less than plastic limit.<br>Moisture content estimated to be near liquid limit.<br>Moisture content estimated to be wet of liquid limit. |
| (Coarse Grained Soils) | D<br>M<br>W | DRY – runs freely through fingers.<br>MOIST – does not run freely but no free water visible on soil surface.<br>WET – free water visible on soil surface. |
| Strength (Consistency)<br>Cohesive Soils | VS<br>S<br>F<br>St<br>VSt<br>Hd<br>Fr<br>( ) | VERY SOFT – unconfined compressive strength ≤ 25kPa.<br>SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.<br>FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.<br>STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.<br>VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.<br>HARD – unconfined compressive strength > 400kPa.<br>FRIABLE – strength not attainable, soil crumbles.<br>Bracketed symbol indicates estimated consistency based on tactile examination or other assessment. |
| Density Index/<br>Relative Density<br>(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<br>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) | Guide to Strength: Field Assessment |
|---|---|---|---|---|
| Very Low Strength | VL | 0.6 to 2 | 0.03 to 0.1 | Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure. |
| Low Strength | L | 2 to 6 | 0.1 to 0.3 | Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling. |
| Medium Strength | M | 6 to 20 | 0.3 to 1 | Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty. |
| High Strength | H | 20 to 60 | 1 to 3 | A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer. |
| Very High Strength | VH | 60 to 200 | 3 to 10 | Hand specimen breaks with pick after more than one blow; rock rings under hammer. |
| Extremely High Strength | EH | > 200 | > 10 | Specimen requires many blows with geological pick to break through intact material; rock rings under hammer. |

## Abbreviations Used in Defect Description

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