GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for____Williams River Steel____

Name of Applicant

Address of site __61 Darley Street, Mona Vale ______________________________

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, ________**Linton Speechley**__________ on behalf of JK Geotechnics Pty Ltd
(Insert Name) (Trading or Company Name)

on this the ____**3 November 2023**____ certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million.
we/I have:

Please mark appropriate box

☒ Prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ ~~I~~ *Are*/am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☒ Have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. *We*/I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ Have examined the site and the proposed development/alteration in detail and *are*/am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my/*our* report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.

☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: **Geotechnical Investigation and Assessment**

Report Date: **3 November 2023** Report Ref No: **35451LrptRev-1**
:
Author: **Linton Speechley**

Author's Company/Organisation: **JK Geotechnics Pty Ltd**

Documentation which relate to or are relied upon in report preparation:

Drawings Prepared by Williams River Steel (Job No.: JN613100, Drawing Nos: A000, A100 to A102, A200 to A202, A300, A400 to A402, A500, A600, A601, A700, A800 and A900 to A902, all revision 1, dated 2 February 2023)

~~I am~~ *We are* aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ~~ensuring~~ *confirming* that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk, *as discussed in the Report*.

Signature ………………………………………..……..

Name ……**Linton Speechley**…………………………………………..

Chartered Professional Status……**CPEng**…………………….

Membership No. …1417342………………………………………

Company: **JK Geotechnics Pty Ltd.**

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for____**Williams River Steel**_______________________________________

Name of Applicant

Address of site __**61 Darley Street, Mona Vale, NSW**______________________________

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: **Geotechnical Investigation and Assessment**

Report Date: **3 November 2023** Report Ref No: **35451Lrpt rev-1**

Author: **Linton Speechley**

Author's Company/Organisation: **JK Geotechnics Pty Ltd**

Please mark appropriate box

☒ Comprehensive site mapping conducted ____24 October 2023_______
(date)

☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)

☒ Subsurface investigation required

- ☐ No Justification …
- ☒ Yes Date conducted ……12 October 2022…………………………………………

☒ Geotechnical model developed and reported as an inferred subsurface type-section

☒ Geotechnical hazards identified

- ☐ Above the site
- ☐ On the site
- ☐ Below the site
- ☐ Beside the site

☒ Geotechnical hazards described and reported

☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009

- ☒ Consequence analysis
- ☒ Frequency analysis

☒ Risk calculation

☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009

☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009

☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009

☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the ~~specified conditions are achieved~~ *recommendations presented in the Report are adopted.*

☒ Design Life Adopted:

- ☒ 100 years
- ☐ Other …………………………………………….
specify

☒ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified

☒ Additional action to remove risk where reasonable and practical have been identified and included in the report.

☐ Risk assessment within Bushfire Asset Protection Zone.

~~I am~~ *We are* aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ~~ensuring~~ *confirming* that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk *as discussed in the Report*.

Signature … Speechley … ……………………………………..

Name …**Linton Speechley**………………………………………..……..

Chartered Professional Status……**CPEng**………………………

Membership No. …1417342…..………………………….

Company **JK Geotechnics Pty Ltd.**

REPORT TO

WILLIAMS RIVER STEEL

ON

GEOTECHNICAL INVESTIGATION & ASSESSEMENT

FOR

PROPOSED COMMERCIAL DEVELOPMENT

AT

61 DARLEY STREET, MONA VALE, NSW

Date: 9 November 2023
Ref: 35451Lrpt-rev1

JKGeotechnics
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Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
35451Lrpt	Final Report	8 December 2022
35451Lrpt-rev1	Stability Risk Assessment & Geotechnical Investigation Report	9 November 2023

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:
a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.
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<u>ATTACHMENTS</u>

1 INTRODUCTION

This report presents the results of a geotechnical investigation and stability risk assessment for the proposed commercial development at 61 Darley Street, Mona Vale, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by Mr Chris White of Williams River Steel by Purchase Order No. JN139594 dated 6 October 2023 and was carried out in accordance with our proposal, Ref:P59383L, dated 21 September 2023.

Based on the provided architectural drawings prepared by Williams River Steel (Job No.: JN613100, Drawing Nos: A000, A100 to A102, A200 to A202, A300, A400 to A402, A500, A600, A601, A700, A800 and A900 to A902, all revision 1, dated 2 February 2023), we understand that it is proposed to construct a two-storey commercial building over a single level basement. The basement is proposed to have a finished floor level at RL3.2m (mAHD), and will abut the north-western, south-western and south-eastern boundaries respectively, and will extend to within about 1.4m of the north-eastern boundary. Based on existing ground levels, maximum excavation depths of up to 5.3m will be required at the south-western end, reducing to about 3m at the north-eastern end.

JK Geotechnics have previously completed a geotechnical investigation at the site for Development Application (DA) purposes (Ref: 35451Lrpt, dated 8 December 2022). We note that at the time of the investigation, the design was at a preliminary stage. The geotechnical investigation was carried out in conjunction with a preliminary waste classification assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E35451Blet-WC dated 4 November 2022, for the results of the waste classification assessment.

We understand that following DA submission, that Northern Beaches Council have requested that a full stability risk assessment be undertaken for the site. The investigation results have been used as a basis for the stability risk assessment, and also to provide comments and recommendations on excavation conditions, shoring, footings, hydrogeology and floor slabs.

2 INVESTIGATION PROCEDURE

Geotechnical Subsurface Investigations

The fieldwork for the geotechnical subsurface investigations was carried out on 12 October 2022 and comprised the drilling of three (3) boreholes, BH1, BH2 and BH3, using our track-mounted JK309 drilling rig to depths ranging from 1.9m (BH3) to 10.0m (BH1) below existing surface levels. The boreholes were drilled using spiral auger techniques and a Tungsten Carbide (‘TC’) bit to the refusal depths of 3.6m and 1.9m in BH2 and BH3, respectively. BH1 was initially spiral auger drilled to a depth of 4.7m and was then extended to a depth of 10m using an NMLC triple tube barrel fitted with a diamond coring bit and water flush.

Prior to the commencement of fieldwork, the borehole locations were electromagnetically scanned by a specialist contractor to identify the location of any buried services. The investigation locations, as shown on Figure 2, were set out using a tape measure from existing surface features. At the time of the fieldwork, no survey drawings had been provided and therefore the surface levels of the boreholes are unknown. Should a survey become available, it should be provided to JK Geotechnics so that the surface levels can be included on the borehole logs.

The strength of the subsurface soils was assessed from Standard Penetration Test (SPT) 'N' values, and augmented by hand penetrometer test results on cohesive samples recovered by the SPT split tube sampler. The strength of the underlying weathered bedrock in BH2 and BH3, as well as the upper weathered bedrock in BH1, was assessed by observation of the resistance to drilling using a Tungsten Carbide (TC) bit attached to the augers, together with inspection of the recovered rock chip samples and subsequent correlation with laboratory moisture content test results. Estimation of rock strength by these methods is approximate only and variations of one strength order should not be unexpected. Where the sandstone was diamond cored in BH1, the recovered rock core was returned to our laboratory where the strength was assessed by Point Load Strength Index (Is_{50}) tests. The Point Load Strength results are shown on the borehole logs and in the attached Table A. Using established correlations, the estimated Unconfined Compressive Strength (UCS) of the rock was determined from the Is_{50} test results, which are also shown in Table A.

Groundwater observations were made in the boreholes during, on completion of drilling and at the end of the fieldwork. We note that water is introduced into the borehole during coring and therefore the water levels measured at completion of coring may be artificially high as the water levels have not had time to stabilise. In all three boreholes, Class 18 machine slotted PVC standpipes were installed and finished with a cast iron gatic cover to allow longer term groundwater monitoring to be completed. Details of the well installations are shown on the borehole logs. No continuous longer term groundwater monitoring has been carried out.

Our geotechnical engineer (Ben Sheppard) was present full time during the fieldwork to set out the investigation locations, nominate the testing and sampling, and prepare the attached borehole logs. For more details of the investigation procedures and their limitations, and a definition of the logging terms and symbols used, reference should be made to the attached Report Explanation Notes.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd (Envirolab), both NATA accredited laboratories, for laboratory testing of Moisture Content, Atterberg Limits, Linear Shrinkage and pH, sulphate content, chloride content and resistivity. The results of the tests are presented in the attached STS Tables A and B and Envirolab Certificate of Analysis No.307950.

Walkover Geotechnical Risk Assessment

Our Senior Geotechnical Engineer, Mr Ben Sheppard, visited site on 24 October 2023 to undertake the geotechnical stability risk assessment. The risk 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 site and 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).

The attached Figure 2 presents shows our mapping of relevant geotechnical and surface features both on the site and in its immediate surrounds, along with the potential geotechnical hazards and borehole locations. Figure 2 is based on the supplied survey drawing prepared by Duggan Mather Surveyors (Job No.2022167 TS1, dated 6 September 2022). 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. Figure 3 and Figure 4 presents typical cross-sections through the site based on the survey data augmented by our mapping observations. Figure 5 defines the mapping terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently undulating topography associated with the low-lying area situated between Mona Vale Beach and Pittwater Bay to the east and west, respectively. Ground levels within the area generally slope down to the north and north-east at about 3° to 4°. The site is bound to the north-east and south-east by Darley Street and Barrenjoey Road, respectively, and by commercial properties to the south-west and north-west.

At the time of the fieldwork, the site predominantly comprised of an Asphaltic Concrete (AC) surfaced carpark. The carpark was generally level along the rear (south-west), before sloping down to the north at about 4° to 5°. The AC pavement was generally in good condition, with some minor hairline cracking observed in isolated areas over the site. Some localised depressions were observed within the pavement within the northern corner of the site. An underground stormwater pit was observed to be about 1.4m deep in this area, although the lateral extent of the pit could not be confirmed. A single storey brick and clad structure was located within the western corner and appeared to be in good external condition, based on a cursory inspection. A concrete driveway slopes down from the carpark towards Darley Street at about 8°. To the north-west of the driveway is a landscaped timber retaining wall, with heights ranging from 0.1m and 1.1m, increasing in height to the east. The timber retaining wall was in fair condition.

The neighbouring property to the north-west of the site (No.63 Darley Street) contained a two-storey brick commercial structure positioned at the eastern end of the property and appeared in good condition based on a cursory external inspection from within the subject site. The adjoining structure on No. 63 Darley Street abutted the subject site boundary. The remainder of the adjoining property at No. 63 comprised a concrete pavement which was assessed to be generally in good condition. Ground levels on the adjoining No. 63 were lower than the subject site by between about 0.7m and 1.2m, from west to east, respectively, with the subject site retained by a double brick retaining wall which appeared to be in good condition. No obvious signs of rotation, distress or displacement were noted within the brick retaining wall.

The neighbouring property to the south-west of the site (No.25 Barrenjoey Road) contained a one to two storey concrete commercial structure which appeared to be in good external condition, besides some isolated horizontal hairline cracks towards the basal 3m of the north-western wall. The structure extended across the majority of the property footprint and abutted the boundary with the subject site.

Grassed road reserves are located adjacent to the north-east and south-eastern common boundaries.

3.2 Subsurface Conditions

The Sydney 1:100,000 Geological Series Sheet 9130 indicates that the site is underlain by the Newport Formation which comprises “interbedded laminate, shale, and quartz, to lithic-quartz sandstone”, however is close to the geological boundary to the overlying Quaternary aged alluvial deposits to the north.

The boreholes encountered a profile comprising pavements and fill overlying residual clays and weathered siltstone bedrock which graded into weathered sandstone bedrock at moderate to shallow depths. Groundwater was not encountered during augering of the boreholes. Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations. A summary of the subsoil conditions, as encountered, is presented below.

Pavements and Fill

Asphaltic Concrete (AC) was encountered at all test locations and ranged from 30mm thick in BH2 to 150mm thick in BH1. Fill was encountered at all test locations underlying the AC to depths of 0.55m, 0.6m and 0.6m, in BH1, BH2 and BH3, respectively. The fill predominantly comprised of silty clay, besides a 120mm thick layer underlying the AC in BH2, which comprised a silty sandy gravel. Inclusions within the fill included ironstone and igneous gravel and ash.

Residual Soil

Residual silty clay was encountered below the fill in all boreholes and extended to depths ranging from 1.3m in BH1 to 2.0m in BH2 below existing ground levels. The silty clay was assessed to be of high plasticity and generally of very stiff to hard strength, besides the upper portion of the residual clays in BH2 which were of stiff to very stiff strength. The clays had varying amounts of ironstone gravels.

Extremely Weathered Siltstone Bedrock

Extremely weathered siltstone was encountered below the residual silty clays in all boreholes and these extremely weathered siltstones extended to depths ranging from 1.7m in BH3) to 3.5m in BH2. The extremely weathered siltstone was assessed to be of hard soil strength and will remould to a material with soil like properties. The extremely weathered siltstone layer was 1.1m, 1.5m and 0.3m thick in BH1, BH2 and BH3, respectively.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered at depths of 2.4m, 3.5m and 1.7m in BH1, BH2 and BH3, respectively. The sandstone within BH2 and BH3 was of high strength on initial contact, with TC bit refusal occurring shortly after initial contact. BH1 encountered very low strength sandstone bedrock on first contact which extended to a depth of 4.6m, before which the sandstone increased to high strength.

Within the cored portion of BH1, the sandstone bedrock was fine grained and generally moderately weathered and of high strength to about 5.7m, reducing to low to medium strength to the termination depth of 10m. There is a significant number of defects within the cored portion of the bedrock and these comprised extremely weathered seams and clay seams up to 24mm thick, sub-horizontal bedding partings, joints inclined at up to 90°, and incipient and healed joints up to 90°.

Groundwater

Groundwater seepage was not encountered during auger drilling of the boreholes, which were all dry on completion of auger drilling. Groundwater levels were measured on 24 October 2023 (about 12 months after the completion of drilling) during our walkover geotechnical risk assessment. Groundwater was measured at depths of 2.75m (≈RL5.45m), 2.35m (≈RL5.45m) and 1.6m (≈RL5.0m) in BH1, BH2 and BH3 respectively.

3.3 Laboratory Test Results

The STS laboratory results are summarised in the attached Tables A and B.

The results of the Atterberg Limits and Linear Shrinkage tests on the residual silty clay samples confirms they are of medium to high and high plasticity, and therefore they would have a high potential for shrink-swell movements with changes in moisture content.

The moisture content tests on the rock samples correlated reasonably well with the field strength assessments. The results of the Point Load Strength Index tests carried out on the recovered rock cores from BH1 correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s(50)}$) tests generally ranged from 0.2MPa to 0.4MPa, besides the upper 1m, which had point load strength index results of 2.1MPa and 3.8MPa. These are also plotted on the attached borehole logs. Estimated unconfined compressive strength (UCS), based on the relationship of UCS = 20 x $I_{s(50)}$), ranged generally from 4MPa to 8MPa; however the upper rock profile had results of 42MPa and 76MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below. The Envirolab Certificate of Analysis No. 307950 is attached and provides further specific details for these tests.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
1	0.6-1.05	Residual Silty Clay	4.6	76	10	16,000
2	1.5-1.8	Residual Silty Clay	4.6	39	<10	33,000
2	0.2-0.4	Silty Clay Fill	7.4	55	10	5100

The above results indicate that the fill and residual soil would have an exposure classification of "Non-Aggressive" and "Mild", respectively, when assessed in accordance with the criteria of concrete piling exposure classification given in Table 6.4.2 (C) of AS2159-2009 "Piling Design Installation". The above results also indicate that the samples would have an exposure classification of "Non Aggressive" when assessed in accordance with the criteria for steel piling exposure classification given in Table 6.5.2 (C) of AS2159-2009 “Piling Design Installation”

4 GEOTECHNICAL RISK ASSESSEMENT

4.1 Potential Landslide Hazards

Based on our assessment, the site is gently sloping and therefore we consider that the primary geotechnical hazard in regard to stability will be;

A. Instability of proposed basement shoring walls.

4.2 Risk Analysis

The attached Table C summarises our qualitative assessment of the potential landslide hazard and of the consequences to property should the landslide hazard occur. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table C indicates that the assessed risk to property is Low, which would be considered ‘acceptable’ in accordance with the criteria given in Reference 1 and the Northern Beaches Council Risk Management Policy.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table D together with the resulting risk calculation. Our assessed risk to life for the person most at risk ranges from 6.6×10^{-8} to 4.1×10^{-8}. Therefore, the risk is considered acceptable in relation to the criteria given in Reference 1 and the Northern Beaches Council Risk Management Policy.

4.3 Risk Assessment

Risk Management requires suitable measures ‘to remove risk’. It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site and/or development cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners are made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, our recommendations there has included an active process of reducing risk, but it does not require the geotechnical engineer to warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

Similarly, we have assumed the design project life be taken as 100 years unless otherwise justified by the applicant. This provides the context within which the geotechnical risk assessment should be made. The required 100 years baseline broadly reflects the expectations of the community for the anticipated life of a residential structure and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 100 year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, as geotechnical engineers we do not warrant the development for a 100 year period, rather we have provided a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

In preparing our recommendations given below we have also assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all Council's buried services are, and will be regularly maintained to remain, in good condition.

We consider that our risk analysis has shown that the site and existing and proposed development can achieve an 'Acceptable Risk Management' criteria provided that the recommendations given in Section 5 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

5 COMMENTS AND RECOMMENDATIONS

5.1 Dilapidation

Prior to the commencement of any site works, including demolition of existing buildings/structures, we recommend that detailed internal and external dilapidation reports be carried out on adjoining properties to the north-west (No.63 Darley Street) and to the south-west (No.25 Barrenjoey Road), including boundary retaining walls. Dilapidation reports provide a record of existing conditions prior to commencement of any site works. The dilapidation reports 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.

The respective owners of the neighbouring properties should be asked to confirm in writing that the dilapidation report presents a fair assessment of existing conditions on their property. 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 reports should be reviewed by JK Geotechnics and the structural engineers prior to commencement of the works.

5.2 Demolition

There are nearby buildings and retaining walls around the site, and therefore demolition should be carried out with care, so as to not destabilise, or undermine any adjoining structures. This work will need to be carried out by suitably experienced (and insured) contractors.

Demolition of concrete slabs, possibly footings and paved surfaces will be required. We recommend that saw cut slots be provided near adjoining buildings, retaining walls and fences, such as near the south-western and north-western boundaries, and use be made of the buckets of hydraulic excavators to lift out pieces so as to reduce the risk of demolition vibrations being transferred to those adjoining structures.

Vibration monitoring should be undertaken at the commencement of demolition and during initial tracking of plant/equipment over the soils, to confirm that potentially damaging vibrations are not occurring. Whether further monitoring during demolition works are required would depend on the results of that initial monitoring. If concerns are raised about vibrations or damage to existing or adjoining structures then works should cease until an assessment can be made by the geotechnical and structural engineer or vibration specialists. A set of Vibration Emission Design Goals (VEDG) are attached for guidance. It would be advisable to try to obtain 'as built' drawings of any adjoining structures to assist with assessing the risk in this regard.

5.3 Excavation

Excavation recommendations provided below should be complemented by reference to the Code of Practice 'Excavation Work', prepared by Safe Work Australia July 2015 or latest revision at the time of works.

5.3.1 Excavation Conditions

Based on the boreholes, excavation for the proposed basement will encounter silty clay fill, residual silty clay, extremely weathered siltstone and weathered sandstone bedrock. Excavation of the soil profile and any extremely weathered or very low strength bedrock will be achievable using conventional earthmoving equipment using a 'digging' bucket fitted to a large size (say 20 tonne) hydraulic excavator. If layers of 'harder' iron-indurated bands or low strength siltstone/sandstone are encountered, then these should be able to be excavated using ripping tynes, provided they are no thicker than about 0.3m.

Sandstone of low or higher strength will be encountered and will require the use of rock excavation techniques for effective excavation. Rock excavation techniques include rock saws (possibly in combination with some ripping with a ripping tyne fitted to a large excavator) or rock grinders. High strength sandstone bedrock was encountered in all boreholes at depths of about 4.6m (≈RL3.6m), 3.5m (≈RL4.3m) and 1.7m (≈RL4.9m), in BH1, BH2 and BH3, respectively. At this stage we do not recommend the use of hydraulic impact hammers for rock excavation due to the risk of causing vibrational damage to adjoining structures. Hydraulic impact hammers would only be considered for use, if continuous quantitative vibration monitoring on adjoining structures and retaining walls is carried out as discussed in Section 5.3.2 below. A copy of this report (including the borehole logs) should be provided to any potential excavation contractor, who should confirm that they have reviewed this report and have allowed to undertake the excavation and monitoring in accordance with these recommendations.

At Bulk Excavation Level (BEL) we expect sandstone bedrock to be exposed. Only one borehole extended to below BEL, and so the quality of the bedrock at this depth is relatively unknown. However, based on the results of BH1, the sandstone bedrock is likely to be very low strength, and possibly high strength.

Groundwater was encountered between about RL5.0m and RL5.5m, which is above the BEL. At this stage, it is unknown whether the measured water level is a true representative groundwater table, or rather ephemeral groundwater seepages flowing across the top of the bedrock or sitting in bedrock 'low points'. Nevertheless, groundwater flows at these depths should be relatively minor during construction and we expect that they can be controlled by conventional sump and pump methods. Further groundwater testing and monitoring may be required, and is discussed further in Section 5.5.

5.3.2 Excavation Vibrations

Considerable 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 existing structures to the north-west and south-west which abut the common boundaries. Due to the relatively shallow depth to rock, we expect that the neighbouring buildings may be founded on the bedrock, however paving and other minor structures and walls are unlikely to be founded on rock. We recommend the neighbours be approached to provide details on the footings and founding conditions for their structures. We also recommend that where adjoining structures or boundary retaining walls abut the subject site boundary, that a few test pits be excavated at any early stage of the design process to assess and/or confirm the footing system and its founding stratum. This will assist in shoring designs.

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.

If excavation of any rock using hydraulic impact hammers is being considered, then it should commence away from likely critical areas and boundaries, using a moderately sized excavator fitted with a relatively low energy hydraulic impact rock hammer.

We recommend continuous quantitative vibration monitoring be carried out if rock excavation using hydraulic impact hammers is to be used. Vibration monitors should be set up at locations nominated by the geotechnical engineers, but these are likely to be on adjoining structures and boundary walls. The vibration monitors 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 by the structural and geotechnical engineers, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec on boundary walls and adjoining structures, assuming the boundary walls and adjoining structures are confirmed to be founded on rock. If boundary walls or adjoining structures are not founded on rock, then a lower PPV may need to be adopted. This limit takes both human comfort and potential structural damage into account and assumes that the structural engineers inspect the adjoining structures and confirms that these adjoining structures 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 recommend to only use excavation contractors with experience on similar sized projects and with a competent and experienced supervisor who is aware of vibration damage risks. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

5.4 Shoring

The depth of excavation ranges from 5.3m to 3m, within the south-western to north-eastern ends of the site, respectively. Based on the depth to weathered bedrock, and that the majority of the basement footprint extends up to the site boundaries, temporary excavation batters will not be feasible and all excavations will need to be supported by insitu shoring systems installed prior to excavation commencing.

Suitable retention systems will comprise of contiguous piled shoring walls or soldier pile shoring walls with shotcrete infill panels. Stiffer contiguous piled shoring walls are recommended adjacent to the adjoining structures (including adjoining boundary retaining walls) along the north-western and south-western boundaries, while soldier pile shoring systems would be suitable elsewhere provided there are no nearby adjoining movement sensitive services. As excavation progressed, the gaps between the contiguous piles must be dry packed or shotcreted to prevent the loss of material from behind the wall.

The shoring systems must be embedded to a suitable depth below bulk excavation level to provide overall lateral shoring wall stability. We note that BH2 and BH3 encountered refusal of our TC bit during drilling on high strength sandstone bedrock at depths of 3.5m and 1.9m respectively. High strength sandstone bedrock was also encountered at a depth of 4.6m in BH1. Therefore, allowance must be made by the shoring contractor for encountering high strength sandstone bedrock above the design shoring toe level. The shoring contractor should be provided with a copy of this report and should allow for suitable equipment to be able to penetrate through the high strength bedrock where necessary to satisfy the shoring design requirements.

Bored piles are considered suitable for the shoring system. However, groundwater is expected to flow into the pile holes and allowances must be made to remove the water prior to pouring concrete. Subject to further groundwater testing, and on the assumption that the groundwater is within the rock profile, seepage rates are expected to be relatively slow. All standing water, water softened materials and side wall collapse must be removed prior to pouring concrete. If standing water cannot be removed, concrete must be poured using tremie techniques.

At least the initial stages of shoring pile drilling should be inspected by a geotechnical engineer to ascertain that the recommended socket material has been reached and to check initial design assumptions. Inspection of piles will require the geotechnical engineer to be on site during the drilling process so that they can inspect both the material being drilled and check the material consistency with nearby borehole logs.

5.4.1 Shoring Wall Design Parameters

The major consideration in the selection of earth pressures and parameters for the design of the retention system is the need to limit deformations occurring outside the excavation. Based on the depth of excavation (up to 5.3m), we expect that the shoring wall will require anchoring or internal propping to limit wall movements.

The characteristic earth pressure coefficients and subsoil parameters provided below may be adopted for the design of the shoring systems.

- A bulk unit weight of 20kN/m^3 may be used for the soil and weathered rock.
- Where walls are to be progressively anchored or propped, then anchored or propped shoring systems may be provisionally designed based on a trapezoidal earth pressure distribution of magnitude 8H kPa (where H is the retained height in metres). These lateral pressures should be held constant for the central 50% of the pressure distribution.
- Where toe restraint of the piles is achieved by socketing the piles below bulk excavation level, a lateral restraint of 250kPa may be adopted for that portion of the socket founded within very low strength sandstone bedrock. The upper 0.5m of the socket formed below BEL should be ignored due to excavation disturbance and any localised deeper excavations. Higher lateral pressures may be adopted where higher strength rock is proven. For piles embedded into bedrock below bulk excavation level, a minimum embedment depth (ignoring the 0.5m allowance above) of 1m should apply. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, tanks, buried services, etc. must be taken into account in the wall design.
- The above lateral pressures assume horizontal backfill surfaces and where inclined backfill is proposed the pressures would need to be increased or the inclined backfill taken as a surcharge load.
- All surcharge loads affecting the walls (e.g. nearby footings, construction loads and traffic etc) are additional to the earth pressure recommendations above and should be included in the design.
- The shoring/retaining walls should be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.
- If the structural engineer wishes to design the contiguous pile walls using appropriate computer software, such as WALLAP or PLAXIS 2D, then further advice should be sought from JK Geotechnics for the provision of appropriate soil parameters. We have found that good design efficiencies can be achieved through the use of such methods.
- The wall designer must make an assessment of likely wall movements during all stages of the analysis and must check that such movements are within the acceptable tolerance for any adjoining structure, existing structure or service.

Anchors should have their bond formed within rock of at least very low strength, with the bond formed beyond a line drawn up at 45° from the base of the excavation. Preliminary design of anchors may be based on an allowable bond stress of 150kPa for sandstone bedrock of at least very low strength. All anchors should be proof loaded to at least 1.3 times the design working load before locking off at about 80% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Anchors are generally carried out on a design and construct basis so that failure of the anchors to hold their test load does not become a contractual issue. Permission must be obtained from adjoining property owners before installing anchors below their property.

Even with good design and construction, some vertical and lateral ground movements beyond the limits of the excavation may occur. The magnitude of movements is directly related to the stiffness of the shoring system and construction techniques used. Therefore, during shoring wall design, the wall designer must make an assessment of the likely shoring wall movements and associated adjoining ground movements, so that an assessment of the risk to adjoining buildings and services can be made.

5.5 Hydrogeological Considerations

Groundwater was measured between about RL5.0m and RL5.5m on 24 October 2023, which is above the BEL, and within the weathered sandstone and siltstone bedrock. From a geotechnical perspective, we consider that these materials (being weathered rock) will have a relatively low permeability and therefore groundwater inflows will be relatively minor. As such from a geotechnical perspective we consider that a drained basement will be feasible. During excavation, we expect that any seepage encountered should be controllable using sump and pump drainage techniques to appropriate discharge locations. Groundwater may require treatment prior to discharging and further advice should be sought from JK Environments.

We note that Water NSW has produced a recent document, “Minimum Requirements for Building Site Groundwater Investigations and Reporting”, dated January 2021 which outlines the minimum scope of investigation required where a basement is proposed and may intersect the groundwater table. As part of this, Water NSW will require at least three months of groundwater monitoring within three wells forming a triangular pattern across the site. Additional permeability testing, water quality testing and acid sulphate assessment will also be required.

The default position for Water NSW, is that where groundwater is encountered, the basement structure needs to be a water-tight (tanked) structure. To assess the feasibility for a drained basement, Water NSW will require the additional continuous groundwater monitoring, permeability testing of the bedrock, groundwater quality testing, and seepage computer analyses. All these results will need to be presented within a geotechnical and hydrogeological report.

5.6 Footings

We expect that weathered sandstone bedrock, ranging from very low strength to high strength may be exposed at bulk excavation level. We expect that pad or strip footings may be adopted to support the building loads.

Footings founded within sandstone bedrock of at least very low strength may be designed based on an allowable bearing pressure (ABP) of 1000kPa. At this stage, only one cored borehole has been drilled to below BEL and therefore limited information is known about the sandstone quality with depth at other locations within the site. If higher bearing pressures are required, additional cored boreholes would need to be drilled. If higher bearing pressures are required, the structural engineer should nominate what bearing pressures are required so that the additional boreholes can target such rock.

For piles socketed into sandstone bedrock of at least very low strength, an allowable shaft adhesion of 10% of the above allowable bearing pressures may be used for the design of piles in compression, or 5% for uplift, provided socket cleanliness and roughness is maintained.

All pad/strip footing excavations and pile drilling should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. Inspection of piles will require the geotechnical engineer to be on site during the drilling process so that they can inspect both the material being drilled and check the pile's consistency with nearby borehole logs.

Any loose of softened material should be removed from the base of pad/strip footings or bored piles, and all pad/strip footings and bored piles should be poured as soon as possible after excavation or drilling, cleaning and inspection.

5.7 Basement Slabs

The subgrade at bulk excavation level will likely comprise of variable quality sandstone bedrock ranging from very low to high strength sandstone. Basement slab-on-grade construction is therefore feasible.

Where sandstone bedrock of at least very low strength is exposed at bulk excavation level, no specific subgrade preparation is required. Provided a strong, durable gravel of at least 100mm thickness is used as the drainage layer, then this will also be suitable for support of the basement slab and will provide a separation layer from the underlying sandstone bedrock. We assume a drained basement will be adopted. Where the basement is designed as a drained basement then it should be underlain by a free draining drainage blanket or a grid of subsoil drains that direct groundwater seepage to a sump with fail-safe pump for pumped disposal. Drainage should be provided below all portions of the basement slab.

5.8 Further Geotechnical Input

Recommended Geotechnical input for the Construction Certificate

- All structural design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.
- All hydraulic design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

- An excavation/retention methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to the proposed excavation techniques, excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types and contingency plans in case of exceedances. The excavation/retention methodology must be reviewed and approved by the geotechnical engineer.

Further Geotechnical Investigation and Construction Inspections

- Additional cored boreholes, including UCS testing, if higher bearing pressures are required;
- Additional Groundwater monitoring to satisfy Water NSW, if required;
- Qualitative vibration monitoring at the commencement of demolition;
- Witnessing proof-load testing of anchors;
- Geotechnical inspection of the drilling of shoring piles;
- Geotechnical inspection of footings.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

Occasionally, the subsurface conditions between the 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.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Commercial Development
Location: 61 Darley Street, Mona Vale, NSW

Report No.: 35451L - A
Report Date: 26/10/2022
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	0.60 - 1.05	18.4	55	23	32	10.0
1	2.40 - 3.00	5.5	-	-	-	-
1	4.60 - 4.70	5.5	-	-	-	-
2	3.00 - 3.30	15.1	-	-	-	-
3	0.60 - 0.95	12.0	47	19	28	9.5
3	1.70 - 1.90	10.0	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 13/10/2022.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory
Number:1327

26/10/2022
Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Williams River Steel **Ref No:** 35451L

Project: Proposed Commercial Development **Report:** A

Location: 61 Darley Street, MONA VALE, NSW **Report Date:** 14/10/22

Page 1 of 1

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	4.75 - 4.78	3.8	76	A
	5.34 - 5.38	2.1	42	A
	5.88 - 5.92	0.4	8	A
	6.31 - 6.34	0.2	4	A
	6.81 - 6.85	0.3	6	A
	7.11 - 7.15	0.4	8	A
	7.81 - 7.84	0.2	4	A
	8.13 - 8.16	0.3	6	A
	8.91 - 8.95	0.2	4	A
	9.16 - 9.19	1.6	32	A
	9.45 - 9.48	0.5	10	A
	9.80 - 9.84	0.4	8	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 307950

Client Details

Client	JK Geotechnics
Attention	Ben Sheppard
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35451L, Mona Vale, NSW**
Number of Samples	3 Soil
Date samples received	13/10/2022
Date completed instructions received	13/10/2022

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	20/10/2022
Date of Issue	20/10/2022

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		307950-1	307950-2	307950-3
Your Reference	UNITS	BH1	BH2	BH2
Depth		0.6-1.05	1.5-1.8	0.2-0.4
Date Sampled		12/10/2022	12/10/2022	12/10/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	13/10/2022	13/10/2022	13/10/2022
Date analysed	-	19/10/2022	19/10/2022	19/10/2022
pH 1:5 soil:water	pH Units	4.6	4.6	7.4
Chloride, Cl 1:5 soil:water	mg/kg	10	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	76	39	55
Resistivity in soil*	ohm m	160	330	51

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			13/10/2022	[NT]	[NT]	[NT]	[NT]	13/10/2022	[NT]
Date analysed	-			19/10/2022	[NT]	[NT]	[NT]	[NT]	19/10/2022	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	98	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	104	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	107	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: WILLIAMS RIVER STEEL
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 61 DARLEY STREET, MONA VALE, NSW

Job No.: 35451L **Method:** SPIRAL AUGER **R.L. Surface:** ~8.2 m
Date: 12/10/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** B.S./L.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										ASPHALTIC CONCRETE: 150mm.t				NO OBSERVED REINFORCEMENT
						8			-	FILL: Silty CLAY, medium to high plasticity, red brown, brown and dark brown, trace of fine grained igneous gravel.	w>PL			
									-	CONCRETE: 150mm.t				NO OBSERVED REINFORCEMENT
					N = 10 3,5,5	7	1		CH	Silty CLAY: high plasticity, grey mottled red brown, trace of fine grained weakly cemented ironstone gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
					N = 26 10,12,14	6	2		-	Extremely Weathered siltstone: silty CLAY, medium plasticity, grey and red brown.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE SOIL 'TC' BIT RESISTANCE
ON 24/10/23									-	SANDSTONE: fine to medium grained, grey.	MW	VL		VERY LOW TO LOW RESISTANCE
						5 4	3 4			as above, but with occasional extremely weathered bands.				VERY LOW RESISTANCE
										SANDSTONE: fine to medium grained, grey.		H		HIGH RESISTANCE
						3 2	5 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 6.2m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.2m TO 2.2m. CASING 2.2m TO 0.1m. 2mm SAND FILTER PACK 6.2m TO 2.0m. BENTONITE SEAL 2.0m TO 0.1m. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

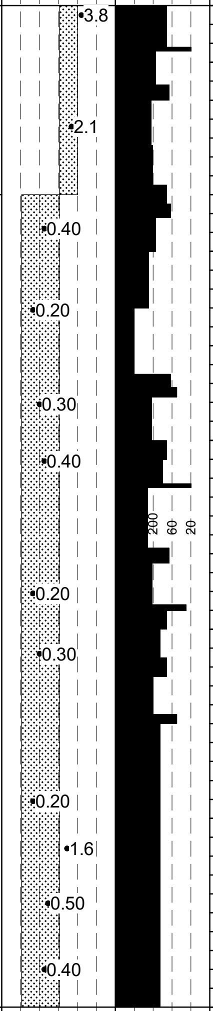
CORED BOREHOLE LOG

Borehole No.
1
2 / 2

Client: WILLIAMS RIVER STEEL
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 61 DARLEY STREET, MONA VALE, NSW

Job No.: 35451L	**Core Size:** NMLC	**R.L. Surface:** ~8.2 m
Date: 12/10/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK309	**Bearing:** N/A	**Logged/Checked By:** B.S./L.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	Formation
		4			START CORING AT 4.70m				
100% RETURN		3	5		SANDSTONE: fine grained, grey, red brown and orange brown, distinctly bedded at 0-15°.	MW	H	3.8 2.1	Newport Formation
		2 1 0 -1	6 7 8 9		SANDSTONE: fine grained, grey and orange brown, distinctly bedded at 0-10°, with occasional iron indurated bands.		L - M	0.40 0.20 0.30 0.40 0.20 0.30 0.20 1.6 0.50 0.40	
		-2	10		END OF BOREHOLE AT 10.00 m				



DEFECT DETAILS
DESCRIPTION
Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness
Specific / General

(4.82m) CS, 0°, 6 mm.t
(4.92m) Be, 0°, P, R, Clay Ct
(4.94m) Be, 0°, P, R, Clay Ct
(5.16m) CS, 0°, 12 mm.t
(5.23m) CS, 0°, 6 mm.t
(5.46m) CS, 0°, 24 mm.t
(5.57m) CS, 0°, 18 mm.t
(5.71m) J, 90°, P, Ji
(5.84m) Cr, 0°, 13 mm.t
(6.05m) J, 20°, P, R, Clay FILLED, 40 mm.t
(6.00-6.60m) Ji& Jh, 0 - 90°, P/Un, Fe
(6.57m) Cr, 0°, 24 mm.t
(6.63m) Jx 2, 70 - 90°, P
(6.72m) Be, 5°, P, R, Fe Sn
(6.88m) Be, 0°, P, R, Clay FILLED, 3 mm.t
(6.92m) Jh, 20°, Un, R, Fe
(7.17m) Bex 2, 0°, Un, R, Fe
(7.23m) Jx 2, 30°, P, R, Fe Sn
(7.25m) J, 45°, P, R, Fe Sn
(7.65m) Be, 0°, P, R, Fe
(7.73m) Be, 0°, P, R, Fe Sn
(7.87m) Be, 0°, P, R, Clay FILLED, 4 mm.t
(7.95m) Be, 0°, P, R, Clay FILLED, 4 mm.t
(8.00m) XWS, 0°, 24 mm.t
(8.20m) Be, 0°, P, R, Fe Sn
(8.25m) Be, 0°, P, R, Fe Sn
(8.45m) Be, 15°, P, Fe Sn
(8.51m) J, 30°, P, R, Fe Sn
(8.65m) Cr, 0°, 8 mm.t
(8.77-8.87m) ROCK IS FRACTURED, SEVERAL Be, 0 - 10°, P, R, Fe Sn, & J, 0-70°, P, Un, R, Fe, Sn
(9.12m) Be, 0°, P, R, Fe Sn
(9.26m) Cr, 0°, 9 mm.t
(9.81m) Bex 2, 20°, Un, R, Fe Sn
(9.95m) Cr, 0°, 8 mm.t

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

Job No: 35451L

Borehole No: 1

Depth: 4.70-10.00m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 1

Client: WILLIAMS RIVER STEEL
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 61 DARLEY STREET, MONA VALE, NSW

Job No.: 35451L **Method:** SPIRAL AUGER **R.L. Surface:** ~7.8 m
Date: 12/10/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** B.S./L.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION		-	ASPHALTIC CONCRETE: 30mm.t	M			NO OBSERVED REINFORCEMENT
			FILL: Silty sandy gravel, fine to coarse grained, dark grey, igneous gravel, fine to medium grained sand.	w>PL			
	N = 7 3,3,4	CH	FILL: Silty clay, medium plasticity, dark grey, trace of fine to coarse grained igneous and ironstone gravel, fine grained sand and ash.	w>PL	St - VSt	190 180 210	RESIDUAL
			Silty CLAY: high plasticity, orange brown and brown, trace of fine grained ironstone gravel.		VSt	240 300 310	
	N > 17 3,13,4/ 0mm REFUSAL	CI	as above, but grey mottled red brown.		VSt - Hd	420 350 350	
			Silty CLAY: medium plasticity, grey, trace of iron indurated bands and extremely weathered siltstone bands, trace of root fibres.				
ON 24/10/23	N > 21 17,21/ 150mm REFUSAL	-	Extremely weathered siltstone: silty CLAY, medium plasticity, grey and red brown, with iron indurated bands and extremely weathered sandstone bands.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE WITH LOW BANDS
		-	SANDSTONE: fine grained, red brown.	MW	H		HIGH RESISTANCE
			END OF BOREHOLE AT 3.60 m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 3.6m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.6m TO 2.1m. CASING 2.1m TO 0.1m. 2mm SAND FILTER PACK 2.6m TO 2.2m. BENTONITE SEAL 2.2m TO 0m. COMPLETED WITH A CONCRETED GATIC COVER.

SAMPLES: ES, U50, DB, DS

RL (m AHD): 7, 6, 5, 4, 3, 2, 1

Depth (m): 1, 2, 3, 4, 5, 6

BOREHOLE LOG

Borehole No.
3
1 / 1

Client: WILLIAMS RIVER STEEL
Project: PROPOSED COMMERCIAL DEVELOPMENT
Location: 61 DARLEY STREET, MONA VALE, NSW

Job No.: 35451L **Method:** SPIRAL AUGER **R.L. Surface:** ~6.6 m
Date: 12/10/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** B.S./L.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION AND AFTER 2 HRS	■ ■							▩	-	ASPHALTIC CONCRETE: 10mm.t	w~PL			NO OBSERVED REINFORCEMENT
								▩		FILL: Silty clay, low to medium plasticity, dark grey, fine to medium grained igneous gravel, fine grained sand and ash.				
	■			■	N = 8 8,3,5	6	1	▨	CH	Silty CLAY: high plasticity, red brown and orange brown, trace of fine to coarse grained ironstone gravel, with iron indurated bands.	w~PL	VSt - Hd	400 410 400	RESIDUAL
ON 24/10/23	■ ■			■	N > 4 8,4/ 50mm REFUSAL	5		▥	-	Extremely Weathered siltstone: silty CLAY, medium plasticity, grey, with iron indurated bands, trace of fine grained sand.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE
								░	-		MW	H		SOIL 'TC' BIT RESISTANCE
							2			SANDSTONE: fine to medium grained, red brown.				HIGH RESISTANCE
						4				END OF BOREHOLE AT 1.90 m				'TC' BIT REFUSAL
						3	3							GROUNDWATER MONITORING WELL INSTALLED TO 1.9m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.9m TO 1.4m. CASING 1.4m TO 0.1m. 2mm SAND FILTER PACK 1.9m TO 1.4m. BENTONITE SEAL 1.4m TO 0.1m. COMPLETED WITH A CONCRETED GATIC COVER.
						2	4							
						1	5							
						0	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 35451L MONAVALE.GPJ <<DrawingFile>> 09/11/2023 14:45 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

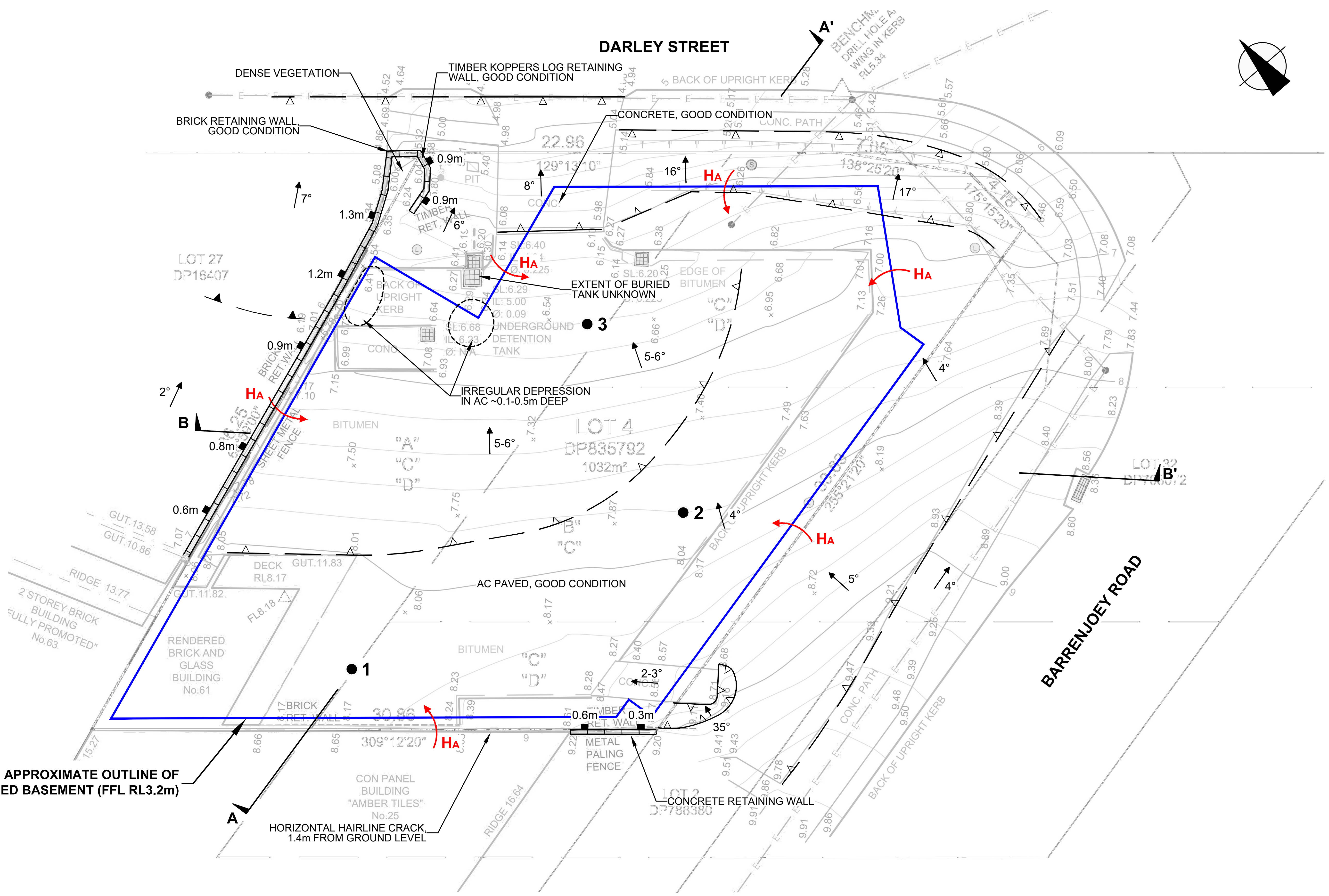
Location: 61 DARLEY STREET,
MONA VALE, NSW

Report No: 35451L

Figure No: 1

JKGeotechnics

PLOT DATE: 2/11/2023 1:26:27 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35451L MONA VALE\CAD\35451L.DWG



LEGEND

●	BOREHOLE
A A'	CROSS SECTION
HA	GEOTECHNICAL HAZARD

NOTES:
1. REFER TO FIGURE 3 FOR CROSS SECTION A-A'.
2. REFER TO FIGURE 4 FOR CROSS SECTION B-B'.
3. REFER TO FIGURE 5 FOR GEOTECHNICAL MAPPING SYMBOLS.

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN AND GEOTECHNICAL MAPPING SHOWING POTENTIAL HAZARDS**

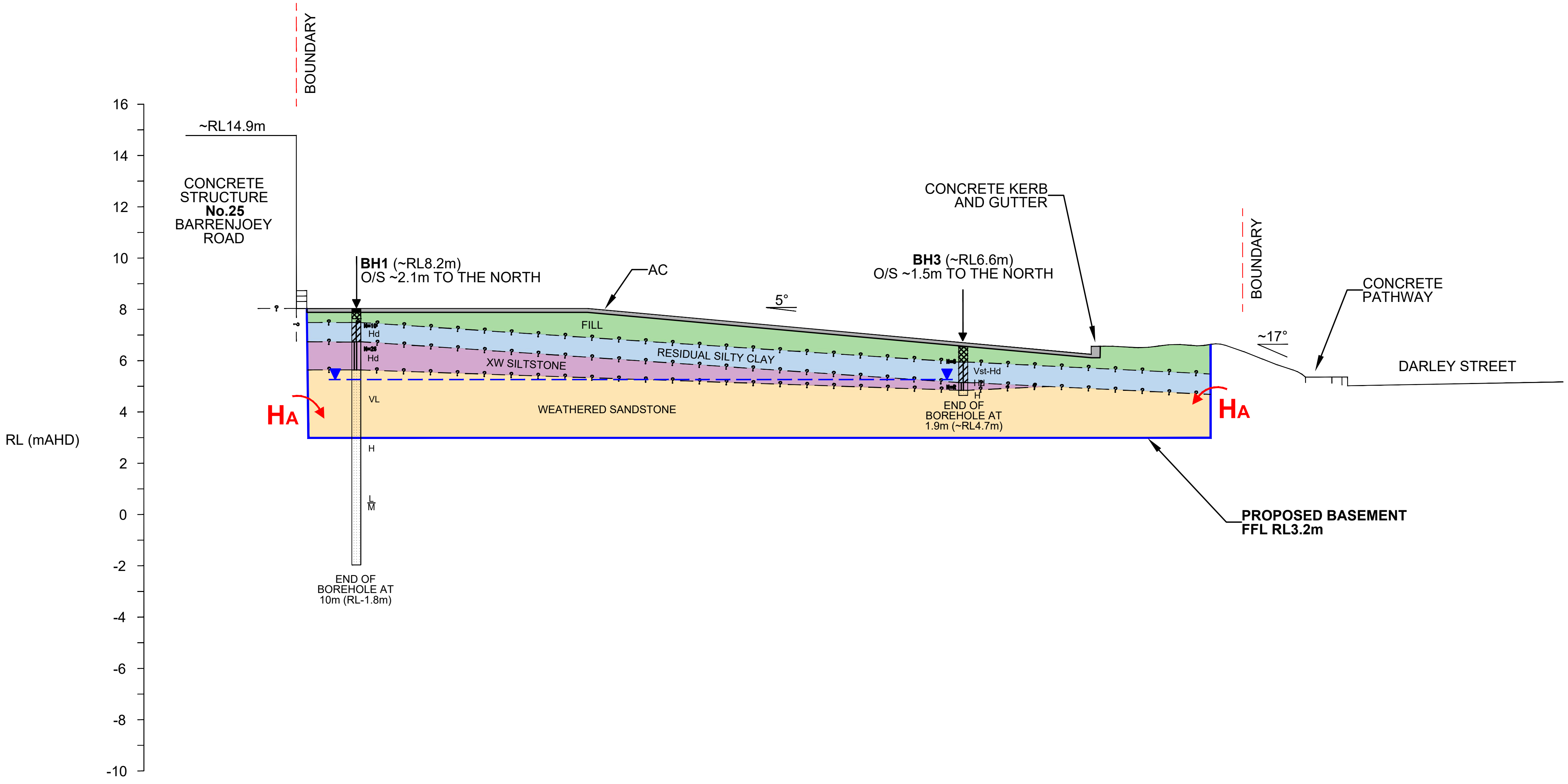
Location: 61 DARLEY STREET,
MONA VALE, NSW

Report No: 35451L

Figure No: 2

JKGeotechnics

PLOT DATE: 3/11/2023 9:16:58 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35451L MONA VALE\CAD\35451L.DWG



LEGEND

— ? — ? —	INFERRED GEOTECHNICAL BOUNDARY
━━━	PROPOSED BASEMENT (FFL RL3.2m)
– – – –	GROUNDWATER LEVEL AS MEASURED ON 24/10/23
↷ HA	GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

■	ASPHALTIC CONCRETE	▩	FILL
▨	SILTY CLAY	░	WEATHERED SANDSTONE
▢	CONCRETE	▥	XW SILTSTONE

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

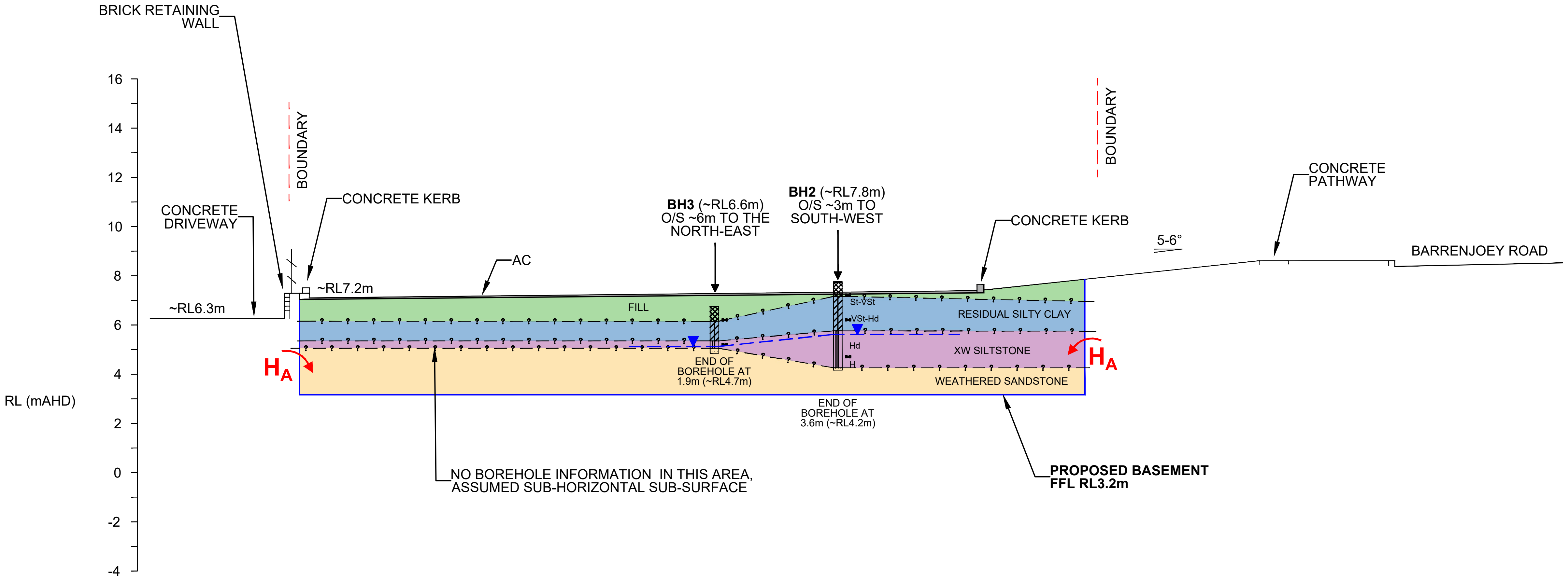
Title:
CROSS SECTION A-A'

Location:
61 DARLEY STREET,
MONA VALE, NSW

Report No: 35451L
Figure No: 3

JKGeotechnics

PLOT DATE: 3/11/2023 9:18:15 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35451L MONA VALE\CAD\35451L.DWG



LEGEND

—?—?— INFERRED GEOTECHNICAL BOUNDARY
—— PROPOSED BASEMENT (FFL RL3.2m)
– – – – GROUNDWATER LEVEL AS MEASURED ON 24/10/23
↷HA GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

■ ASPHALTIC CONCRETE
▨ SILTY CLAY
▫ CONCRETE
▩ FILL
🟨 WEATHERED SANDSTONE
▥ XW SILTSTONE

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title:
CROSS SECTION B-B'

Location: 61 DARLEY STREET,
MONA VALE, NSW

Report No: 35451L
Figure No: 4

JKGeotechnics

PLOT DATE: 3/11/2023 9:18:47 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35451L MONA VALE\CAD\35451L.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

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

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

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

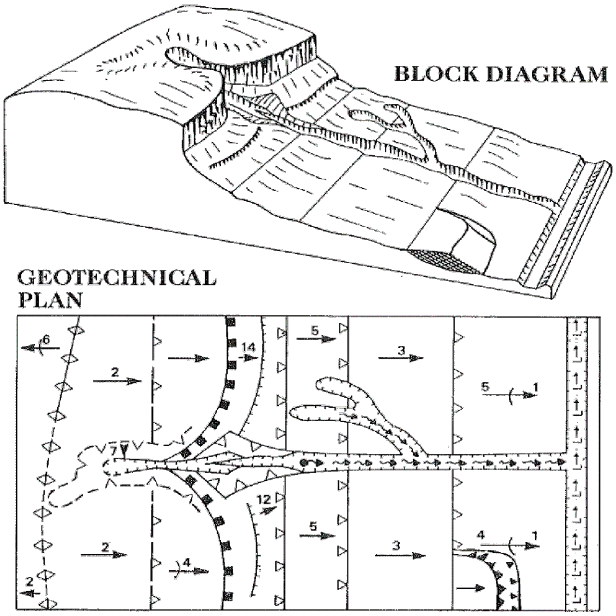
Hummocky or irregular ground

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:

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



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

Title:
GEOTECHNICAL MAPPING SYMBOLS
Location: 61 DARLEY STREET,
MONA VALE, NSW
Report No: 35451L
Figure No: 5
JKGeotechnics

TABLE C

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A Instability of Proposed Shoring Walls
Assessed Likelihood	Rare
Assessed Consequence	Medium to Major
Risk	LOW
Comments	All shoring walls to be engineer designed. Consequence outcome depends on the founding conditions of the neighbouring properties. Geotechnical aspects of the proposed shoring wall design to be checked by the geotechnical engineer. Geotechnical engineer to witness the initial stages of pile drilling to confirm socket material has been reached and cross check the material with the borehole logs.

TABLE D
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A** **Instability of Proposed Shoring Walls**
Assessed Likelihood	Rare
Indicative Annual Probability	**1 x 10^{-5}**
Duration of Use of area Affected (Temporal Probability)	(i) 8 hour/day during construction **3.3x10^{-1}** (ii) After Construction - Say an average of 8 hours per day occupancy **3.3x10^{-1}**
Probability of not Evacuating Area Affected	(i) 0.2 – Warning signs prior to failure (ii) 0.1 – Obvious warning signs prior to failure
Spatial Probability	(i) Say wall fails over half its length impacting 10% of the site. 0.1 (ii) Say wall failure impacts 25% of the building 0.25
Vulnerability to Life if Failure Occurs Whilst Person Present	(i) **1.0** – Crushed (ii) **0.5** – building may not collapse
Risk for Person most at Risk	(i) **6.6 x 10^{-8} During Construction** (ii) **4.1 x10^{-8} When Building Complete**

APPENDIX A

LANDSLIDE RISK
MANAGEMENT
TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

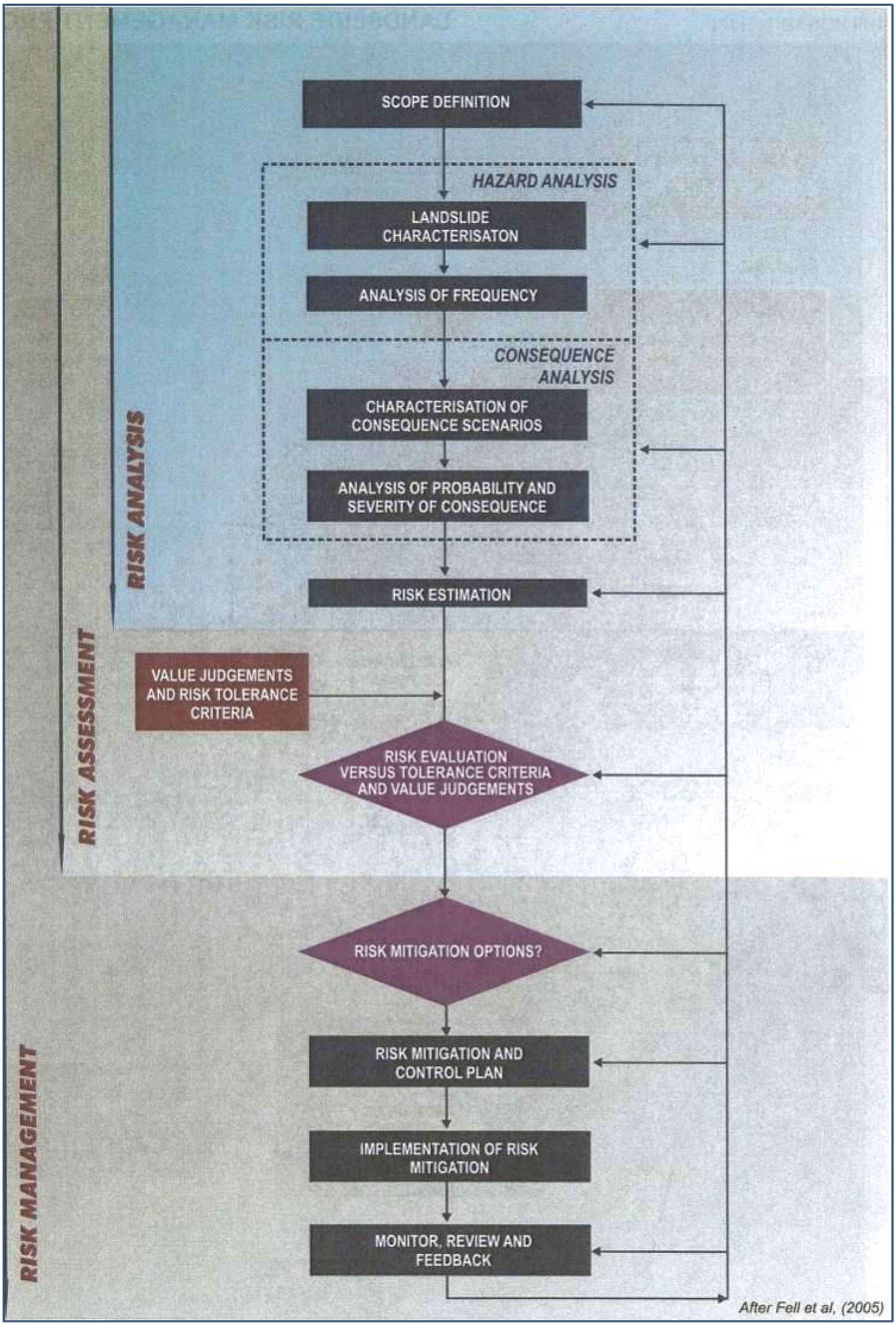
Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
Consequence	The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
Hazard	A condition with the potential for causing an undesirable consequence (the landslide). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.
Individual Risk to Life	The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
Landslide Intensity	A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. These are two main interpretations: (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability *(continued)*	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

NOTE: Reference should be made to Figure A1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.



After Fell et al, (2005)

FIGURE A1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-2}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate cost of Damage		Description	Descriptor	Level
Indicative Value	**Notional Boundary**			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (continued)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book “Guideline Document Landslide Hazards” published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board’s website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both “potential” and “actual” landslides must be taken very seriously. The present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with series consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can “run-out” from above, “regress” from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else’s land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

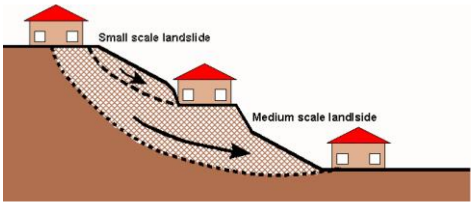


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

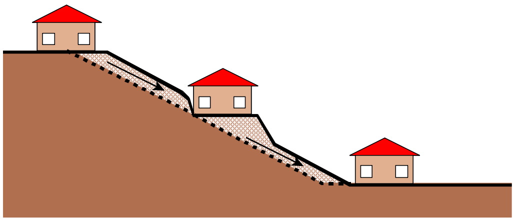


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

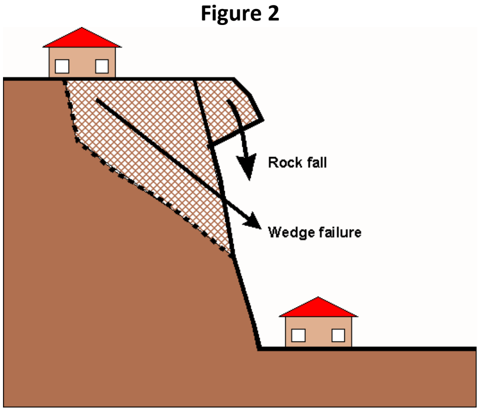


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

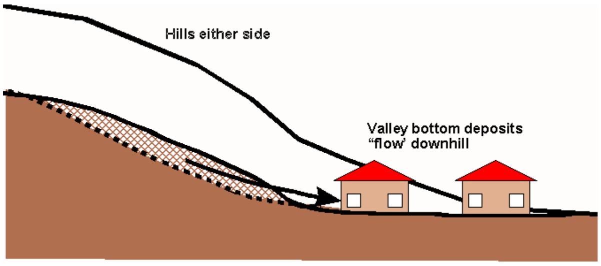


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of the repairs and temporary loss of use if the landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2 – LIKELIHOOD

Likelihood	**Annual Probability**
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable risk level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		**Significance - Geotechnical engineering requirements**
Very high	VH	**Unacceptable** without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**May be tolerated** in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding, ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other Australian GeoGuides:

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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 (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.

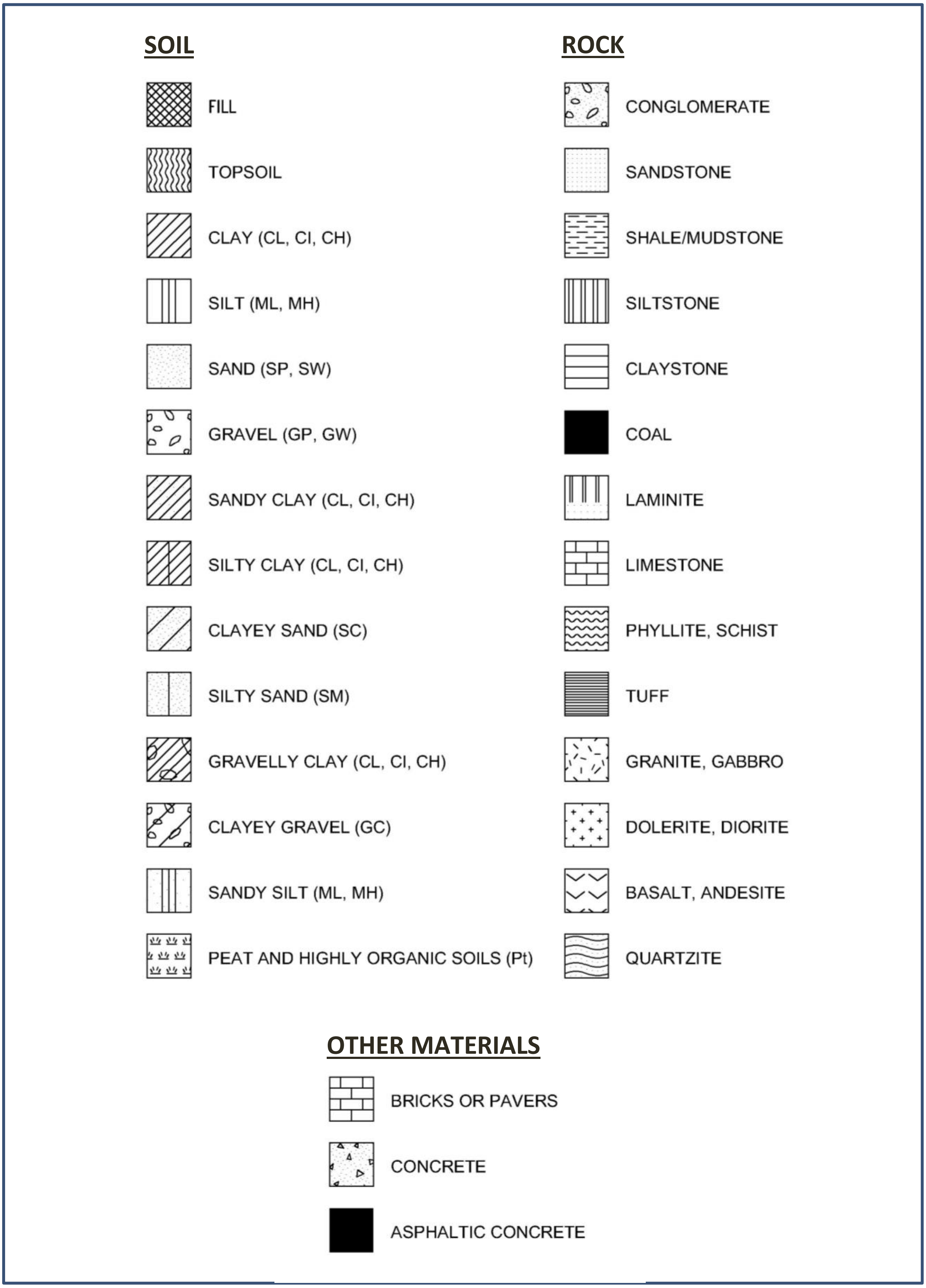
Requirements could range from:

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:

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

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

NOTES:

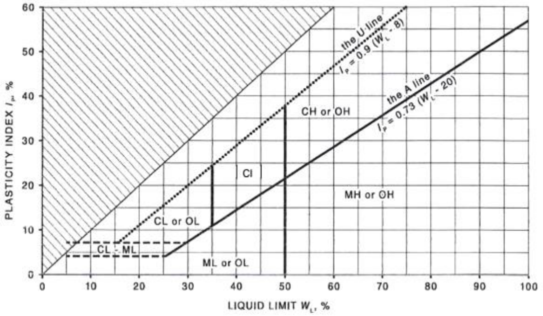
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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