GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for ______________________

Name of Applicant

Address of site ______________________

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

I, PAUL ROBERTS *on behalf of* **JK GEOTECHNICS**
(Insert Name) (Trading or Company Name)

on this the 3 JUNE 2021 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:

Please mark appropriate box

- [x] have 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
- [ ] Are 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/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my/our Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- [ ] have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- [ ] have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: GEOTECHNICAL ASSESSMENT FOR PROPOSED STABILISATION OF LANDSLIDE

Report Date: 3/6/21 Report Ref No: 32115RptZ rev1

Author: PAUL ROBERTS

Author's Company/Organisation: **JK GEOTECHNICS**

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

COASTAL ENGINEERING DRAWINGS 0001^C, 0011^C, 0013^D & 0021^B DATED 3/6/21, 0031^A dated 26/4/21 & 0041^B & 0042^A dated 14/5/21) & TECHNICAL SPEC. PA1900-RHD-00-SP-MA-0001 ver. 2, dated 4/2/21) PREPARED BY RHDHV

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 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 ......Paul Roberts......

Name ......PAUL ROBERTS......

Chartered Professional Status......CP Eng......

Membership No. ......2307698......

Company......**JK GEOTECHNICS**......

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

Development Application for ______________________________
Name of Applicant
Address of site ______________________________

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 ASSESSMENT FOR PROPOSED STABILISATION OF LANDSLIDE
Report Date: 3/6/21 Report Ref No: 32115Rrpt2 rev1
Author: PAUL ROBERTS
Author's Company/Organisation: JK GEOTECHNICS

Please mark appropriate box

- ☑ Comprehensive site mapping conducted 16/1/19 (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 16 & 17/1/19 & 9/5/19
- ☑ 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 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. N/A

We are aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for 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 Paul Roberts
Name PAUL ROBERTS
Chartered Professional Status CP Eng
Membership No. 2307698
Company **JK GEOTECHNICS**

REPORT TO

ROYAL HASKONING DHV

ON

GEOTECHNICAL ASSESSMENT (In Accordance with Pittwater Council Risk Management Policy)

FOR

PROPOSED STABILISATION OF LANDSLIDE

AT

FORESHORE AREA ADJACENT TO 148 HUDSON PARADE, CLAREVILLE, NSW

Date: 3 June 2021
Ref: 32115Rrpt2 rev1

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
32115Rrpt2	Final Report	23 March 2021
32115Rrpt2 rev1	Revision 1 following Council review	3 June 2021

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1 INTRODUCTION

This report presents the results of our geotechnical assessment for proposed seawall along a portion of the southern foreshore of Pittwater immediately to the north of 148 Hudson Parade, Clareville, NSW. The location of the site is shown in Figure 1. The assessment was commissioned on behalf of Northern Beaches Council (NBC) by Stephen Hjelm of Royal HaskoningDHV (RHDHV) in an email dated 8 February 2021. The commissions were on the basis of our proposal (Ref P51266RM Rev2) dated 27 January 2021. The site was inspected by our Associate geotechnical engineer on 5 February 2021, in order to assess the existing stability of the site and the effect on stability of the proposed development.

We note that we have prepared the following previous geotechnical reports for the site:

1. A geotechnical investigation report (Ref. 32115Rrpt Rev1) dated 31 January 2020 which included two phases of geotechnical investigation and numerical analysis of the original preferred solution for stabilisation of the existing seawall.
2. A geotechnical report (Ref. 32115RMlet) dated 31 March 2020 for temporary stabilisation measures to support the foreshore slope that had been impacted by the landslide that impacted the slope following an extreme rainfall event in February 2020.
3. A geotechnical design report (Ref. 32115RMlet2 rev2) dated 3 June 2021 which provided the results of our numerical analysis for the preferred option for permanent stabilisation of the foreshore area. The report included a "Specification for Permanent Rock Bolts, Shotcrete Seawall and Steel Mesh Facing".

With regard to the above geotechnical reports we provide below a brief summary of the project history:

- A report on the condition of the existing seawall (Project No 30014279, Register No SI – ST001, dated 29 May 2018) was prepared on behalf of Council by SMEC. The SMEC report assessed that due to the condition of the existing seawall, current risk levels were at 'Unacceptable' levels and recommended a range of short and long term remediation options in order to improve safety and which included four potential methods of improving the stability of the existing seawall. Based on the results of the site meeting between RHDHV and Council, a permanent seawall stabilisation solution was required by Council, with a 50 year design life.
- Two phases of geotechnical investigation were undertaken by JK Geotechnics (JKG) to assist in arriving at a preferred 'Long-Term Solution' of seawall remediation measures similar to 'Option 3' presented in the SMEC report. The intent of the proposed seawall remediation measures was to improve the stability of the foreshore slope and reduce risk to an 'Acceptable' level.
- Initially the preferred solution included removal of the existing seawall and construction of terraced gabion retaining walls. However, the preferred solution ultimately selected chose to leave the existing seawall in place in order to reduce the amount of excavation back into the slope. The remediation measures were therefore intended to comprise a new 0.45m thick reinforced concrete retaining wall constructed immediately in front of the existing seawall and re-profiling of the foreshore slope immediately landward of the new seawall crest, without constructing new terraced gabion retaining walls. JKG Report 1, dated 31 January 2020 (referenced above) provided geotechnical advice in relation to this preferred seawall remediation option.

- Following a low-pressure weather system developing with extreme rainfall in February 2020, a landside occurred at the site which was included failure of the entire length of existing seawall.
- Since the landslide, JKG and RHDHV have assisted NBC with implementation of temporary stabilisation works. The temporary stabilisation works that have been completed at the site have comprised removal of some of the old sea wall and landslide debris, and installation of a bulka bag wall installed along the overturned base of the collapsed which was founded directly on the siltstone bedrock wave cut platform. This work was completed under the direction of NBC and JKG and JKG have continued to undertake periodic site inspections (particularly after heavy rainfall events) to assess the stability of the site.
- The landslide made the previous preferred solution no longer feasible. JKG carried out an initial analysis of a revised preferred permanent stabilisation measures and presented the results in JKG Report 2, dated 31 March 2020 (referenced above). At the request of NBC, this report was peer reviewed by Douglas Partners and JKG were then engaged by NBC to complete a numerical analysis of the preferred option for permanent stabilisation of the foreshore area; the results were presented in JKG Report 3, dated 3 December 2020 (referenced above). Details of the proposed stabilisation measures are presented in Section 5 below. In summary, however, it is proposed to construct slope stabilisation measures comprising permanent rock bolts in conjunction with steel mesh to control near surface soil erosion over the slope face and include a permanent reinforced shotcrete face anchored in place by rock bolts to support the toe of the slope. A gabion wall would then be constructed to cover the seaward face of the shotcrete seawall in order to provide protection from waves and a more suitable aesthetic appearance to blend in with adjacent existing seawalls supporting the toes of the neighbouring portions of the foreshore slopes to the east and west.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 6 below. It is understood that the report will be submitted to Council as part of the DA documentation. Our report is preceded by the completed Council Forms 1 and 1a.

We note that JK Environments (JKE) have prepared a Remediation Action Plan (RAP); Ref. E32115Brpt2-RAP-rev1, dated 2 June 2021 and an Asbestos Management Plan (AMP); Ref. E32115Brpt3-AMP-rev1, dated 2 June 2021. This report should be read in conjunction with the RAP and AMP.

2 ASSESSMENT METHODOLOGY

2.1 Walkover Survey

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

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 7 following our geotechnical assessment.

The attached Figure 2 presents a geotechnical sketch plan showing the principal geotechnical features present at the site. Figure 2 is based on aerial imagery sourced form *Nearmap* with the provided survey plan (Plan No A1 – 10981D1^{C}, dated 11 October 2020) prepared by Byrne and Associates superimposed. Figure 3 also presents a geotechnical section through the site based on the survey data augmented by our mapping observations. Additional features on Figures 2 and 3 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed seawall remediation measures, we recommend they be located more accurately using instrument survey techniques.

2.2 Subsurface Investigation

The fieldwork for the investigation was carried out on 16 & 17 January and 9 May 2019. The investigation was limited by access constraints to the use of portable hand held equipment and comprised:

- Four boreholes (BH1, BH2, BH4 and BH5) were hand auger drilled to refusal depths between 0.8m to 3.2m below existing surface levels. The paved surfaces in BH4 were initially diatube core drilled (with water flush) to 0.6m depth.
- Two boreholes (BH4 and BH5) were then extended using Melvelle portable drilling equipment and wash boring techniques (with water flush) to respective depths of 1.7m and 2.0m.
- Five boreholes (BH1 to BH5) were extended using Melvelle portable drilling equipment and 'TT56' diamond coring techniques, with water flush, to depths between 3.12m (BH3) and 7.0m (BH5). BH3 was commenced on the bedrock wave cut platform immediately seaward of the seawall toe.
- Four Dynamic Cone Penetration (DCP) tests (DCP1, DCP2, DCP4 and DCP5) were completed adjacent to the respective boreholes. The DCP tests were extended to refusal depths ranging between about 2.0m and 3.2m below existing surface levels.

Prior to the commencement of both phases of fieldwork, the test locations were scanned for the presence of buried services by a specialist sub-contractor.

The test locations, as indicated on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface RLs at the test locations were interpolated between spot levels shown on the provided survey plan. The survey datum is the Australian Height Datum (AHD).

The compaction of the fill and the strength of the residual clayey soils were assessed from the DCP blow counts, augmented by hand penetrometer test results on remoulded cohesive soil samples recovered from the hand auger. The refusal depth of the DCP tests can also provide an indicative depth to bedrock, though we note that refusal can also occur on obstructions in fill, 'floaters' and other hard layers. The strength of the bedrock within the cored portions of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were made during and on completion of auger drilling, and on completion of wash boring and coring. We note that water is used as part of the wash boring and coring processes, and therefore water levels may not have stabilised in the short time period after core drilling. No longer term groundwater monitoring has been carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The fieldwork for the investigation was carried out under the direction of our geotechnical engineers (Warren Smith and Joanne Lagan), who were present full-time on site and set out the test locations, directed the buried services scans, logged the encountered subsurface profile, and nominated in-situ testing and sampling. The borehole logs (which include field test results and groundwater observations) and DCP test results sheets are attached, together with a glossary of logging terms and symbols used.

The recovered rock core was returned to the Soil Test Services (STS) NATA registered laboratory where it was photographed and Point Load Strength Index Tests completed. A summary of the Point Load Strength Index tests and estimated Unconfined Compressive Strengths are attached in STS Table A. The Point Load Strength Index tests are plotted on the cored borehole logs. The core photographs are included opposite the relevant cored borehole logs.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figures 2 and 3.

The site is located at the base of a hillside that steps and slopes down to a portion of the southern foreshore of Pittwater, to the west of Clareville Beach and about 300m east of Taylors Point headland and wharf. The subject portion of the foreshore slope extends down to the north from the northern boundary of No. 148 Hudson Parade, at an overall angle of about 33°.

At the time of our most recent inspection (5 February 2021), the site featured a landslide about 15m wide which, at the foreshore, extended westwards from the timber boatshed (the eastern end) to a neighbouring roughly mortared sandstone block retaining wall (1.5m height) founded on the bedrock platform, to the west. The width of the landslide was similar to the northern frontage of the upslope property (No. 148 Hudson Parade) and the length of the failed seawall.

The landslip extended upslope (to the south) about 6m to 7m from the rear of the temporary bulka bag retaining wall supporting the toe area of the landslide. The bulka bag wall comprised stacked gravel filled bags with plastic matting landward of the crest to provide temporary erosion protection from any wave overtopping. The bulka bag wall had been placed on the remains of the overturned concrete seawall (about 0.3m thick) which had failed in February 2020 and covered a portion of the stepped siltstone bedrock wave cut platform. The stepped bedrock wave cut platform extended east and west beyond the site boundaries.

Seaward of the bulka bag wall a temporary platform was also constructed as part of the works to facilitate the temporary stabilisation measures. The temporary platform was formed using imported large concrete blocks with a surface level at about RL1m. A temporary stockpile predominantly comprising sandstone blocks was located to the west of the platform area.

The slope above (landward) of the temporary plastic matting was partially covered with tarpaulin and extended to the sub-vertical landslide backscarp (about 1m to 2m height). The soil exposed beside and under the tarpaulin appeared to be predominantly silty sand fill with some areas of clay soil and gravel and cobble sized sandstone inclusions. Based on our previous site observations, the backscarp has been assessed to be situated at the location of stacked sandstone retaining wall which we assume collapsed due to the landslide About 1m to 2m upslope of the backscarp a stacked sandstone retaining wall (maximum height about 1m) supported a portion of the overgrown slope.

The portion of foreshore slope landward of the backscarp featured a narrow dilapidated concrete stepped walkway that extended south-westwards upslope from the boat shed. A stacked and roughly mortared sandstone retaining wall (maximum 2m height) formed the upslope side of the stepped walkway and in places the wall face was eroding and/or blocks were missing.

A densely vegetated area extended eastwards from the dilapidated stepped walkway and sandstone retaining wall and sloped down to the north at about 31°. A soil surfaced track extended east-west above the sandstone retaining wall and steep foreshore slope. To the south of the track the site surface extended landward to the retaining wall forming the northern boundary of No. 148 Hudson Parade and sloped down to the north at about 20° to 30°. The 'Dial Before You Dig' plans for the site area indicate that Telstra cables and a sewer main are located below the soil surfaced track and adjacent upper slope.

The uneven surfaced densely vegetated foreshore slope extended to the east and west beyond the site boundaries at similar inclinations. About 5m to the west, the neighbouring hillside had been terraced with sandstone steps winding down to the foreshore. Within the upper portion of the neighbouring slope to the west there were some exposures of residual soil and highly fractured (Class IV) sandstone bedrock.

A neighbouring gabion retaining wall (about 1m height) founded on the bedrock platform, extended eastwards from the eastern end of the site. The gabion wall face was bulging. Behind the crest of the gabion wall the neighbouring steep overgrown foreshore area sloped down to the foreshore at a maximum of about 35^{o}. The slope surface was uneven and several trees were leaning over from vertical back to the south.

The neighbouring roughly mortared sandstone seawall to the west was uneven and the eastern end adjacent to the toe of the foreshore slope area within the site, was in a dilapidated condition.

The southern site boundary (No 148 Hudson Parade) was formed by a concrete block retaining wall (about 1.2m to 1.8m height) which retained the grass covered rear yard. The wall was previously in a dilapidated state and has been supported by vertical steel posts and horizontal steel beams.

The neighbouring two and three storey brick house (No. 148 Hudson Parade) was set-back about 9.0m from the southern site boundary. A neighbouring pool with paved surrounds lined the neighbouring section of southern site boundary to the west. The pool surrounds were supported by a sandstone masonry retaining wall (about 1.8m height) formed the neighbouring section of southern site boundary to the west. The neighbouring two and three storey rendered house (No. 150) was set-back at least about 6.0m from the adjacent section of southern site boundary to the west.

A neighbouring one and two storey sandstone masonry and weatherboard house (No. 146) was set-back about 6.0m from the adjacent section of southern site boundary to the east; the neighbouring grass surfaced yard area extended to the crest of the retaining wall supporting the yard area.

Based on a cursory inspection from within the site, unless otherwise described above, and with the exception of the seawall within the site, the buildings and structures within and neighbouring the site appeared to be in good condition.

4 SUBSURFACE CONDITIONS AND LABORATORY TEST RESULTS

4.1 Subsurface Conditions

The 1:100,000 series geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone overlying the Newport Formation comprising interbedded Siltstone and Sandstone.

Generally, the boreholes encountered a variable thickness of fill over lying residual clays with bedrock exposed or encountered at shallow to moderate depth. Groundwater was not encountered over the depth of the investigation. Reference should be made to the attached borehole logs and DCP test results sheet for specific details at each location. Figure 3 presents a cross section through the site and provides a summary of the subsurface profile. The results of the geotechnical investigation are presented in Appendix B. A summary of the pertinent subsurface characteristics is presented below:

Paved Surfaces

The concrete paved surface of the stepped walkway at BH4 was 80mm thick and was overlying a 320mm thickness of sandstone blocks which we infer was an older surface of the walkway. The 200mm thickness of sandy fill below the sandstone blocks probably represent a bedding layer.

Fill

Fill comprising clayey topsoil assessed to be of low plasticity was encountered in BH1, BH2 and BH5 and ranged between 0.05m and 0.3m thickness. Below the topsoil fill in BH1 and BH2, silty sandy clay assessed to be of medium plasticity and with a varying gravel, cobble and boulder content extended to respective depths of about 1.8m and 1.1m. We note that the upper 0.6m of the wash bored section of BH1 (which extended down to 1.8m depth) was interpreted to indicate fill, based on the uneven progression of the wash bore equipment. We note that an approximately 2.0m thickness of fill was exposed in a sub-vertical face immediately landward of the return at the eastern end of the seawall. Based on the DCP test results, the fill was assessed to be poorly compacted.

Residual Clays

Residual silty and sandy clays of low to high plasticity and variable strength (firm to hard) were encountered below the fill or paved surfaces in all the boreholes except BH3, and extended depths of 3.0m (BH1), 3.2m (BH2), 2.0m (BH4) and 2.76m (BH5). In this regard, we note that the lower 1.2m wash bored section of BH1, the 0.9m wash bored section of BH4 and the upper 0.3m section of 'no core' in BH4 (1.7m to 2.0m depth) , and the upper 0.76m section of 'no core' in BH5 (2.0m to 2.76m depth) have been interpreted to indicate residual clays.

Weathered Bedrock

Weathered bedrock was encountered below the residual clays (BH1, BH2, BH4 and BH5) and from the wave cut platform surface (BH3). The bedrock surface steps and slopes down to the north to the foreshore from about RL8.4to about RL 0.6m (BH3) and is indicated on Figure 3. The bedrock has been interpreted to represent the lower section of Hawkesbury Sandstone overlying the Newport Formation exposed along the foreshore.

On first contact, Hawkesbury Sandstone comprising sandstone (with occasional siltstone interbeds) in BH4 and BH5 was assessed to be moderately and highly weathered and of variable strength; very low to medium (rarely high) strength with no appreciable improvement with depth and numerous 'no core' zones.

The interface with the underlying Newport Formation has been inferred below depths of about 5.5m (BH5) and 4.9m (BH4), and extends to the termination depth of the boreholes at 5.44m and 7.0m depth, respectively. BH1 to BH3 encountered the Newport Formation from surface level (BH3) and depths of 3.0m (BH1) and 3.2m (BH2). The Newport Formation comprised siltstone with sandstone interbeds. In BH1 the siltstone was of poor quality predominantly comprising 'no core' zones with bands of highly weathered bedrock of very low strength. In BH2 and BH3, the siltstone was assessed to be predominantly moderately weathered and of medium strength (BH2) and slightly weathered and of medium strength becoming very high strength. There was little improvement in the siltstone in BH2. However, below about 2.2m depth in BH3, sandstone assessed to be slightly weathered and of high strength was encountered.

There were a relatively large number of defects recorded in the recovered core and these comprised sub-horizontal extremely weathered seams (XWS) and clay seams (maximum 280mm thick), sub-horizontal to and moderately dipping bedding partings and moderately to steeply dipping curved, undulating and planar joints.

The following 'no core' zones (other than those described above interpreted to represent residual clays) were encountered in the boreholes:

- BH1: 3.28m depth (1.18m thick), 4.63m (0.92m thick).
- BH2: 3.2m depth (0.06m thick), 5.15m depth (0.1m thick).
- BH4: 2.26m depth (0.3m thick), 3.23m depth (0.25m thick), 4.17m depth (0.4m thick), and 4.91m depth (0.34m thick).
- BH5: 3.0m depth (0.8m thick), 4.12m depth (0.36m thick), and 5.53m depth (1.31m thick).

The 'no core' zones may be interpreted as representing XWS, clay seams and/or fractured bands.

Groundwater

All boreholes were dry during, and on completion of, hand auger drilling. Standing water flush levels were only recorded on completion of core drilling in BH3 at 0.2m depth. Water flush returns were variable and estimated to range between 50% and 100% which indicates a variable permeability rock mass. The loss of water flush and/or no recording of standing water levels in BH1, BH2, BH4 and BH5 is believed to be due to the presence of the 'no core' zones which probably represent zones of potential water flow through the rock mass. No longer term groundwater monitoring has been carried out.

4.2 Laboratory Test Results

The point load test results indicated that the rock cored in the boreholes was of very low to very high strength, with estimated Unconfined Compressive Strengths (UCS) ranging between 1MPa and 70MPa. Approximately 50% of the estimated UCS values ranged between 10MPa and 20MPa (medium strength) and 25% each ranged between 1MPa and 4MPa (very low to low strength) and between 26MPa and 70MPa (high to very high strength).

5 PROPOSED DEVELOPMENT

The agreed form of the stabilisation measures that was adopted for our detailed design analyses (refer to our geotechnical design report (Ref. 32115RMlet2 rev2) dated 3 June 2021 presented in Appendix C) were:

- Four rows of rock bolts spaced evenly between RL8.7m and RL2.4m comprising 25mm diameter fully threaded Glass-Fibre Reinforced Plastic bolts (GRP60) fully grouted in minimum 60mm diameter drilled holes.
- Composite high tensile steel mesh (such as Macaferri Steelgrid® HR PVC with hexagonal mesh) draped over the soil slope surface engaged with the rock bolts. Beneath the mesh hessian fabric or similar (such as Maccaferri MacMat HS) placed to prevent erosion and promote vegetation growth. Complemented by establishing suitable native vegetation species planted through the mesh.
- A seawall formed at the toe of the slope comprising a reinforced shotcrete face engaged with the fourth row of rock bolts (RL2.4m) and provided with fifth row of steeply inclined permanent toe rock bolts installed at RL1.0m and strip drains and weep holes installed at 1.0m lateral centres. The crest of the shotcrete seawall will be formed at RL3.0m to reduce wave overtopping and a reno mattress will be placed over the slope surface immediately landward of the seawall crest to control erosion from overtopping events. A gabion wall will also be placed on the foreshore bedrock platform immediately seaward of the shotcrete seawall.

We note that the full details of the proposed stabilisation measures are presented in the coastal engineering drawings (Drawing Numbers 0001^{C}, 0011^{C}, 0012^{C}, 0013^{D} and 0021^{B} dated 3 June 2021, 0031^{A} dated 26 April 2021 and 0041^{B} and 0042^{A} dated 14 May 2021) and Technical Specification (Ref. PA1900-RHD-00-SP-MA-0001 Version 2, dated 4 February 2021) prepared by RHDHV.

6 GEOTECHNICAL ASSESSMENT

The foreshore slope comprises a variable thickness soil cover (comprising fill and residual soils) overlying a bedrock surface that steps down to the foreshore. The fill was predominantly located over relative flat terraced areas supported by variable condition retaining walls and the landslide debris temporarily supported by a gravel filled bulka bag wall.

The bedrock is of variable quality, particularly at the transition between the Hawkesbury Sandstone and the underlying Newport Formation. Back (to the south) into the slope, we have assumed that the Hawkesbury sandstone improves in quality; close to the slope surface more extensive weathering of the bedrock profile typically occurs where seepage penetrates open defects which have developed due to stress relief effects as erosion over the geologically recent past has formed the Pittwater topography.

Following a low-pressure weather system developing with extreme rainfall in February 2020, a landside occurred at the above site which was previously retained by a dilapidated seawall referred to above. The length of the landslide was about 15m, (the same length as the former seawall) and extended back about 5m from the original face of the seawall and had slipped a similar distance onto the beach. The material exposed in the backscarp and surface of the landslip debris was a mix of silty sand fill with sandstone cobbles and natural sandy and clayey soils. Large sections of masonry wall debris were also evident in the debris.

Prior to the landslide the existing foreshore slope was supported by a dilapidated seawall and the landslide was most likely triggered by elevated water levels in the slope caused by a combination of discharge of existing stormwater pipes into the slope and/or ineffective drainage of groundwater behind the seawall such that the weight of saturated soils and elevated water levels caused collapse of the seawall.

To date, the temporary stabilisation works that have been completed at the site have comprised removal of some of the old sea wall and landslide debris, and installation of a bulka bag wall installed along the siltstone bedrock wave cut platform. This work was completed under the direction of NBC and JK Geotechnics (JKG) and JKG have continued to undertake periodic site inspections (particularly after heavy rainfall events) to assess the stability of the site. The temporary stabilisation measures that are protecting the toe of the slope have been performing satisfactorily to date. With regard to the landslide, installation of temporary stabilisation measures and on-going stability of the site, our advice has been presented in Site reports 1 to 10 prepared between 13 February and 4 November 2020.

6.1 Potential Landslide Hazards

We consider that the potential landslide hazards associated with the site to be the following:

A Instability of existing retaining walls:
- (i) Landscape retaining walls on the foreshore slope;
- (ii) Retaining wall supporting the rear yard of No. 148;
- (iii) Temporary foreshore bulka bag wall; and
- (iii) Seawalls neighbouring the site.

B Instability of the natural slope:
- (i) Large scale instability impacting the foreshore slope site area;
- (ii) Instability of the existing landslide backscarp;
- (iii) Large scale instability impacting the foreshore slope to the sides of the foreshore site area; and

(iv) Above the foreshore site area.

C Instability of temporary excavation batters.

D Instability of proposed foreshore stabilisation measures.

A number of these potential hazards are indicated in schematic form on the attached Figure 3.

6.2 Risk Analysis

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Use has been made of data in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. 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 A indicates that the assessed risk to property varies:

- Under existing conditions, between Moderate and Very Low.
- Following completion of the stabilisation measures, between Moderate and Very Low.

In accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy, Very Low and Low risk levels are considered 'Acceptable' and Moderate risk levels are considered 'Tolerable'. In this regard we note that the 'Tolerable' risk levels under existing conditions are associated with the existing landslip and the neighbouring foreshore slopes to the east and west and following construction of the stabilisation measures within the site are only associated with the neighbouring foreshore slopes.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal, spatial and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is as follows:

- Under existing conditions, ranges between about $5x10^{-5}$ and $9.2x10^{-13}$.
- Following completion of the stabilisation measures, ranges between about $5x10^{-6}$ and $9.2x10^{-13}$.

With regard to the above levels of risk to life, in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy we note the following:

- Under existing conditions, risk levels range between 'Tolerable' and 'Acceptable'. The 'Tolerable' risk level is associated with instability of the existing slope.
- Following completion of the stabilisation measures, risk levels are 'Acceptable' although we note that the risk level for the neighbouring foreshore slopes to the east and west is $5x10^{-6}$, which is midway between the 'Tolerable' and 'Acceptable' criteria.

6.3 Risk Assessment

The Pittwater Risk Management Policy 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, where the policy requires that 'reasonable and practical measures have been identified to remove risk', it means that there has been 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, the Pittwater Risk Management Policy requires that the design project life be taken as 100 years unless otherwise justified by the applicant. This requirement 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, the Policy does not seek the geotechnical engineer to warrant the development for a 100 year period, rather to provide a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.
In preparing our recommendations given below we have adopted the above interpretations of the Risk Management Policy requirements. 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 landslide site and the stabilisation measures can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy provided that the recommendations given in Section 7 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

7 COMMENTS AND RECOMMENDATIONS

We consider that the proposed stabilisation measures may proceed provided the following specific design, construction and maintenance recommendations are adopted to maintain and reduce the present risk of instability of the site and to control future risks. These recommendations address geotechnical issues only and other conditions may be required to address other aspects.

7.1 Conditions Recommended to Establish the Design Parameters

7.1.1 The site has been impacted by a landslide and detailed design of the stabilisation measures has been carried out by JK Geotechnics in conjunction with RHDHV and the results are presented in our geotechnical design report (Ref. 32115RMlet2 rev3) dated 3 June 2021 which provided the results of our numerical analysis for the preferred option for permanent stabilisation of the foreshore area. The report included a "Specification for Permanent Rock Bolts, Shotcrete Seawall and Steel Mesh Facing". Full details of the proposed stabilisation measures are presented in the coastal engineering drawings and Technical Specification prepared by RHDHV and referenced in Section 5 above. Reference should be made to our geotechnical design report for details of the design parameters adopted and the results of our 2 dimensional Finite Element analysis. The Factor of Safety (FOS) for the final construction stage of the stabilisation measures for a theoretical 'global failure' surface was 1.4 and therefore corresponded to a FOS of 1.65 for a SLOPE/W software analysis which employs limit equilibrium methods.

7.1.2 All proposed permanent rock bolts must be socketed/bonded into bedrock as outlined in our geotechnical design report (Ref. 32115RMlet2 rev2) dated 3 June 2021 and in accordance with the coastal engineering drawings and Technical Specification prepared by RHDHV.

7.1.3 The condition of the existing neighbouring seawalls that will abut the proposed reinforced shotcrete seawall may require localised stabilisation measures. These portions of the neighbouring seawalls should be inspected by the coastal and geotechnical engineers to confirm their condition, stability and detail any stabilisation measures that may be required. It may be that the interface between the proposed seawall and the neighbouring seawalls is modified such that support is provided by the new seawall.

7.1.4 Subject to inspection by a geotechnical engineer sub-vertical temporary batters for the proposed excavations at the toe of the slope to facilitate installation of the shotcrete wall will need to be completed in two vertical lifts (each a maximum 1.5m height) and at maximum lateral spacings of 3m to 4m. The lateral spacings represent the expected panel spacings within which rock bolts and shotcrete can be installed sequentially. The panels will need to be completed in a 'hit 1 miss 2' underpin style sequence to avoid large continuous sections of the toe of the slope remaining unsupported for short durations during the works. The actual spacings will be finalised in discussion with the Contractor based on their expected productivity in the tidal environment. All surcharge loads must be kept well clear of the excavation perimeter.

7.1.5 Where rock bolts will not run below adjoining properties. The number, lateral spacing and length of the rick bolts has been detailed to avoid rock bolts running below adjoining properties.

7.1.6 The existing stormwater drainage that discharges onto the slope must be diverted to the existing stormwater system or appropriately discharged at the foreshore.

7.1.7 The single sized granular material (or 'no fines' gravel) currently used to fill the bulka bags may be re-used as backfill and, where necessary, re-used as drainage material and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

7.1.8 Where a concrete capping layer is placed over the existing soil surface as part of the management of in-situ soils containing asbestos, then the existing subgrade must be proof rolled using a hand held whacker packer or small (say 2 tonne) smooth drum vibratory roller, where access permits. During proof rolling, adjoining structures must be closely monitored by the site supervisor and if there are causes for concern then the static (no-vibration) mode should be used or work immediately stop and this office be contacted for further advice. The aim of the proof rolling is to identify any soft or unstable areas, which if detected should be excavated down to a sound base and backfilled with thoroughly compacted engineered fill. Any localised excavation of in-situ soils must be carried out in accordance with the RAP and AMP.

7.1.9 Well graded imported granular materials such as demolition rubble would be suitable for use as engineered fill provided it is free of deleterious substances and has a maximum particle size not exceeding 40mm. Imported well graded granular fill should be compacted to at least 98% of Standard Maximum Dry Density (SMDD) and within 2% of their Standard Optimum Moisture Content (SOMC). Such fill should be compacted in horizontal layers as described above. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

7.2 Conditions Recommended to the Detailed Design to be Undertaken for the Construction Certificate

7.2.1 All coastal engineering design drawings and specifications must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

7.2.2 Dilapidation surveys must be carried out on the neighbouring rear yard structures to the south (No. 148). A copy of the dilapidation report must be provided to the neighbours and Council or the Principal Certifying Authority.

7.2.3 A construction method statement must be prepared prior to commencing the stabilisation measures. The method statement must include but not be limited to proposed site preparation, sequencing of rock bolt installation, sequencing of the works at the foreshore level in the tidal environment, geotechnical inspection intervals or hold points, environmental controls etc.

7.2.4 The construction method statement must be reviewed and approved by the geotechnical and coastal engineers.

7.3 Conditions Recommended During the Construction Period

7.3.1 The construction of the landslide stabilisation measures must be witnessed by the geotechnical and coastal engineers.

7.3.2 The geotechnical engineer must inspect the bedrock exposed at the wave cut platform prior to forming the reinforced shotcrete seawall.

7.3.3 The approved construction method statement must be followed.

7.3.4 The excavation sequencing and installation of rock bolts and reinforced shotcrete must be completed in accordance with the guidance provided in section 7.1.4 and as agreed between the Contractor, geotechnical and coastal engineers.

7.3.5 If they are to be retained, the existing stormwater system must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer, and repaired if found to be leaking. This is likely to include the adjacent properties at No. 146, 148 and 150.

7.3.6 The geotechnical engineer must inspect any subsurface drains that are required prior to backfilling.

7.3.7 The sewer running across the crest of the slope must be accurately located to avoid clashes with the upper row of rock bolts, and also checked for leaks and repaired as necessary, in a similar manner as described in paragraph 7.3.5 above for the stormwater system.

7.3.8 An 'as-built' drawing of all buried services at the site must be prepared (including all pipe diameters, pipe depths, pipe types, inlet pits, inspection pits, etc).

7.3.9 All rock bolts, steel mesh, reinforcing mesh and shotcrete must be in accordance with the "Specification for Permanent Rock Bolts, Shotcrete Seawall and Steel Mesh Facing" prepared by JK Geotechnics and presented as Appendix C of the Technical Specification (Ref. PA1900-RHD-00-SP-MA-0001 Version 2, dated 4 February 2021) prepared by RHDHV.

7.3.10 Compaction density of any engineered fill required to replace subgrade soft spots must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798 (Table 8.1), and the results submitted to the geotechnical engineer.

7.3.11 The geotechnical engineer must confirm that the proposed stabilisation measures have been completed in accordance with the geotechnical reports.

We note that all above Conditions must be complied with. Where this has not been done, it may not be possible for Form 3, which is required for the Occupation Certificate to be signed.

7.4 Conditions Recommended for Ongoing Management of the Site/Structure(s)

The following recommendations have been included so that the current and future owners of the subject property are aware of their responsibilities:

7.4.1 All existing and proposed subsurface drains must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than ten yearly intervals; including provision of a written report confirming scope of work completed (with reference to the 'as-built' drawing) and identifying any required remedial measures.

7.4.2 The stabilisation measures must be inspected by an experienced engineer/engineering geologist at ten yearly intervals; including provision of a written report confirming scope of work completed and identifying any required remedial measures.

7.4.3 The existing retaining walls on the foreshore slope that are to remain must be inspected by a structural engineer at no more than ten yearly intervals; including the provision of a written report confirming scope of work completed and identifying any required remedial measures.

7.4.4 No cut or fill in excess of 0.5m (e.g. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Council.

8 OVERVIEW

The existing foreshore slope has been impacted by a landslide which occurred during heavy rainfall in February 2020. The slope was supported by a dilapidated seawall and the landslide was most likely triggered by elevated water levels in the slope caused by a combination of discharge of existing stormwater pipes into the slope and/or ineffective drainage of groundwater behind the seawall such that the weight of saturated soils and elevated water levels caused collapse of the seawall.

The site is currently supported by temporary stabilisation measures including a bulka bag wall at the toe and some erosion protection. The temporary measures were installed on the understanding that a permanent solution would be formulated and constructed.

The proposed slope stabilisation measures seek to improve the stability of a marginally stable slope and improve risk levels. Otherwise, if left temporarily supported, the slope would, over time, continue to be subject to instability which could impact the foreshore, the boat shed, the track above the slope and possibly recede landward back towards the sewer and residential property at No. 148.

In our opinion, the proposed solution offers a more aesthetically acceptable outcome that the previous dilapidated seawall which was showing signs of instability and distress prior to its failure in 2020.

9 GENERAL COMMENTS

It is possible that the subsurface soil, rock or groundwater conditions encountered during construction may be found to be different (or may be interpreted to be different) from those inferred from our surface observations and previous subsurface investigations in preparing this report and previous reports. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

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.

Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

Reference 2: MacGregor, P, Walker, B, Fell, R, and Leventhal, A (2007) *'Assessment of Landslide Likelihood in the Pittwater Local Government Area'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp183-196.

TABLE A

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	**EXISTING CONDITIONS**								**DURING AND AFTER COMPLETION OF PROPOSED DEVELOPMENT AND IMPLEMENTATION OF RECOMMENDATIONS OUTLINED IN SECTION 7**								
	A - Stability of existing retaining walls				**B - Stability of the natural slope**				**A - Stability of existing retaining walls**			**B - Stability of the natural slope**				**C**	**D**
	(i) Landscape retaining walls on the foreshore slope	**(ii)** Retaining wall supporting the rear yard of No. 148	**(iii)** Temporary foreshore bulka bag wall	**(iv)** Seawalls neighbour-ing the site	**(i)** Large scale instability impacting the foreshore slope in the site	**(ii)** Instability of the existing landslide backscarp	**(iii)** Large scale instability impacting the foreshore slope to the sides of the foreshore site area	**(iv)** Above the foreshore site area	**(i)** Landscape retaining walls on the foreshore slope	**(ii)** Retaining wall supporting the rear yard of No. 148	**(iv)** Seawalls neighbour-ing the site	**(i)** Large scale instability impacting the foreshore slope in the site	**(ii)** Instability of the existing landslide backscarp	**(iii)** Large scale instability impacting the foreshore slope to the sides of the foreshore site area	**(iv)** Above the foreshore site area	Instability of temporary excavation batters	Instability of proposed foreshore stabilisation measures
Assessed Likelihood	Likely	Unlikely	Possible	Possible	Likely	Likely	Possible	Unlikely	Possible	Unlikely	Possible	Rare	Rare	Possible	Unlikely	Unlikely	Rare
Assessed Consequence	Insignificant	Minor	Insignificant	Insignificant	Minor	Insig-nificant	Minor	Insig-nificant	Insignificant	Minor	Insignificant	Major	Insig-nificant	Minor	Insig-nificant	Minor	Major
Risk	Low	Low	Very Low	Very Low	Moderate	Low	Moderate	Very Low	Very Low	Low	Very Low	Low	Very Low	Moderate	Very Low	Low	Low
Comments	A (i) to (iv), B (ii) and B (iv) – assumes localised instability. A (ii) – assumes stabilisation measures have been engineer designed								A (i), (ii) & (iv), B (ii) and B (iv) – assumes localised instability. A (ii) – assumes stabilisation measures have been engineer designed. B (i) and D – assumes stabilisation measures are constructed in accordance with the advice presented in this report and the RHDHV drawings and specification. C – assumes recommended batter slopes and sequencing of removal of bulka bags and excavations for the permanent support at the toe of the slope are carried out in accordance with the advice presented in this report and the RHDHV drawings and specification.								

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	EXISTING CONDITIONS								DURING AND AFTER COMPLETION OF PROPOSED DEVELOPMENT AND IMPLEMENTATION OF RECOMMENDATIONS OUTLINED IN SECTION 7								
	A - Stability of existing retaining walls				B - Stability of the natural slope				A - Stability of existing retaining walls			B - Stability of the natural slope				C	D
	(i) Landscape retaining walls on the foreshore slope	(ii) Retaining wall supporting the rear yard of No. 148	(iii) Temporary foreshore bulka bag wall	(iv) Seawalls neighbour-ing the site	(i) Large scale instability impacting the foreshore slope in the site	(ii) Instability of the existing landslide backscar p	(iii) Large scale instability impacting the foreshore slope to the sides of the foreshore site area	(iv) Above the foreshore site area	(i) Landscape retaining walls on the foreshore slope	(ii) Retaining wall supporting the rear yard of No. 148	(iv) Seawalls neighbour -ing the site	(i) Large scale instability impacting the foreshore slope in the site	(ii) Instability of the existing landslide backscarp	(iii) Large scale instability impacting the foreshore slope to the sides of the foreshore site area	(iv) Above the foreshore site area	Instability of temporary excavation batters	Instability of proposed foreshore stabilisation measures
Assessed Likelihood	Likely	Unlikely	Possible	Possible	Likely	Likely	Possible	Unlikely	Possible	Unlikely	Possible	Rare	Rare	Possible	Unlikely	Unlikely	Rare
Indicative Annual Probability	$1x10^{-2}$	$1x10^{-4}$	$1x10^{-3}$	$1x10^{-3}$	$1x10^{-2}$	$1x10^{-2}$	$1x10^{-3}$	$1x10^{-4}$	$1x10^{-3}$	$1x10^{-4}$	$1x10^{-3}$	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-3}$	$1x10^{4}$	$1x10^{-4}$	$1x10^{-5}$
Persons at risk	Person in foreshore area at the crest of the slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or in the rear yard.	Person in foreshore area at the crest of the slope or at the toe of the slope.	Person on the foreshore slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or at the toe of the slope.		Person on the foreshore slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or in the rear yard.	Person in foreshore area at the crest of the slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or in the rear yard.	Person on the foreshore slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or at the toe of the slope.		Person on the foreshore slope or at the toe of the slope.	Person in foreshore area at the crest of the slope or in the rear yard.	Persons at crest or workers within excavation	Person in foreshore area at the crest of the slope or at the toe of the slope.
Number of Persons Considered	2																
Duration of Use of area Affected (Temporal Probability)	Walking $4.6x10^{-5}$ (average walking rate of 4 seconds per 5m length for 12 months of the year). Stationary 0.01 (assumes 15 minutes per day) 1hr/day each i.e. 0.04 (yard)								Walking $4.6x10^{-5}$ Stationary 0.01 1hr/day each i.e. 0.04 (yard) Site personnel (crest of excavation) 1hr/day each over say 6 weeks i.e. $4.6 x 10^{-3}$ Site personnel (within excavation) 6hrs/day each over say 6 weeks i.e. 0.03								
Probability of not Evacuating Area Affected	Slope crest 0.01 Slope toe 0.1	0.01	Slope crest 0.01 Slope toe 0.1	0.1	Slope crest 0.1 Slope toe 0.5	0.01	0.5	0.1	0.1	0.01	0.1	Slope crest 0.1 Slope toe 0.5	0.01	0.5	0.1	0.4	Slope crest 0.1 Slope toe 0.5

Spatial Probability	1m failure over 5m length of wall i.e. 0.2		3m failure over 15m length of excavation i.e. 0.2	3m failure over assumed 15m length of seawall i.e. 0.2	1 (impacts entire slope)	1m failure over 5m length of backscarp i.e. 0.2	1 (impacts entire slope)	1m failure over 5m length of slope i.e. 0.2	1m failure over 5m length of wall i.e. 0.2		3m failure over assumed 15m length of seawall i.e. 0.2	1 (impacts entire slope)	1m failure over 5m length of backscarp i.e. 0.2	1 (impacts entire slope)	1m failure over 5m length of slope i.e. 0.2	3m failure over 15m length of excavation i.e. 0.2	1 (impacts entire length of stabilised slope)
Vulnerability to Life if Failure Occurs Whilst Person Present	0.1	0.1	0.1	0.1	1	0.01	1	0.01	0.1	0.1	0.1	1	0.01	1	0.01	0.1	1
Risk for Person most at Risk	$9.2x10^{-11}$ Walking crest $2x10^{-8}$ Stationary crest $9.2x10^{-10}$ Walking slope toe $2x10^{-7}$ Stationary slope toe	$9.2x10^{-13}$ Walking crest $2x10^{-10}$ Stationary crest $8x10^{-10}$ Rear yard	$9.2x10^{-12}$ Walking crest $2x10^{-9}$ Stationary crest $9.2x10^{-11}$ Walking foreshore $2x10^{-8}$ Stationary foreshore	$9.2x10^{-12}$ Walking $2x10^{-9}$ Stationary	$4.6x10^{-8}$ Walking crest $1x10^{-5}$ Stationary crest $2.3x10^{-7}$ Walking foreshore $5x10^{-5}$ Stationary foreshore	$9.2x10^{-12}$ Walking $2x10^{-9}$ Stationary	$2.3x10^{-8}$ Walking $5x10^{-6}$ Stationary	$9.2x10^{-13}$ Walking $2x10^{-10}$ Stationary	$9.2x10^{-12}$ Walking crest $2x10^{-9}$ Stationary crest $9.2x10^{-11}$ Walking slope toe $2x10^{-8}$ Stationary slope toe	$9.2x10^{-13}$ Walking crest $2x10^{-10}$ Stationary crest $8x10^{-10}$ Rear yard	$9.2x10^{-12}$ Walking $2x10^{-9}$ Stationary	$4.6x10^{-11}$ Walking crest $1x10^{-8}$ Stationary crest $2.3x10^{-10}$ Walking slope toe $5x10^{-8}$ Stationary slope toe	$9.2x10^{-15}$ Walking $2x10^{-12}$ Stationary	$2.3x10^{-8}$ Walking $5x10^{-6}$ Stationary	$9.2x10^{-13}$ Walking $2x10^{-10}$ Stationary	$3.7 x 10^{-9}$ $2.4 x 10^{-8}$	$4.6x10^{-11}$ Walking crest $1x10^{-8}$ Stationary crest $2.3x10^{-10}$ Walking slope toe $5x10^{-8}$ Stationary slope toe
Total Risk	$1.8x10^{-10}$ Walking crest $4x10^{-8}$ Stationary crest $1.8x10^{-9}$ Walking slope toe $4x10^{-7}$ Stationary slope toe	$1.8x10^{-12}$ Walking crest $4x10^{-10}$ Stationary crest $1.6x10^{-9}$ Rear yard	$1.8x10^{-11}$ Walking crest $4x10^{-9}$ Stationary crest $1.8x10^{-10}$ Walking slope toe $4x10^{-8}$ Stationary slope toe	$1.8x10^{-11}$ Walking $4x10^{-9}$ Stationary	$9.2x10^{-8}$ Walking crest $2x10^{-5}$ Stationary crest $4.6x10^{-7}$ Walking slope toe $1x10^{-4}$ Stationary slope toe	$1.8x10^{-11}$ Walking $4x10^{-9}$ Stationary	$4.6x10^{-8}$ Walking $1x10^{-5}$ Stationary	$1.8x10^{-12}$ Walking $4x10^{-10}$ Stationary	$1.8x10^{-11}$ Walking crest $4x10^{-9}$ Stationary crest $1.8x10^{-10}$ Walking slope toe $4x10^{-8}$ Stationary slope toe	$1.8x10^{-12}$ Walking crest $4x10^{-10}$ Stationary crest $1.6x10^{-9}$ Rear yard	$1.8x10^{-11}$ Walking $4x10^{-9}$ Stationary	$9.2x10^{-11}$ Walking crest $2x10^{-8}$ Stationary crest $4.6x10^{-10}$ Walking slope toe $1x10^{-7}$ Stationary slope toe	$1.8x10^{-14}$ Walking $4x10^{-12}$ Stationary	$4.6x10^{-8}$ Walking $1x10^{-5}$ Stationary	$1.8x10^{-12}$ Walking $4x10^{-10}$ Stationary	$3.7 x 10^{-9}$ $2.4 x 10^{-8}$	$9.2x10^{-11}$ Walking crest $2x10^{-8}$ Stationary crest $4.6x10^{-10}$ Walking slope toe $1x10^{-7}$ Stationary slope toe

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 28 DEC 2018.

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

Title:	SITE LOCATION PLAN
Location:	FORESHORE AREA ADJACENT TO, 148 HUDSON PARADE, CLAREVILLE, NSW
Report No: 32115Rrpt2	Figure No: 1

JKGeotechnics

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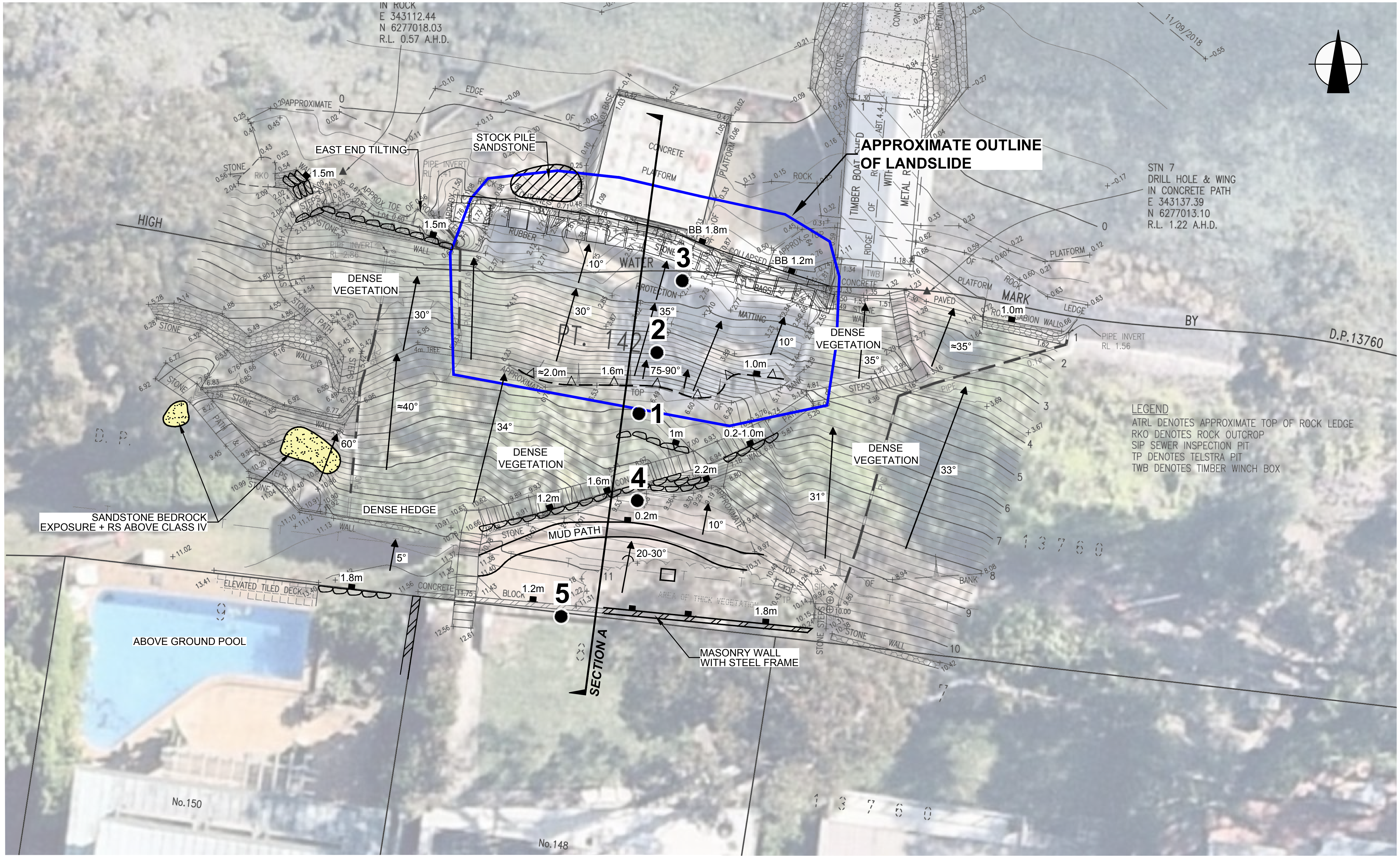


NOTES:

1. BB1.8m IS HEIGHT OF BULKA BAG WALL.
2. FOR SECTION A SEE FIGURE 3.
3. FOR EXPLANATION OF GEOTECHNICAL MAPPING SYMBOLS SEE FIGURE 4.

LEGEND

● BOREHOLES COMPLETED IN 2019.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

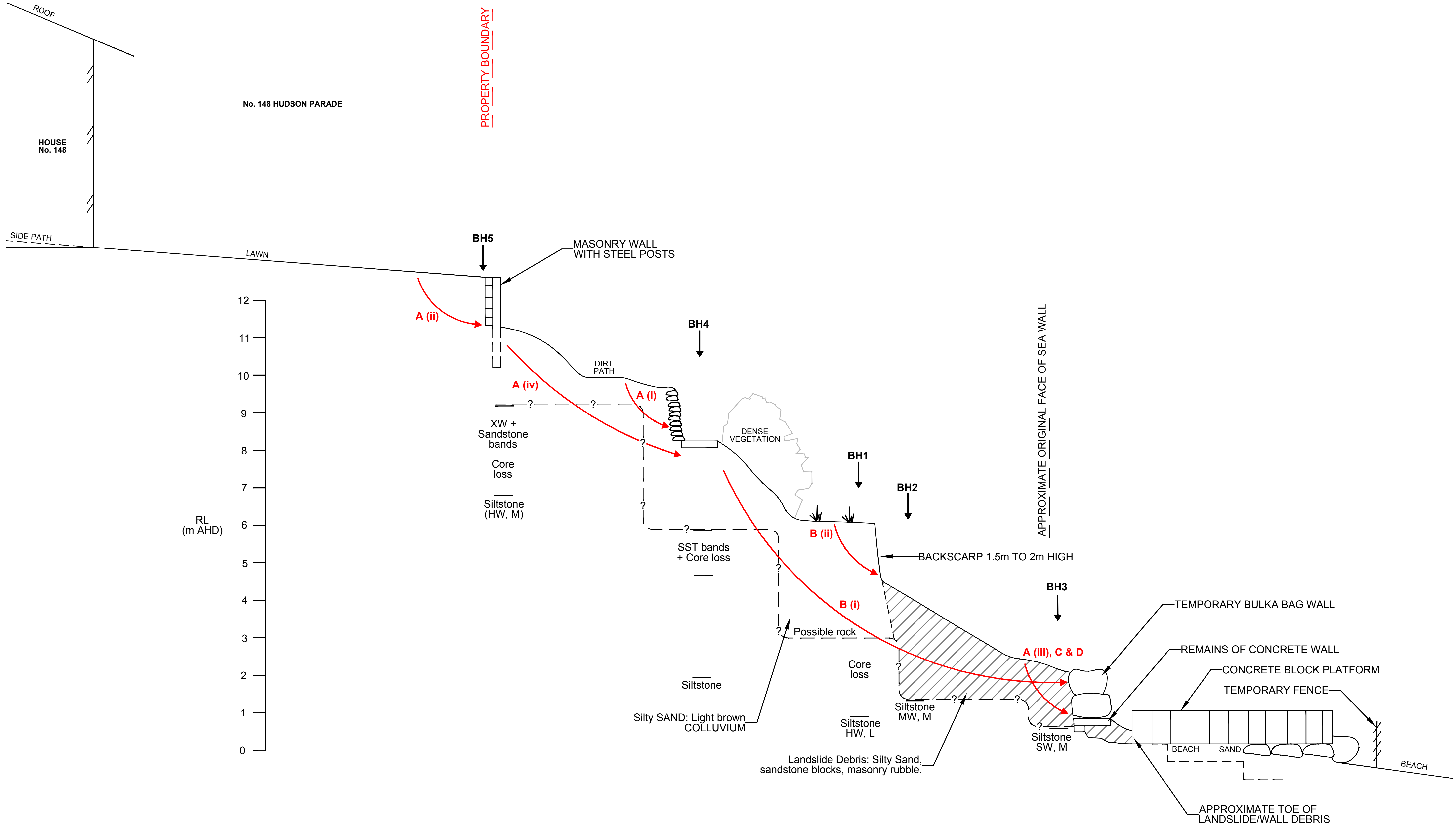
SCALE 1:150 @A3 METRES (0, 1.5, 3, 4.5, 6, 7.5)

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

Title:	GEOTECHNICAL SITE PLAN
Location:	FORESHORE AREA ADJACENT TO, 148 HUDSON PARADE, CLAREVILLE, NSW
Report No:	32115RMrpt2
Figure No:	2

JKGeotechnics

PLOT DATE: 22/03/2021 10:01:14 AM DWG FILE: Y:\32000'S\32115R CLAREVILLE\CAD\32115RMRPT2.DWG



LEGEND

A (i) GEOTCHNICAL HAZARD

SCALE 0 1 2 3 4 5 METRES 1:100 @A3

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

Title: **SECTION A LOOKING WEST**

Location: FORESHORE AREA ADJACENT TO, 148 HUDSON PARADE, CLAREVILLE, NSW

Report No: 32115RMrpt2

Figure No: 3

JKGeotechnics

PLOT DATE: 22/03/2021 2:30:53 PM DWG FILE: Y:\32000'S\32115R CLAREVILLE\CAD\32115RMRPT2.DWG

TOPOGRAPHY

Symbol *Ground Profile*

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

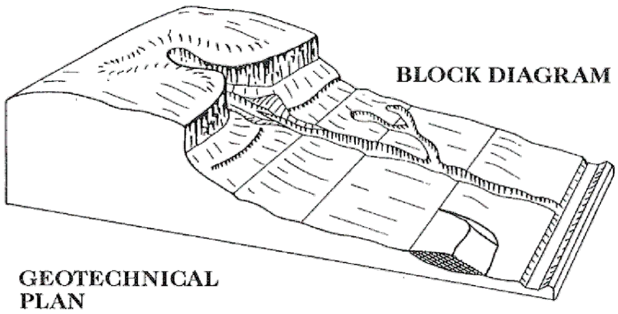
OTHER FEATURES

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

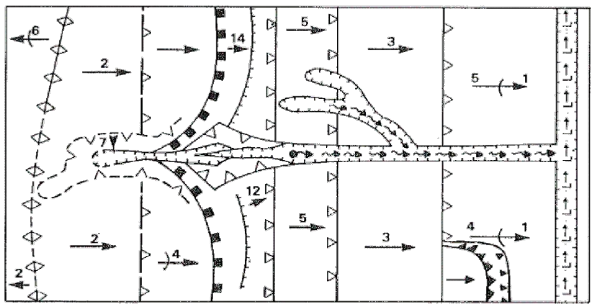
EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

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

BLOCK DIAGRAM



GEOTECHNICAL PLAN



PLOT DATE: 22/03/2021 10:01:30 AM DWG FILE: Y:\32000's\32115R CLAREVILLE\CAD\32115RMRPT2.DWG

Title:	GEOTECHNICAL MAPPING SYMBOLS
Location:	FORESHORE AREA ADJACENT TO, 148 HUDSON PARADE, CLAREVILLE, NSW
Report No: 32115Rrpt2	Figure No: 4

JKGeotechnics

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

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.

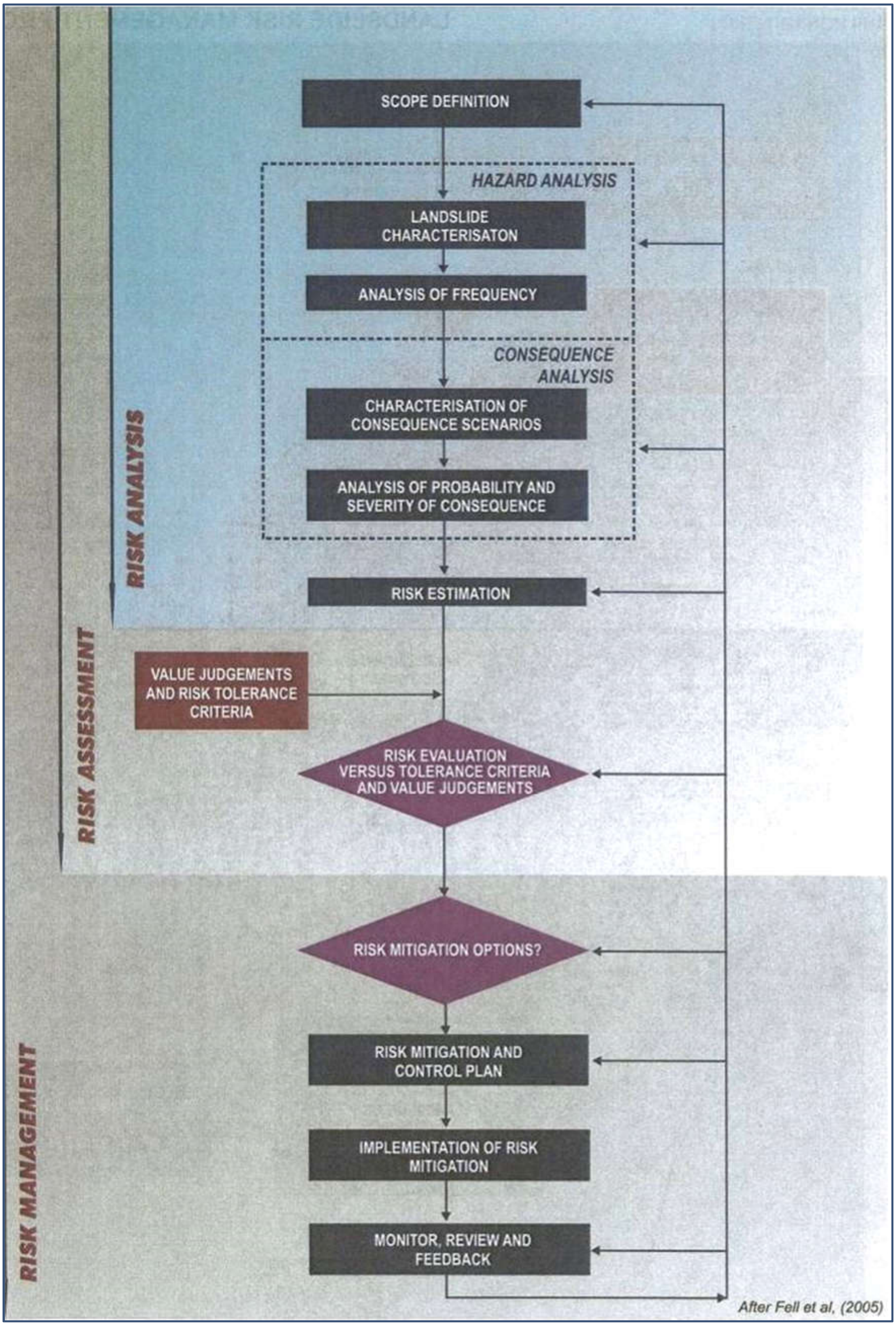


FIGURE A1: Flowchart for Landslide Risk Management.

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}		10 years		The event is expected to occur over the design life.	ALMOST CERTAIN	A
	5×10^{-2}		20 years			
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
	5×10^{-3}		200 years			
10^{-3}		1000 years		The event could occur under adverse conditions over the design life.	POSSIBLE	C
	5×10^{-4}		2000 years			
10^{-4}		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
	5×10^{-5}		20,000 years			
10^{-5}		100,000 years		The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
	5×10^{-2}		200,000 years			
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%		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
	100%			
60%		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
	40%			
20%		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
	10%			
5%		Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
	1%			
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.

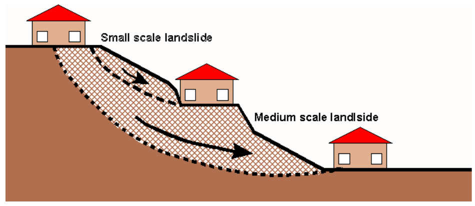


Figure 1

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.

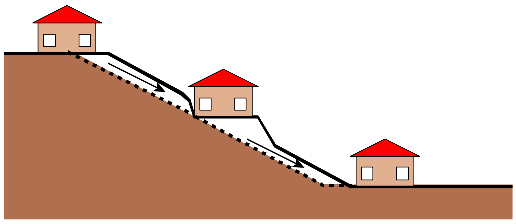


Figure 2

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.

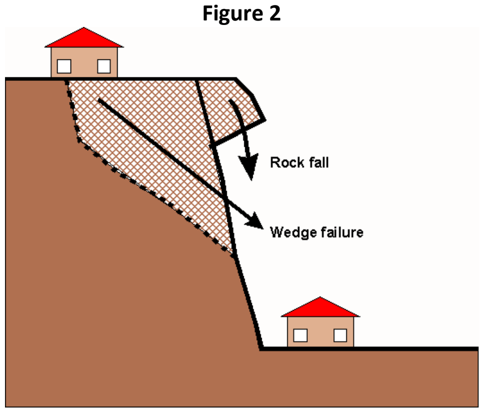


Figure 3

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.

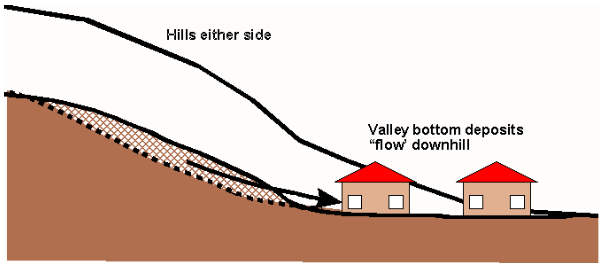


Figure 4

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:

- 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.

APPENDIX B

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Macquarie Park NSW 2113
Telephone: 02 9888 5000
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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Seawall Repairs
Location: 148 Hudson Parade, Clareville, NSW
Ref No: 32115R
Report: A
Report Date: 14/05/2019
Page 1 of 1

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	4.50 - 4.53	0.07	1
	5.87 - 5.90	0.2	4
2	3.58 - 3.61	0.2	4
	4.13 - 4.17	0.5	10
	4.96 - 5.00	1.0	20
	5.67 - 5.71	0.9	18
3	0.13 - 0.16	0.5	10
	0.72 - 0.76	0.7	14
	1.38 - 1.42	3.5	70
	2.22 - 2.25	1.3	26
	2.92 - 2.95	1.4	28
4	2.08 - 2.11	1.6	32
	3.64 - 3.67	0.7	14
	3.79 - 3.82	0.07	1
	3.92 - 3.95	0.7	14
	4.71 - 4.73	2.1	42
	6.12 - 6.15	1.0	20
	6.37 - 6.40	1.8	36
5	2.97 - 3.00	0.7	14
	3.95 - 3.98	0.09	2
	4.77 - 4.80	0.4	8
	4.97 - 5.00	0.7	14
	5.31 - 5.34	0.07	1

NOTES:

1. In the above table testing was completed in the Axial direction.
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 by the following approximate relationship and rounded off to the nearest whole number : U.C.S. = 20 IS (50)

JKGeotechnics

BOREHOLE LOG

Borehole No.
1

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R | **Method:** HAND AUGER/ WASHBORE | **R.L. Surface:** ~6.4 m
Date: 16/1/19 | **Datum:** AHD
Plant Type: | **Logged/Checked By:** W.S./P.R.

Groundwater Record	SAMPLES (ES / U50 / DB / 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							FILL: Silty sandy clay topsoil, low plasticity, dark brown, trace of root fibres.	w<PL			GRASS COVER
			6				FILL: Silty sandy clay, medium plasticity, red brown, dark grey and orange brown, fine to medium grained sand, fine to coarse grained sandstone gravel, trace of concrete fragments and sandstone boulders.				APPEARS POORLY COMPACTED
				1		CI	Inferred CLAY, with gravel to cobble size sandstone inclusions.				HAND AUGER REFUSAL AT 1.2m WASHBORING COMMENCE
			5			CI	Silty sandy CLAY: light grey and brown.	w<PL	(St - VSt)		SUBSURFACE PROFILE INFERRED FROM ADJACEMENT BOREHOLES AND PROGRESS OF WASHBORE EQUIPMENT
			4	2							
				3			REFER TO CORED BOREHOLE LOG				
			3	4							
			2	5							
			1	6							
			0								

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R | **Core Size:** TT56 | **R.L. Surface:** ~6.4 m
Date: 16/1/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** W.S./P.R.

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 $I_s(50)$	DEFECT SPACING (mm)	DEFECT DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
		4			START CORING AT 3.00m						
50% RETURN		3	3		SILTSTONE: light grey and dark grey, bedded at 8°.	HW	VL			(3.00m) XWS, 0°, 100 mm.t	Newport Formation
			4		NO CORE 1.18m						
		2			SILTSTONE: light grey and dark grey, bedded at 8°.	HW	VL			(4.45m) XWS, 0°, 130 mm.t	Newport Formation
			5		NO CORE 0.92m						
		1			SILTSTONE: dark grey, interbedded sandstone, fine to medium grained, light grey and orange brown, bedded at 15°.	HW	L			(5.55m) XWS, 0°, 100 mm.t	Newport Formation
		0	6		END OF BOREHOLE AT 6.00 m						
		-1	7								
		-2	8								

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH. Spacing scale: 600, 200, 60, 20.

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32115R CLAREVILLE.GPJ <<DrawingFile>> 24/10/2019 12:43 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


JK Geotechnics
JOB No. 32115R BH1 STARTING DEPTH AT 3.0m
3
NO CORE 1.2m
4
NO CORE 0.92m
5
6
End of BOREHOLE AT 6.0m.

JKGeotechnics

BOREHOLE LOG

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R **Method:** HAND AUGER **R.L. Surface:** ~4.6 m
Date: 16/1/19 **Datum:** AHD
Plant Type: **Logged/Checked By:** W.S./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, 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							FILL: Silty sandy clay topsoil, low plasticity, dark brown, trace of root fibres.	w<PL			GRASS COVER
			4				FILL: Silty sandy clay, medium plasticity, red brown, dark brown and orange brown, fine to medium grained sand, trace of fine to coarse grained sandstone gravel, and organic material.	w~PL			APPEARS POORLY COMPACTED TOO FRIABLE FOR HP TESTING
				1		CH	Silty sandy CLAY: high plasticity, light grey and orange brown, trace of fine to medium grained ironstone gravel.	w~PL	St	250 100 150	RESIDUAL
			3				as above, but light grey.		VSt		
			2	2				w<PL	Hd		TOO FRIABLE FOR HP TESTING
				3							
			1				REFER TO CORED BOREHOLE LOG				REFUSAL ON BEDROCK
				4							
			0								
				5							
			-1								
				6							
			-2								

JKGeotechnics

CORED BOREHOLE LOG

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R	**Core Size:** TT56	**R.L. Surface:** ~4.6 m
Date: 16/1/19	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: MELVELLE	**Bearing:** N/A	**Logged/Checked By:** W.S./P.R.

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 $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	SPACING (mm) 600 200 60 20	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
		2	3		START CORING AT 3.20m						
70% RETURN		1			NO CORE 0.06m					(3.27m) XWS, 10°, 20 mm.t	
					SANDSTONE: fine to medium grained, light grey and orange brown, bedded at 10°.	HW	L				Newport Formation
		0	4 5		SILTSTONE: dark grey, interbedded sandstone, fine to medium grained, light grey, bedded at 0-10°.	MW	M			(3.62m) Be, 10°, C, S, Clay Ct (3.68m) J, 20°, Un, S, Clay Ct (3.75m) J, 20°, Un, R, Fe Sn (3.85m) Be, 5°, P, S, Fe Sn (3.88m) Be, 10°, P, S, Fe Sn (3.97m) XWS, 5°, 30 mm.t (4.07m) Be, 5°, P, S, Fe Sn (4.78m) J, 12°, Un, R, Fe Sn (4.83m) J, 50°, C, Cn	Newport Formation
					NO CORE 0.10m					(5.14m) J, 80°, Un, R, Fe Sn	
		-1	6		SILTSTONE: dark grey, interbedded sandstone, fine to medium grained, light grey, bedded at 0-10°.	MW	M		60 20	(5.26m) Ji, 60°, Un, Cn (5.28m) J, 25°, Un, R, Fe Sn (5.40m) J, 50°, C, R, Fe Sn	Newport Formation
		-2 -3 -4	7 8		END OF BOREHOLE AT 6.10 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No 32115R BH2 STARTING DEPTH 3·2m
3
NO CORE
4
5
NO CORE 0·10m
6
END OF BOREHOLE AT 6·1m

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 3

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R | **Core Size:** TT56 | **R.L. Surface:** ~0.6 m
Date: 17/1/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** W.S./P.R.

CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components

START CORING AT 0.00m

Water Loss\Level	Barrel Lift	Depth (m)	Core Description	Weathering	Strength	Formation
ON COMPLETION OF CORING; 100% RETURN		0.00–2.20	SILTSTONE: dark grey, with interbedded sandstone, fine to medium grained, light grey, bedded at 0-10°.	SW	M (0.00–1.30); VH (1.30–2.20)	Newport Formation
		2.20–3.12	SANDSTONE: fine to medium grained, light grey.	SW	H	Newport Formation
		3.12	END OF BOREHOLE AT 3.12 m			

POINT LOAD STRENGTH INDEX $I_s(50)$: VL –0.1 L –0.3 M –1 H –3 VH –10 EH

SPACING (mm): 600 200 60 20

DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness

Specific:

- (0.32m) Be, 5°, Un, R, Fe Sn
- (0.38m) J, 18°, Un, R, Fe Sn
- (0.56m) J, 20°, Un, R, Fe Sn
- (0.59m) J, 20°, St, R, Fe Sn
- (0.65m) J, 67°, P, S, Cn
- (0.70m) Be, 5°, Un, R, Cn
- (0.71m) J, 90°, Un, Cn
- (0.84m) Be, 10°, P, S, Clay Ct
- (0.90m) J, 65°, P, R, Fe Sn
- (0.98m) J, 45°, C, Cn
- (1.12m) Be, 10°, P, S, Fe Sn
- (1.20m) J, 90°, Un, R, Cn
- (1.28m) XWS, 5°, 40 mm.t
- (1.36m) XWS, 5°, 10 mm.t
- (1.40m) J, 53°, P, S, Cn
- (1.60m) J, 25°, Un, R, Fe Sn
- (1.82m) J, 60°, Un, R, Fe Sn
- (1.95m) J, 68°, Un, R, FS Cn
- (2.54m) J, 90°, P, R, Fe Sn
- (2.85m) Be, 8°, Un, R, Clay Ct

General:

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
JOB No. 32115R BH3 STARTING DEPTH AT 0.0m
0
1
2
3
END OF BOREHOLE AT 3.12m

JKGeotechnics

BOREHOLE LOG

Borehole No. 4

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R
Method: DIATUBE/ HAND AUGER / WASHBORE
R.L. Surface: 8.1 m
Date: 9/5/19
Datum: AHD
Plant Type:
Logged/Checked By: J.L./P.R.

Groundwater Record	SAMPLES (ES, U50, DB, 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			8	0			CONCRETE: 80mm.t				NO OBSERVED REINFORCEMENT
						-	SANDSTONE BLOCKS 320mm.t				
	ES, DS					-	FILL: Sand, fine to medium grained, light brown, trace of silt fines.	M			
						CL	Sandy CLAY: low plasticity, light orange brown, trace of ironstone gravel.	w~PL	F	90 80 80	RESIDUAL
	ES		7	1		-	Inferred sandy CLAY: as above		(F)		HP TESTING ON REMOULDED SAMPLE
											COMMENCE WASHBORING
			6	2			REFER TO CORED BOREHOLE LOG				
			5	3							
			4	4							
			3	5							
			2	6							

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CORED BOREHOLE LOG

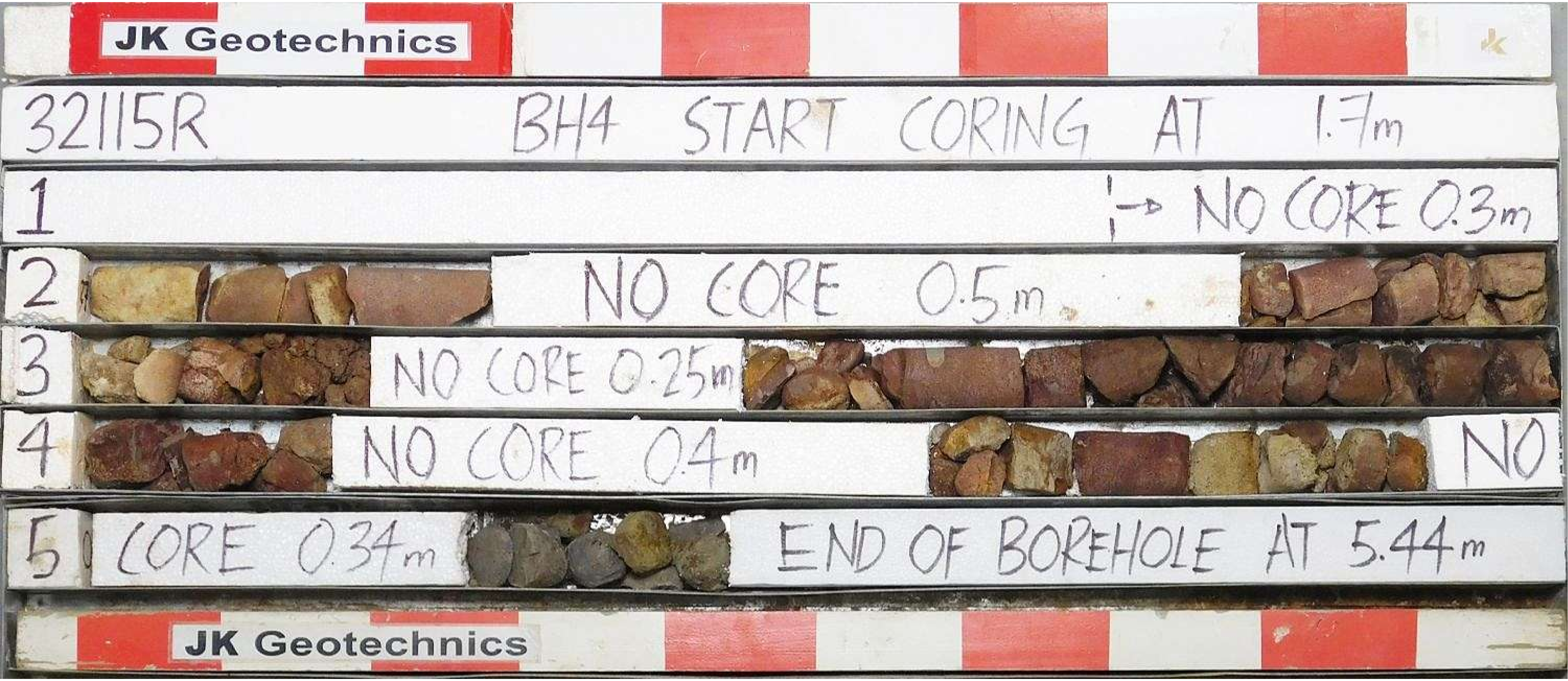
Borehole No.
4

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R | **Core Size:** TT56 | **R.L. Surface:** 8.1 m
Date: 9/5/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** J.L./P.R.

Water Loss\Level	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	Point Load Strength Index $I_s(50)$	Defect Details – Description (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) – Specific	Formation
		START CORING AT 1.70m					
50% RETURN	1.70	NO CORE 0.30m					Hawkesbury Sandstone
50% RETURN	2.00	SANDSTONE: fine to medium grained, brown, purple and orange, bedded sub horizontally.	MW	H	1.6	(2.04m) J, 60°, Un, R, Fe Sn (2.22m) J, 90°, Un, R, Fe Sn	Hawkesbury Sandstone
80% RETURN		NO CORE 0.50m					Hawkesbury Sandstone
80% RETURN		SANDSTONE: fine to medium grained, brown, purple and orange, bedded sub horizontally.	HW	L		(2.82m) J, 80°, Un, R, Fe Sn (3.08m) J, 70 - 90°, Un, R, Fe Sn (3.17m) XWS, 0°, 60 mm.t	
80% RETURN		NO CORE 0.25m					Hawkesbury Sandstone
80% RETURN		Interbedded SANDSTONE and SILTSTONE: fine to medium grained, brown, purple, orange and light grey, bedded at 0-30°.	MW	VL - M	0.70 0.070 0.70	(3.70m) J, 40°, Un, R, Fe Sn (3.73m) J, 40°, Un, R, Fe Sn (3.75m) Be, 20°, Cn (3.90m) Be, 0 - 20°, Cn (4.00m) J, 80 - 90°, Cn (4.14m) J, 60°, Cn	Hawkesbury Sandstone
80% RETURN		NO CORE 0.40m					
80% RETURN		SANDSTONE: fine to medium grained, brown, purple and light grey, bedded sub horizontally.	MW	H	2.1	(4.66m) J, 50°, Cn (4.78m) J, 90°, Cn (4.82m) CS, 0 - 30°, 40 mm.t (4.90m) CS, 0°, 20 mm.t	
80% RETURN	5.00	NO CORE 0.34m					Newport Formation
80% RETURN		SILTSTONE: light grey, with dark grey lamination.	HW	VL		(5.42m) CS, 0°, 50 mm.t	Newport Formation
	5.44	END OF BOREHOLE AT 5.44 m					

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
32115R
BH4 START CORING AT 1.7m
1
NO CORE 0.3m
2
NO CORE 0.5m
3
NO CORE 0.25m
4
NO CORE 0.4m
NO
5
CORE 0.34m
END OF BOREHOLE AT 5.44m
JK Geotechnics

JKGeotechnics

BOREHOLE LOG

Borehole No. 5

1 / 2

Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R **Method:** HAND AUGER/ WASHBORE **R.L. Surface:** 11.2 m
Date: 9/5/19 **Datum:** AHD
Plant Type: **Logged/Checked By:** J.L./P.R.

Groundwater Record	SAMPLES (ES / U50 / DB / 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			11	0			FILL: Silty clay topsoil, low plasticity, dark brown, with fine to medium grained sand and root fibres.	w<PL			APPEARS POORLY COMPACTED
						CH	Silty CLAY: high plasticity, orange brown, trace of fine to medium grained sand, and ironstone gravel.	w~PL	St	170 190 170	RESIDUAL HP TESTING ON REMOULDED SAMPLE
			10	1					St - VSt	210 280 180	
							as above, but orange brown and light grey, with fine to medium grained sand.				
						CL-CI	Sandy CLAY: low to medium plasticity, light grey and orange brown, fine to medium grained sand.		F - St	120 90 160	COMMECE WASHBORING AT 1.5m NATURAL
				2			(inferred as above)				
			9				REFER TO CORED BOREHOLE LOG				
			8	3							
			7	4							
			6	5							
			5	6							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
5

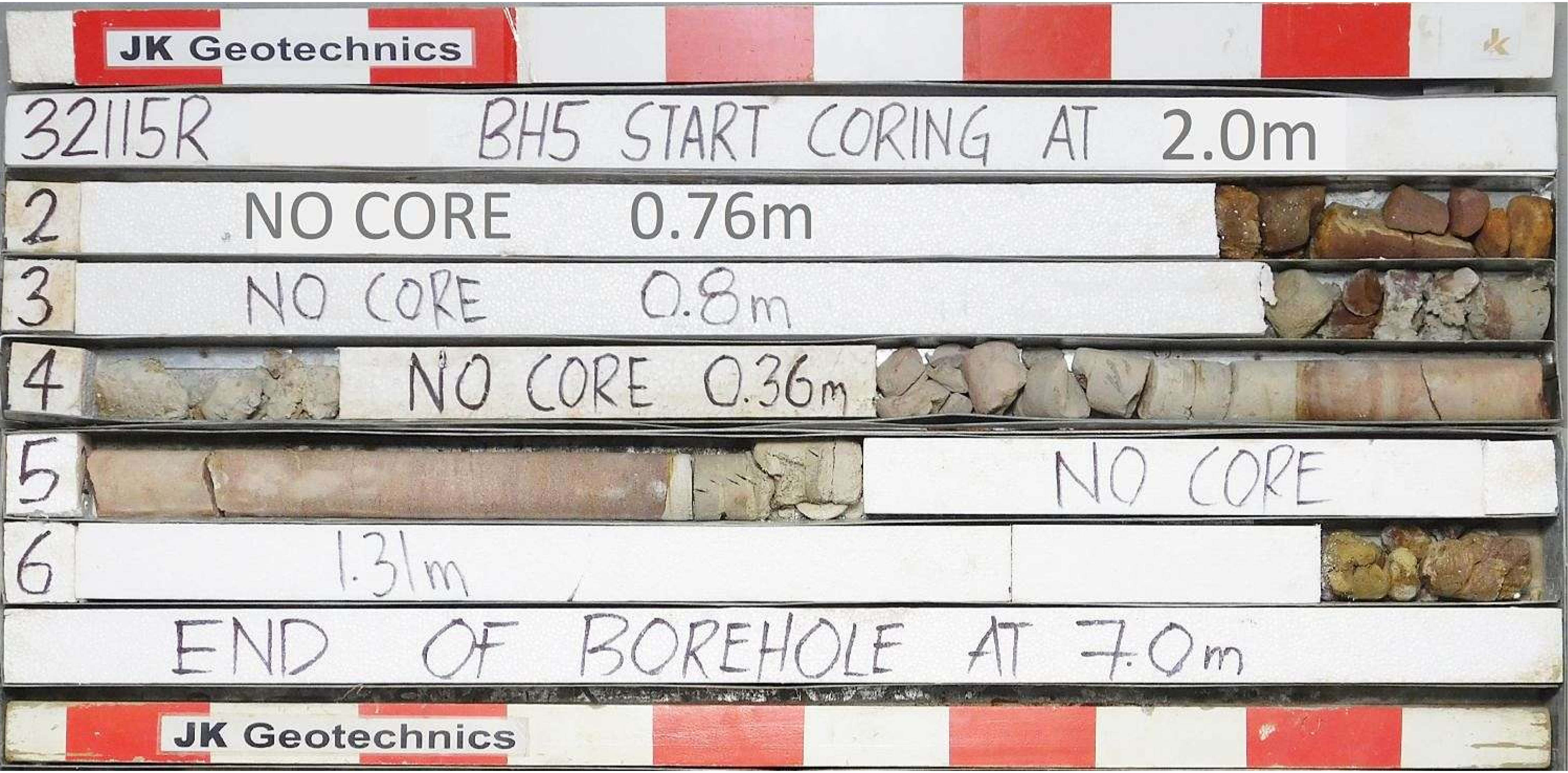
Client: ROYAL HASKONINGDHV
Project: PROPOSED SEAWALL REPAIRS
Location: 148 HUDSON PARADE, CLAREVILLE, NSW

Job No.: 32115R | **Core Size:** TT56 | **R.L. Surface:** 11.2 m
Date: 9/5/19 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** J.L./P.R.

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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness – Specific	General	Formation
		10			START CORING AT 2.00m							
70% RETURN		9	2		NO CORE 0.76m							
					SANDSTONE: fine to medium grained, red orange brown and light grey.	MW	M	0.70		(2.76m) XWS, 0°, 90 mm.t (2.80m) Be, 0°, P, R, Fe Sn (2.90m) J, 80°, P, R, Fe Sn		Newport Formation
		8	3		NO CORE 0.80m							
			4		SANDSTONE: fine to medium grained, light grey and red orange brown.	HW	VL - L	0.090		(3.98m) XWS, 0°, 140 mm.t		
		7			NO CORE 0.37m				600 200 60 20			
					Extremely Weathered sandstone: silty CLAY, low plasticity, light gey and red brown, bedded at 0-20°.	w<PL	Hd					Newport Formation
		6	5		SANDSTONE: fine to medium grained, light grey and red brown, bedded at 0-20°, occasional VL strength bands.	HW	M	0.40 0.70 0.070		(4.83m) Be, 0°, P, Fe Sn (5.47m) CS, 0°, 90 mm.t		
		5	6		NO CORE 1.31m							
			7		Interbedded SANDSTONE and SILTSTONE: fine to medium grained, red brown and orange.	HW	L			(6.84m) CS, 0 - 40°, 50 mm.t		Newport Formation
		4			END OF BOREHOLE AT 7.00 m				600 200 60 20			

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 32115R CLAREVILLE.GPJ <<DrawingFile>> 24/10/2019 12:43 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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
32115R
BH5 START CORING AT 2.0m
2
NO CORE 0.76m
3
NO CORE 0.8m
4
NO CORE 0.36m
5
NO CORE
6
1.31m
END OF BOREHOLE AT 7.0m
JK Geotechnics

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV
Project:	PROPOSED SEAWALL REPAIRS
Location:	148 HUDSON PARADE, CLAREVILLE, NSW

Job No.	32115R	Hammer Weight & Drop: 9kg/510mm
Date:	16-1-19	Rod Diameter: 16mm
Tested By:	W.S.	Point Diameter: 20mm

Test Location	**1**	**2**		Test Location	**1**	**2**	
Surface RL	≈6.42m	≈4.62m		Surface RL	≈6.42m	≈4.62m	
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1		3000-3100	10/50mm	11	
100 - 200	4	1		3100-3200	REFUSAL	16	
200 - 300	5	2		3200-3300		11/50mm	
300 - 400	2	1		3300-3400		REFUSAL	
400 - 500	2			3400-3500			
500 - 600	4			3500-3600			
600 - 700	3			3600-3700			
700 - 800	8			3700-3800			
800 - 900	12	2		3800-3900			
900 - 1000	7			3900-4000			
1000 - 1100	5	1		4000-4100			
1100 - 1200	2	2		4100-4200			
1200 - 1300	2	2		4200-4300			
1300 - 1400	3	2		4300-4400			
1400 - 1500	4	1		4400-4500			
1500 - 1600	5	6		4500-4600			
1600 - 1700	3	17		4600-4700			
1700 - 1800	4	6		4700-4800			
1800 - 1900	3	8		4800-4900			
1900 - 2000	3	7		4900-5000			
2000 - 2100	8	10		5000-5100			
2100 - 2200	7	8		5100-5200			
2200 - 2300	8	13		5200-5300			
2300 - 2400	8	3		5300-5400			
2400 - 2500	3	15		5400-5500			
2500 - 2600	5	16		5500-5600			
2600 - 2700	18	24		5600-5700			
2700 - 2800	15	26		5700-5800			
2800 - 2900	18	20		5800-5900			
2900 - 3000	15	16		5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	ROYAL HASKONINGDHV
Project:	PROPOSED SEAWALL REPAIRS
Location:	148 HUDSON PARADE, CLAREVILLE, NSW

Job No.	32115R	Hammer Weight & Drop: 9kg/510mm	
Date:	9-5-19	Rod Diameter: 16mm	
Tested By:	J.L.	Point Diameter: 20mm	

Test Location	4	5		Test Location			
Surface RL	≈8.1m	≈11.2m		Surface RL			
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	CORED	1		3000-3100			
100 - 200		1		3100-3200			
200 - 300		1		3200-3300			
300 - 400		1		3300-3400			
400 - 500				3400-3500			
500 - 600		2		3500-3600			
600 - 700	11	2		3600-3700			
700 - 800	9	4		3700-3800			
800 - 900	4	2		3800-3900			
900 - 1000	6	2		3900-4000			
1000 - 1100	4	4		4000-4100			
1100 - 1200	3	5		4100-4200			
1200 - 1300	3	20		4200-4300			
1300 - 1400	3	11		4300-4400			
1400 - 1500	3	3		4400-4500			
1500 - 1600	3	14		4500-4600			
1600 - 1700	3	16		4600-4700			
1700 - 1800	13	14		4700-4800			
1800 - 1900	10	16		4800-4900			
1900 - 2000	8	30		4900-5000			
2000 - 2100	12	REFUSAL		5000-5100			
2100 - 2200	11			5100-5200			
2200 - 2300	15/180mm			5200-5300			
2300 - 2400	REFUSAL			5300-5400			
2400 - 2500				5400-5500			
2500 - 2600				5500-5600			
2600 - 2700				5600-5700			
2700 - 2800				5700-5800			
2800 - 2900				5800-5900			
2900 - 3000				5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

APPENDIX C

Date: 3 June 2021
Ref: 32115RMlet2 rev2

Northern Beaches Council

Attention: Charles Sawley
Email: Charles.Sawley@northernbeaches.nsw.gov.au

GEOTECHNICAL DESIGN
STABILISATION OF LANDSLIDE
SHORELINE FRONTING 148 HUDSON PARADE, CLAREVILLE, NSW

1 Introduction

This letter reports the results of our numerical analyses completed to assist with the detailed design of the permanent coastal protection works at the above site. Based on the results of our analyses and liaison with the project coastal engineers (Royal HaskoningDHV [RHDHV]) we have also prepared a specification for the geotechnical aspects of the coastal protection works.

We note that we have prepared a previous repot (Ref. 32115RMlet) dated 31 March 2020 for temporary stabilisation measures to support the foreshore slope that had been impacted by the landslide that impacted the slope following an extreme rainfall event in February 2020. Northern Beaches Council (BNC) requested Douglas Partners Pty Ltd (DP) to complete a peer review of our initial temporary slope stabilisation design presented in our report dated 31 March 2020. We have reviewed the DP peer review presented in their report (Ref. 99787.00) dated 28 August 2020 and further comments are provided in Section 2 below.

To date, the temporary stabilisation works that have been completed at the site have comprised removal of some of the old sea wall and landslide debris, and installation of a bulka bag wall installed along the shale rock shelf, as described in our Site Report 2. This work was completed under the direction of NBC and JK Geotechnics (JKG) and JKG have continued to undertake periodic site inspections (particularly after heavy rainfall events) to assess the stability of the site. The temporary stabilisation measures that are protecting the toe of the slope have been performing satisfactorily.

2 DP Peer Review

The principal conclusions of the DP Peer review report were:

1. DP agrees that the proposed rock bolt and steel mesh solution is appropriate for the site, in combination with construction of a new permanent seawall.
2. The JK stability analysis has adopted quite conservative shear strength properties for the weathered bedrock, but non-conservative assumptions about potential for saturation of the near surface soils.

3. DP considers that there is potential scope for reducing the rock bolt lengths in the upper three rows by undertaking a detailed back-analysis of the original landslide to obtain better estimates of the soil and rock properties.

In relation to item 2 we agree that the previously adopted shear strength parameters for the bedrock were quite conservative and we have re-assessed these parameters. However, we disagree that our assessment of saturation of the soil profile was non-conservative. Our opinion, based on our site inspections immediately following the landslide and the results of our previous investigations, was that it was very unlikely that groundwater would have fully saturated the slope and that the influence of water on the shear strength of the soil profile would have been predominantly influenced by uncontrolled discharge of stormwater pipes onto the slope and/or into the slope.

In relation to Item 3 we have completed a back analysis of the slope immediately prior to the landslide based on our best estimate of the subsurface profile, groundwater levels and the most likely profile of the slip surface.

The results of our review of shear strength parameters, groundwater levels, soil saturation and back analysis are presented in Section 3 below and the results of our analyses to inform the detailed design are presented in the following Section 4. A specification for the geotechnical aspects of the coastal protection works, based on the results of our analyses is presented in the attached Appendix A.

3 SLOPE/W NUMERICAL ANALYSES

3.1 Review of Previous SLOPE/W Analysis

Using the Geostudio SLOPE/W software we reviewed our previous analysis as follows:

- Undertaking a critical review of the bedrock shear strength parameters.
- Completing a back analysis.
- Reviewing a likely groundwater level for detailed design.

The analysis section adopted is the same as out previous analyses (see Figure 1).

The geotechnical parameters adopted for the SLOPE/W analyses presented in our reported dated 31 March 2020 are provided in the table below.

Geotechnical Parameters Adopted for Slope/W Analysis March 2020			
Unit	**Unit Weight (kN/m^3)**	**Cohesion (c) (kPa)**	**Internal Angle of Friction (ϕ)**
Residual Clay	18	3	28°
Existing Fill	16	0	26°
Class V Sandstone	20	5	32°
Newport Formation	20	5	35°

We have completed a review of the bedrock shear strength parameters. As part of the review we refined our parameters to be consistent with classification of the bedrock presented in Section 5 of our report (Ref. 32115Rrpt Rev1) dated 31 January 2020 such that:

- The upper portion of the bedrock profile comprised Class V sandstone overlying Class V siltstone ('shale'), and
- The lower portion comprising Newport Formation (Class IV or better siltstone ['shale']) which is also exposed along the foreshore bedrock platform.

The back analysis was completed based on our understanding of the likely subsurface, topographical profiles at the time of the landslide and our observations of the landslip immediately following the event. We therefore assumed:

- A specified slip surface which extended immediately below the existing seawall and impacted the soil profile landward of the seawall.
- The soils were saturated such that the bulk unit weight was at a maximum level. This represents the impact of on-going discharge of water over and into the sloe by surface run-off and uncontrolled discharge of stormwater pipes.
- The groundwater level was adopted at just above the bedrock surface. In our opinion, considering our previous investigation results and the nature of the generally cohesive nature of the soil profile a groundwater level at surface level was unrealistic.

We note that our previous analyses considered two groundwater profiles:

- At RL 0m AHD (approximate high tide level), which was considered reasonable given the likely drained conditions of the hillside.
- Along the inferred stepped rock surface profile, a 'worst case' situation which was considered to have been relevant to initiating the landslip although the source of water may well have been water discharging from stormwater pipes into the hillside.

For the back analysis the revised bedrock shear strength parameters were adopted together with the saturated bulk unit weights for the soils and the groundwater level just above the bedrock surface. The shear strength parameters of the fill and residual clay soils were the adjusted in order to obtain a Factor of Safety (FOS) at, or just below 1. Figure 2 presents the geotechnical model adopted for the back analysis and Figure 3 presents the output from SLOPE/W for the back analysis.

The revised shear strength parameters derived from our review and back analysis are provided in the table below.

Revised Geotechnical Parameters Following Review and Back Analysis			
Unit	**Saturated Unit Weight (kN/m^3)**	**Cohesion (c) (kPa)**	**Internal Angle of Friction (ϕ)**
Residual Clay	19	6.5	28°
Debris	17	1	26°
Class V Sandstone	20	20	30°
Class V Siltstone	20	10	30°
Newport Formation	20	30	35°

3.2 SLOPE/W Analysis of Stabilisation Measures

Using the Geostudio SLOPE/W software and the results of our back analyses, we undertook a review of the initial form of stabilisation measures, which included several rows of permanent rock bolts and a gabion basket seawall at the toe of the slope. The review and analyses indicated that four rows of permanent rock bolts were required to achieve a FOS of 1.5 or more. Figure 4 indicates that a minimum FOS of 1.7 was achieved.

The analysis adopted the groundwater level at the soil-bedrock interface. We carried out a sensitivity analysis of the model with regard to an elevated groundwater level at a maximum of about 2.0m above the bedrock surface and sloping down to the foreshore; a minimum FOS of 1.45 was obtained. We regard this groundwater level as being unrealistic but represents a guide to the sensitivity of the design to an elevated groundwater level.

RHDHV raised concerns regarding the size of the gabion seawall to support the slope, in particular:

- The potential for the required wall width to encroach seaward into the foreshore area,
- The potential landward extent of temporary excavations into the toe of the landslide, an
- The consequent implications for constructability in the foreshore environment whilst maintaining a safe work environment.

Several variations on the form of the permanent stabilisation measure initially outlined in our report dated 31 March 2020 were considered based on liaison between JKG, RHDHV and NBC. Following discussion on 10 September 2020 between NBC, RHDHV and JKG, a preferred alternative seawall concept was discussed and agreed. The alternative concept adopted the permanent rock bolts to support the slope, in conjunction with steel mesh to control near surface soil erosion over the slope face and included a permanent reinforced shotcrete face anchored in place by rock bolts to support the toe of the slope. A gabion wall would then be constructed to cover the seaward face of the shotcrete seawall in order to provide protection from waves and a more suitable aesthetic appearance which blended with adjacent existing gabion walls supporting the toes of the neighbouring portions of the foreshore slopes. This alternative design required additional analysis using finite element software in order to determine structural actions such as bending moments and shear forces acting on the shotcrete seawall to assist RHDHV in their detailed design.

4 NUMERICAL ANALYSES FOR DETAILED DESIGN

4.1 Proposed Stabilisation Measures

The agreed form of the stabilisation measures was:

- Four rows of rock bolts spaced evenly between RL8.7m and RL2.4m comprising fully grouted 25mm diameter fully threaded Glass-Fibre Reinforced Plastic bolts (GRP60).
- Composite high tensile steel mesh (such as Macaferri Steelgrid® HR PVC with hexagonal mesh) draped over the soil slope surface engaged with the rock bolts. Beneath the mesh hessian fabric or similar (such as Maccaferri MacMat HS) placed to prevent erosion and promote vegetation growth. Complemented by establishing suitable native vegetation species planted through the mesh.
- A seawall formed at the toe of the slope comprising a reinforced shotcrete face engaged with the fourth row of rock bolts (RL2.4m) and provided with a steeply inclined permanent toe rock bolts installed at RL1.0m. The crest of the shotcrete seawall will be formed at RL3m to reduce wave overtopping and a reno mattress will be placed over the slope surface immediately landward of the seawall crest to control erosion from overtopping events. A gabion wall will also be placed on the foreshore bedrock platform immediately seaward of the shotcrete seawall.

4.2 Methodology

The finite element approach has been used to carry out the slope stability analysis. PLAXIS 2D, a two-dimensional (2D) finite element (FE) computer program was used to complete the numerical analysis.

The geotechnical parameters for the subsurface materials were selected based on empirical correlations well established in geotechnical engineering and our back analysis, as discussed in Section 3 above. In our selection of parameters, consideration was given to the inherent uncertainty associated with natural, non-engineered materials such as variations in rock strength, cross bedding, anisotropy, etc. In this regard, in some instances conservative geotechnical parameters have been adopted.

It was assumed that the groundwater level followed the interface between soil and bedrock.

Staged numerical modelling was completed to simulate the slope and the construction of the stabilisation measures.

4.3 Model Geometry and Applied Loads

The geometry of the slope and reinforcement system are presented in Figure 5 and the analysis section is at a similar location as for the SLOPE/W analysis (see Figure 1).

The applied surcharges and loads in the analysis are presented below:

- Construction surcharge load of 5 kPa on the slope benches.
- Wave loads based on the information provided by RHDHV; for ease of modelling we adopted a more conservative upper uniform pressure distribution of 8kPa and a lower uniform pressure distribution of 12kPa.

4.4 Model Parameters and Stages

4.4.1 Geotechnical Parameters

The Mohr-Coulomb model was used to simulate the behaviour of the subsurface profile. The adopted parameters in the numerical analysis are presented in the table below.

Geotechnical Parameters					
Unit	Saturated Unit Weight (kN/m^3)	Cohesion (c) (kPa)	Internal Angle of Friction (ϕ)	Young's Modulus (E) (MPa)	Poisson's Ratio
Residual Clay	19	6.5	28°	20	0.3
Debris	17	1	26°	10	0.3
Class V Sandstone	20	20	30°	80	0.3
Class V Siltstone	20	10	30°	70	0.3
Newport Formation	20	30	35°	100	0.3

4.4.2 Structural Properties

The shotcrete wall and the high resistance Geocomposite mesh system have been modelled as plate elements. The rock bolt and grout body have been modelled using node to node anchor and embedded beam row elements, respectively. The adopted structural properties in the numerical analysis are presented in the table below.

Structural Element	Thickness/Diameter (mm)	EI (kNm^2)	EA (kN)	Poisson's Ratio
Shotcrete	300	4.5×10^4	6.0×10^6	0.2
SteelGrid HR[1]	-	1.3×10^{-2}	2.87×10^4	0.2
Rock Bolt-GRP60	25	-	2.08×10^4	-
Grout body	60	15.8	7.04×10^4	-

[1] High resistance Geocomposite mesh system

The adopted configurations for the rock bolts are presented in the table below.

Row No.	Head RL (m)	Inclination (degrees)	Horizontal Spacing (m)	Indicative Total Length (m)	Minimum Bond Length in Rock (m)
1	8.7	25	3.0	8.0	3.0
2	6.4	25	3.0	6.0	3.0
3	4.4	25	3.0	6.0	3.0
4	2.4	25	3.0	6.0	3.0
5 (Toe Bolt)	1.0	45	3.0	3.0	3.0

4.4.3 Model Stages

The numerical model was run through a number of stages to simulate the construction procedure. These are summarised below:

1. Initial phase to generate the initial stresses in the soil and rock mass;
2. Post-failure unreinforced slope;
3. Installation of top bolts (i.e. rows 1 to 3) and installation of high resistance Geocomposite mesh system (i.e. SteelGrid HR system) on the slope surface above the shotcrete wall;
4. Excavation to RL 2.0m, install 4th row of rock bolts and install shotcrete wall;
5. Excavation to RL 0.5m (foreshore bedrock surface) and install toe bolt at RL 1.0m and install shotcrete wall to the foreshore bedrock surface;
6. Apply wave loads on the shotcrete wall;
7. Apply 5kPa surcharge on the slope benches;
8. Construction of 0.5m width gabion wall; and
9. Construction of reno rock mattress (0.5m thick) landward of the seawall crest.

With regard to stages 4 and 5, to avoid localised failure of the temporary cut face in the model, the shotcrete and bolts were installed contemporaneously. We warned in our previous report that the construction staging (both shore parallel and shore normal) for the seawall would need to be sequential in order to maintain stability. Further comments are provided in Section 5 below. This issue of localised failures indicated by the model represents a limitation of the 2D FE modelling which could only otherwise be overcome by undertaking 3D FE modelling which was beyond the scope of our commission.

4.5 Model results

The model output plots of structural actions for the shotcrete and SteelGrid HR[1] system are presented in Figures 6 to 11.

The maximum working forces of the rock bolts are presented in table below.

Rock Bolt Row Number	Rock Bolt Maximum Working Load (kN)
1	3.0
2	3.2
3	14.5
4	48.0
Toe Bolt	8.0

[1] High resistance Geocomposite mesh system

The FOS for different construction stages are presented in table below.

Stage Number	Factor of Safety
4	1.095
5	1.23
9	1.4

With regard to the FOS obtained in SLOPE/W and PLAXIS, we note the following:

- The SLOPE/W software employs limit equilibrium method for the slope stability analysis, which involves passing a slip surface through the soil mass and dividing the inscribed section into vertical slices. For the assumed slip surface, the static equilibrium equations are used to calculate the FOS and stresses for each slice employing the Morgenstern-Price methodology.
- The FOS derived from the PLAXIS analysis is computed using a Phi-c reduction technique, which reduces the soil parameters (i.e. friction angle and cohesion) until the soil collapses.
- Based on the available literature, for a similar analysis model, PLAXIS produces a FOS of about 18% lower than the corresponding SLOPE/W result (Khabbaz et al. 2012).

Based on the above, the equivalent SLOPE/W FOS for the above PLAXIS model stages 4, 5 and 9 are 1.3, 1.45 and 1.65, respectively. PLAXIS therefore indicates that with regard to global stability of the slope when supported by the proposed stabilisation measures, a FOS in excess of 1.5 is achieved.

4.6 Additional Comments

PLAXIS 2D has been used to assess the stability of shoring walls for the proposed development. Whilst efforts have been made to check the reasonableness of the reported results, the simulation of geotechnical problems by means of the finite element method implicitly involves some inevitable numerical approximations. Consequently, while results have been calculated to several decimal places, it is improbable that their accuracy is to this order but allows comparison of the effects of the various stages of development used in the models.

The modelling has been based on information available to us, which has been checked for accuracy to the extent reasonably possible. If additional information becomes available at any stage during the project which appears in conflict with current assumptions then we should immediately be notified and asked to review our analysis.

5 CONSTRUCTION CONSIDERATIONS

The landslip area is currently marginally stable. To construct the permanent seawall along the landslide toe will require removal of the toe material which, if not completed with due care and consideration, has the potential to cause further landslide movement. Further landslide movement would pose a danger to site personnel and could potentially impact the upslope buried services and pathway.

In order to provide a 'safe' work environment for construction of the permanent seawall and to improve stability the proposed stabilisation measures include:

- Stabilisation of the landslip using a grid of permanent rock bolts bonded into the underlying weathered bedrock. The rock bolts will support a layer of high tensile steel mesh, and to assist in controlling soil erosion a layer of hessian/geofabric will also be provided under the mesh. The rock bolts and mesh will be sequentially constructed starting at the top of the slope.
- The permanent rock bolts will then allow construction of the shotcrete wall in two vertical 'lifts' and in a sequential manner laterally along the foreshore. The staged and sequential sequencing will reduce the amount of exposed and unsupported toe during the works and will need to consider tidal 'windows' which allow construction activities to take place.

The rock bolts comprise lightweight glass-fibre reinforced plastic for ease of handling on the steep slope.

The nominated 60mm diameter drill holes for the rock bolts allows for them to readily installed using rope access techniques or boom-lift mounted equipment.

The shotcrete seawall includes construction joints at 7.5m spacings. There will also be 'cold' joints at the interfaces between each panel of shotcrete as follows:

- The horizontal interface at RL1m between the two vertical 'lifts' of shotcrete.
- At lateral spacings of 3m to 4m representing the expected panel spacings within which rock bolts and shotcrete can be installed sequentially. The panels will need to be completed in a 'hit 1 miss 2' underpin style sequence to avoid large continuous sections of the tow of the scope remaining unsupported for short durations during the works. The actual spacings can be finalised in discussion with the Contractor based on their expected productivity in the tidal environment.

Regards

For and on behalf of
JK GEOTECHNICS

Matthew Pearce
Associate |Geotechnical Engineer

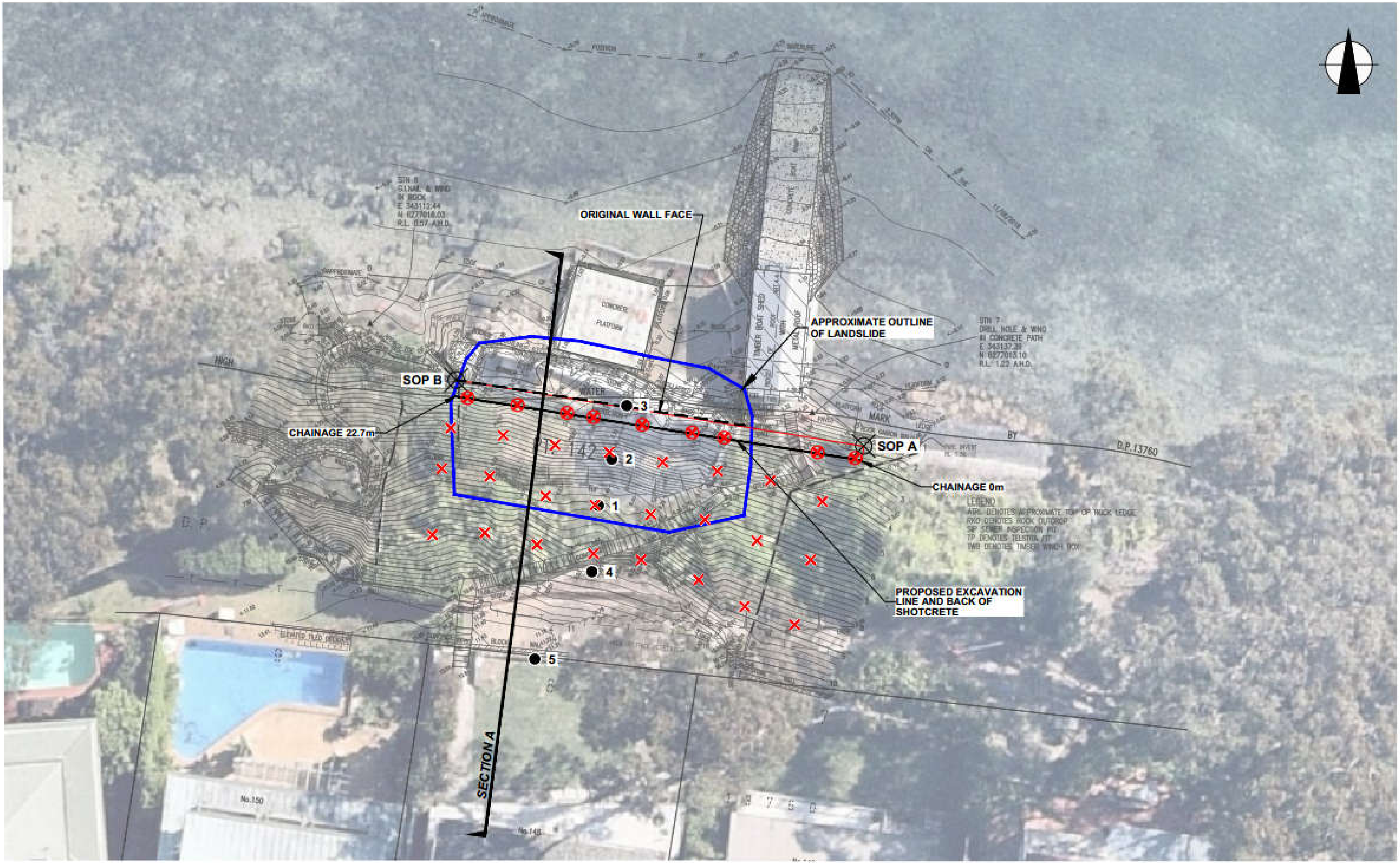
Reviewed By

Paul Roberts
Principal Associate | Engineering Geologist

Attachments

Figure 1: Location of Adopted Section in Analysis
Figure 2: Adopted Geotechnical Model in SLOPE/W Back Analysis
Figure 3: Results of SLOPE/W Back Analysis
Figure 4: Results of SLOPE/W Analysis for Reinforced Slope Adopting Revised Shear Strength Parameters
Figure 5: Adopted Geotechnical Model PLAXIS 2D Analysis
Figure 6: Envelope of Axial Forces in Shotcrete
Figure 7: Envelope of Shear Forces in Shotcrete
Figure 8: Envelope of Bending Moments in Shotcrete
Figure 9: Envelope of Axial Forces in SteelGrid
Figure 10: Envelope of Shear Forces in SteelGrid
Figure 11: Envelope of Bending Moments in SteelGrid

Appendix A Specification for Permanent Rock Bolts, Shotcrete Seawall and Steel Mesh Facing



LEGEND

- BOREHOLE
- ROW 5 RL1.0m
  ROW 4 RL2.4m
- ROW 3 RL4.4m
- ROW 2 RL6.4m OR ABOVE STEP AT REAR OF SLIP
- ROW 1 RL8.7m

NOTES:
1. ACTUAL BOLT PLACEMENT DEPENDANT ON LOCALISED GEOMETRY & TO BE SET OUT BY GEOTECHNICAL ENGINEER.
2. ROW 4+5 BASED ON ELEVATIONS @ CHAINAGES: 0.6m, 2.5m, 8.0m, 9.5m, 12.25m, 15.0m, 16.5m, 19.25m AND 22.0m.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

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

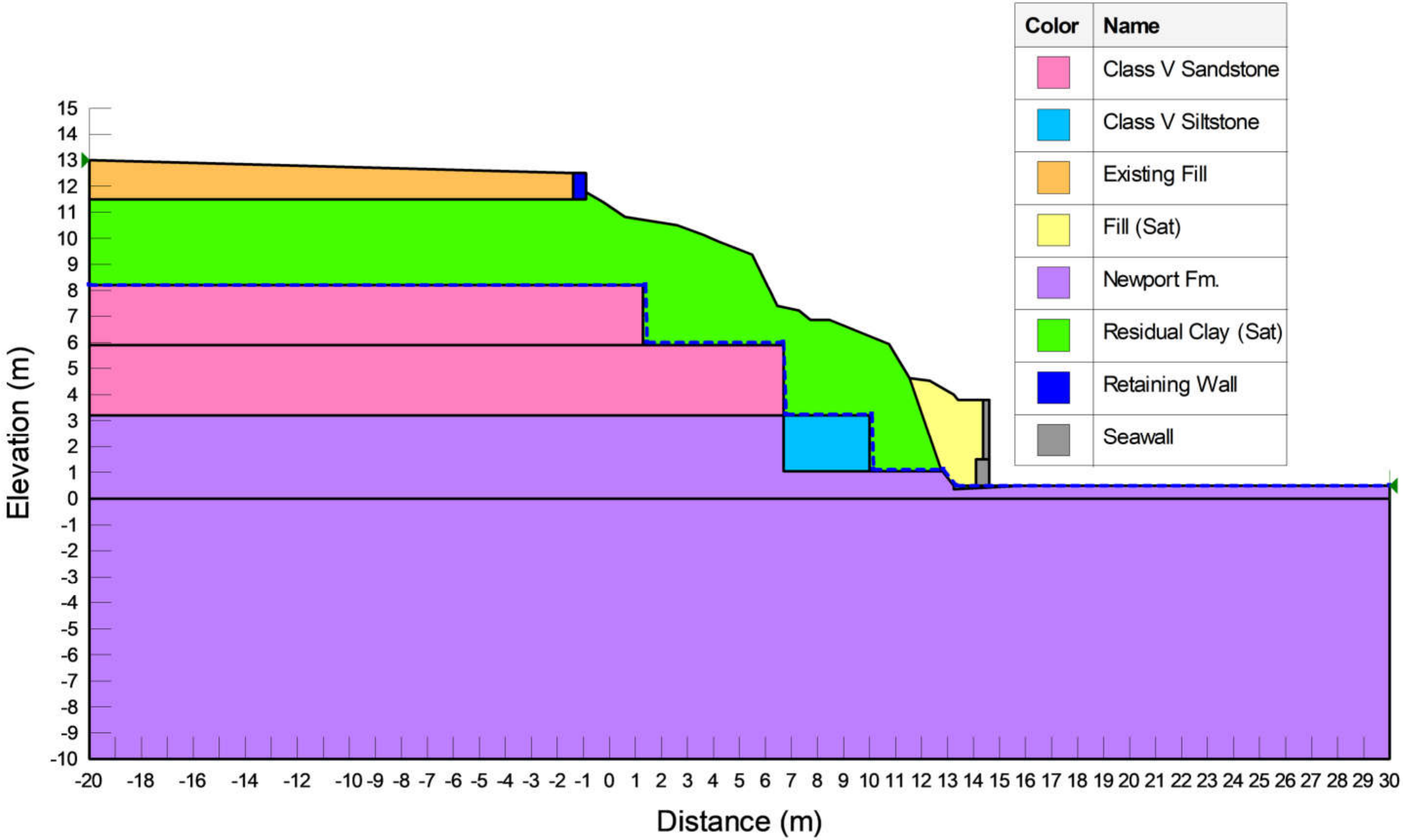
Title: **Location of Adopted Section in Analysis**

Location: 148 HUDSON PARADE CLAREVILLE, NSW

Report No: 32115RMlet2

Figure No: 1

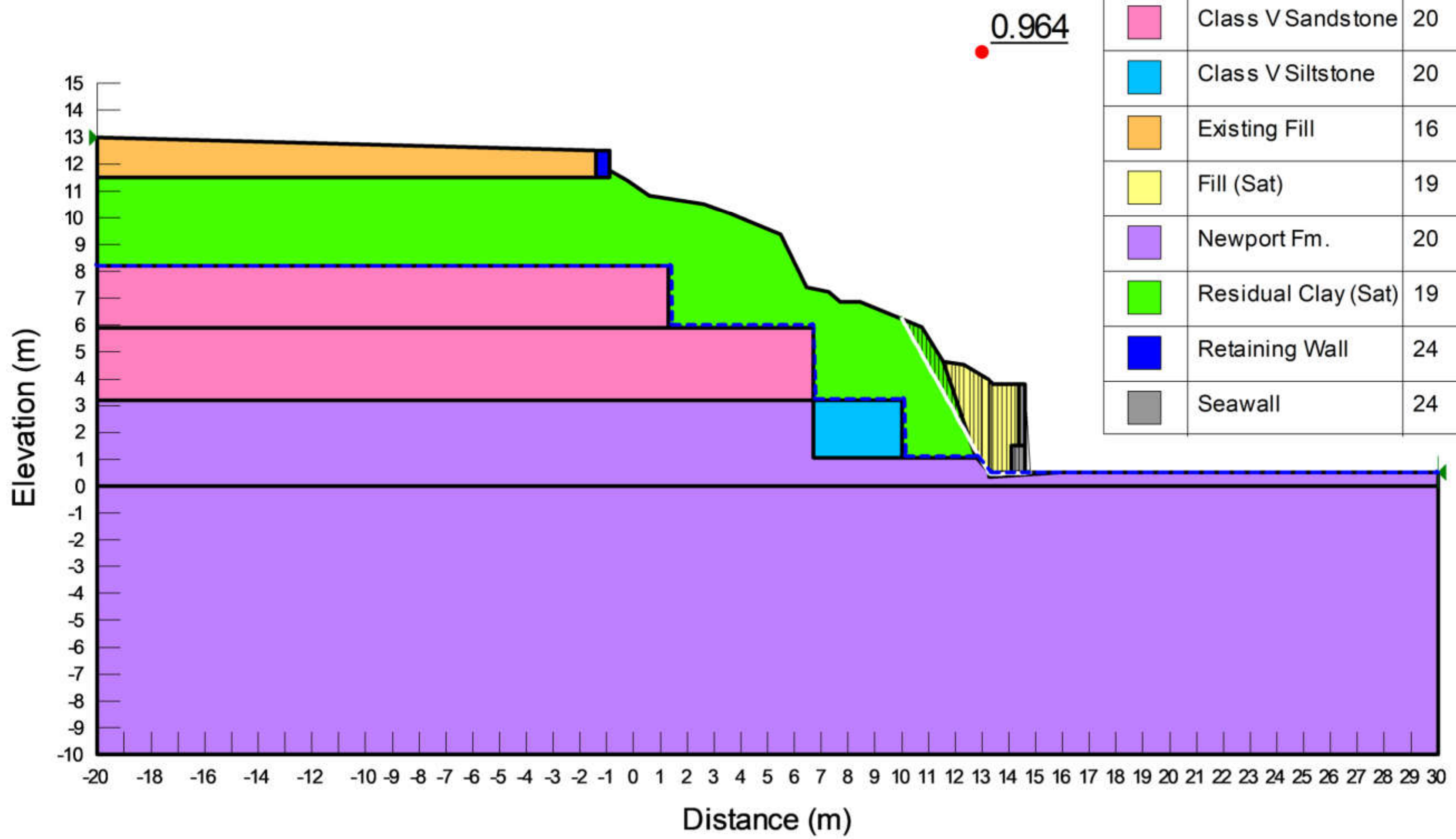
JKGeotechnics



Adopted Geotechnical Model in SLOPE/W Back Analysis

JKGeotechnics

Report No. 32115RMlet2 Figure No. 2



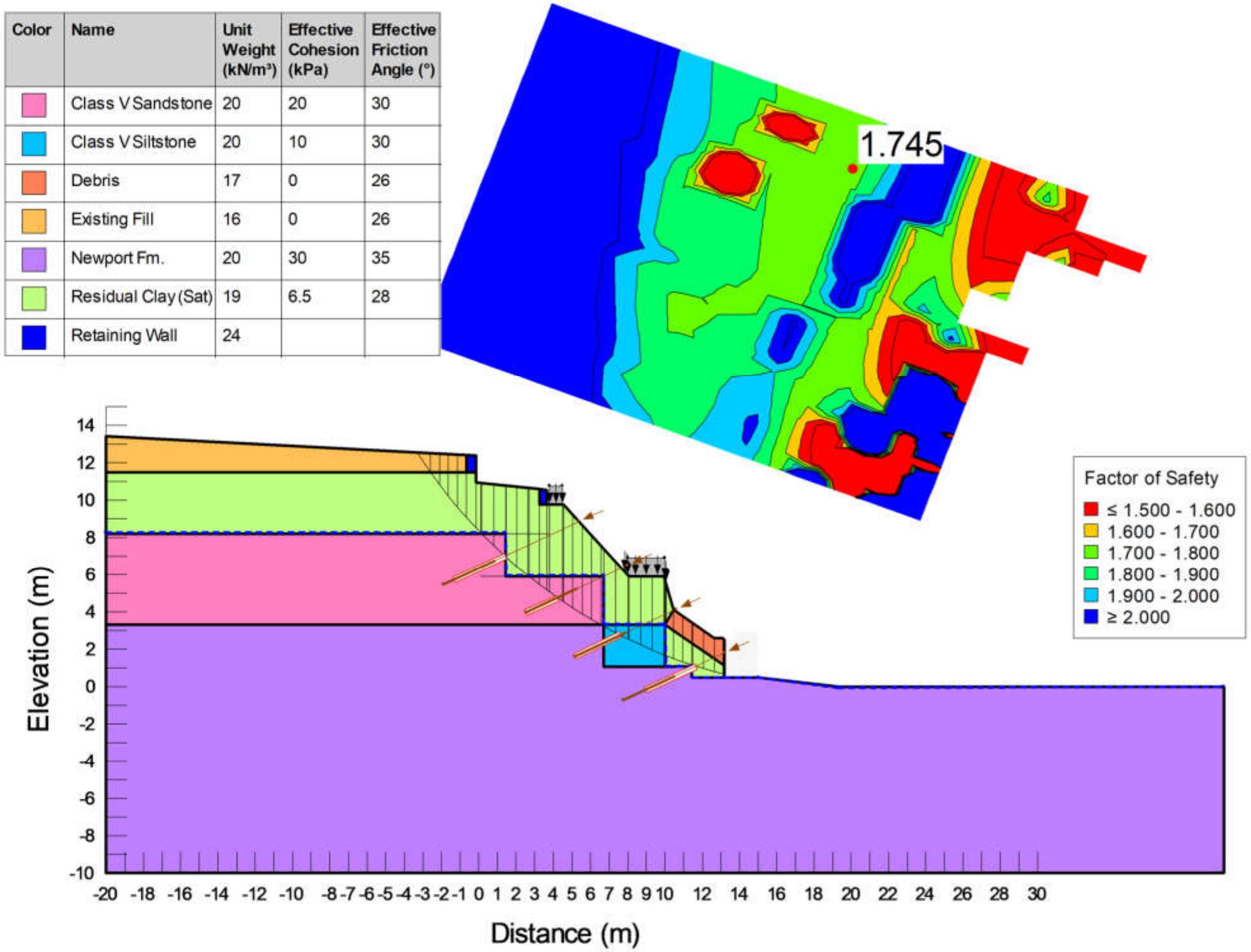
Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Class V Sandstone	20	20	30
	Class V Siltstone	20	10	30
	Existing Fill	16	0	26
	Fill (Sat)	19	2	32
	Newport Fm.	20	30	35
	Residual Clay (Sat)	19	6.5	28
	Retaining Wall	24		
	Seawall	24		

Results of SLOPE/W Back Analysis

JKGeotechnics

Report No. 32115RMlet2 Figure No. 3

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
(pink)	Class V Sandstone	20	20	30
(light blue)	Class V Siltstone	20	10	30
(orange-red)	Debris	17	0	26
(orange)	Existing Fill	16	0	26
(purple)	Newport Fm.	20	30	35
(light green)	Residual Clay (Sat)	19	6.5	28
(blue)	Retaining Wall	24		



Rock Bolts Specifications

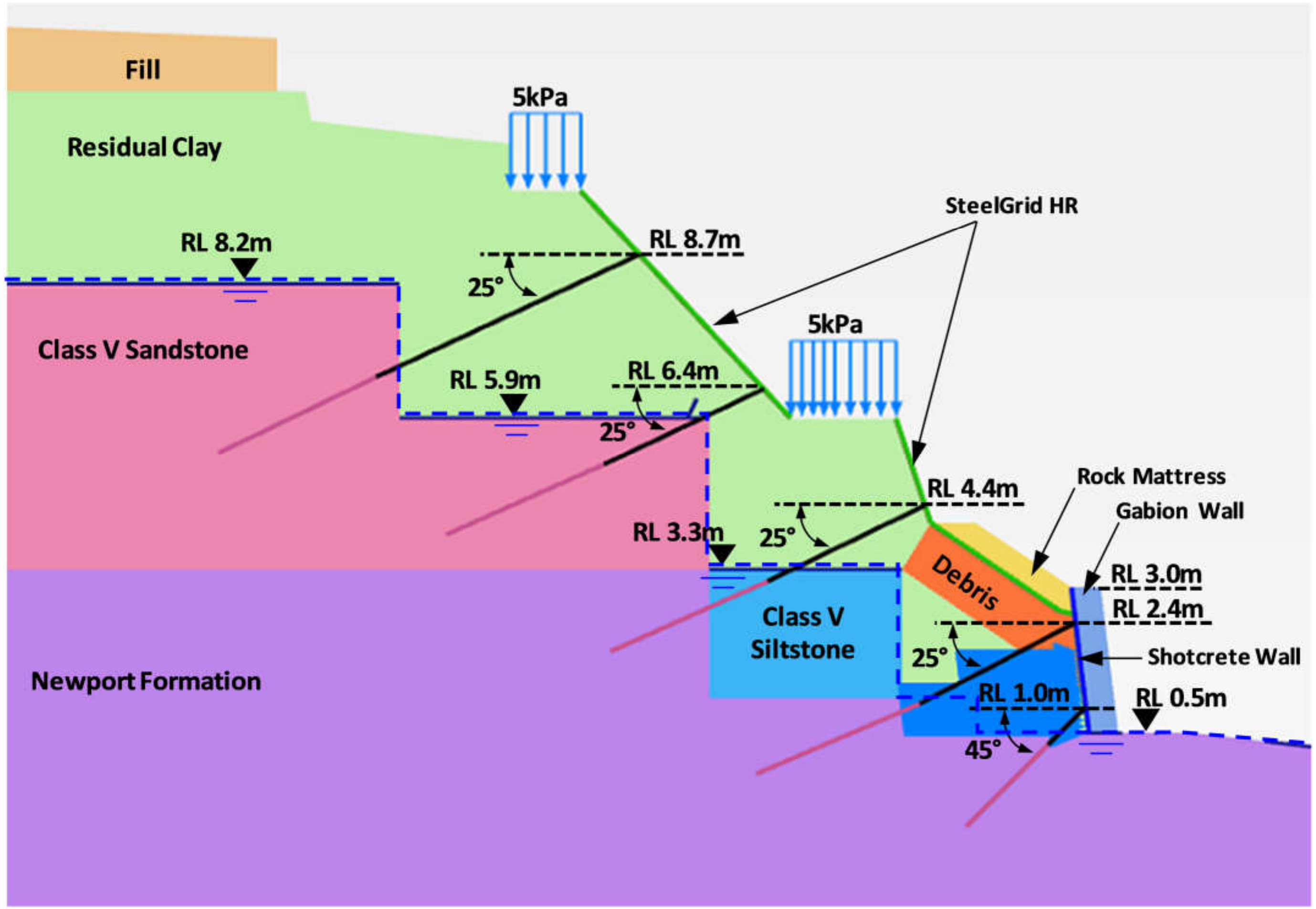
Row No	1	2	3	4
Minimum Bond Length (m)	3	3	3	3
Inclination (Deg)	25	25	25	25
Hole Diameter (mm)	60	60	60	60
Horizontal Spacing (m)	3	3	3	3
Rock Bolt Head RL (m)	8.7	6.4	4.4	2.4
Pull-out Resistance (kPa)	70	70	70	70

Color	Name	Type	Pullout Resistance (kPa)	Tensile Capacity (kN)	Bond Length (m)	Bond Diameter (m)	Out-of-Plane Spacing (m)	Face Anchorage	Factored Pullout Resistance	Factored Tensile Capacity
(brown)	3H_3m Bond_70kPa	Anchor	70	310	3	0.06	3	Yes	4.3982 kN/m/m	51.667 kN/m

Results of SLOPE/W Analysis for Reinforced Slope Adopting Revised Shear Strength Parameters

JKGeotechnics

Report No. 32115RMlet2 Figure No. 4



Adopted Geotechnical Model for PLAXIS 2D Analysis

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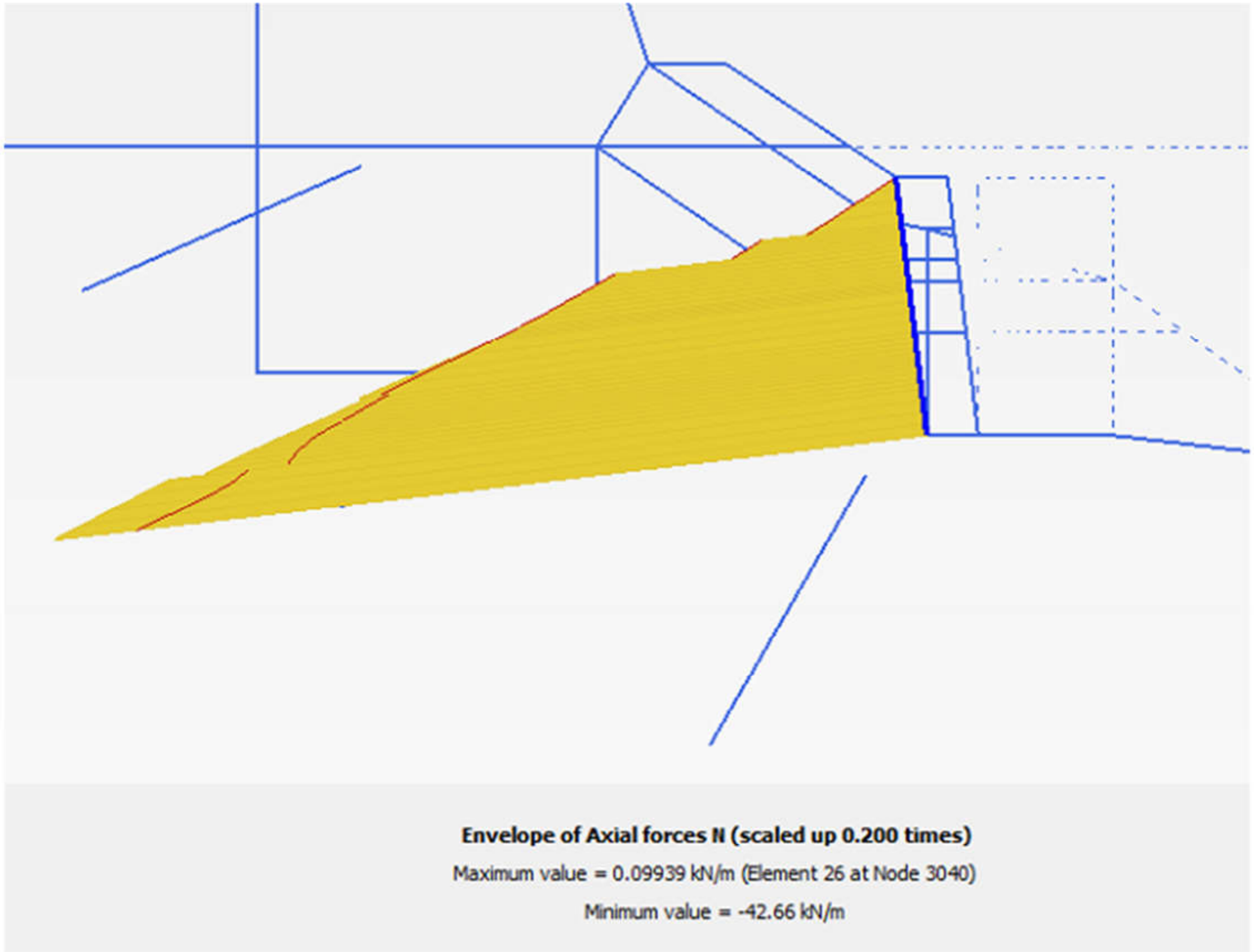
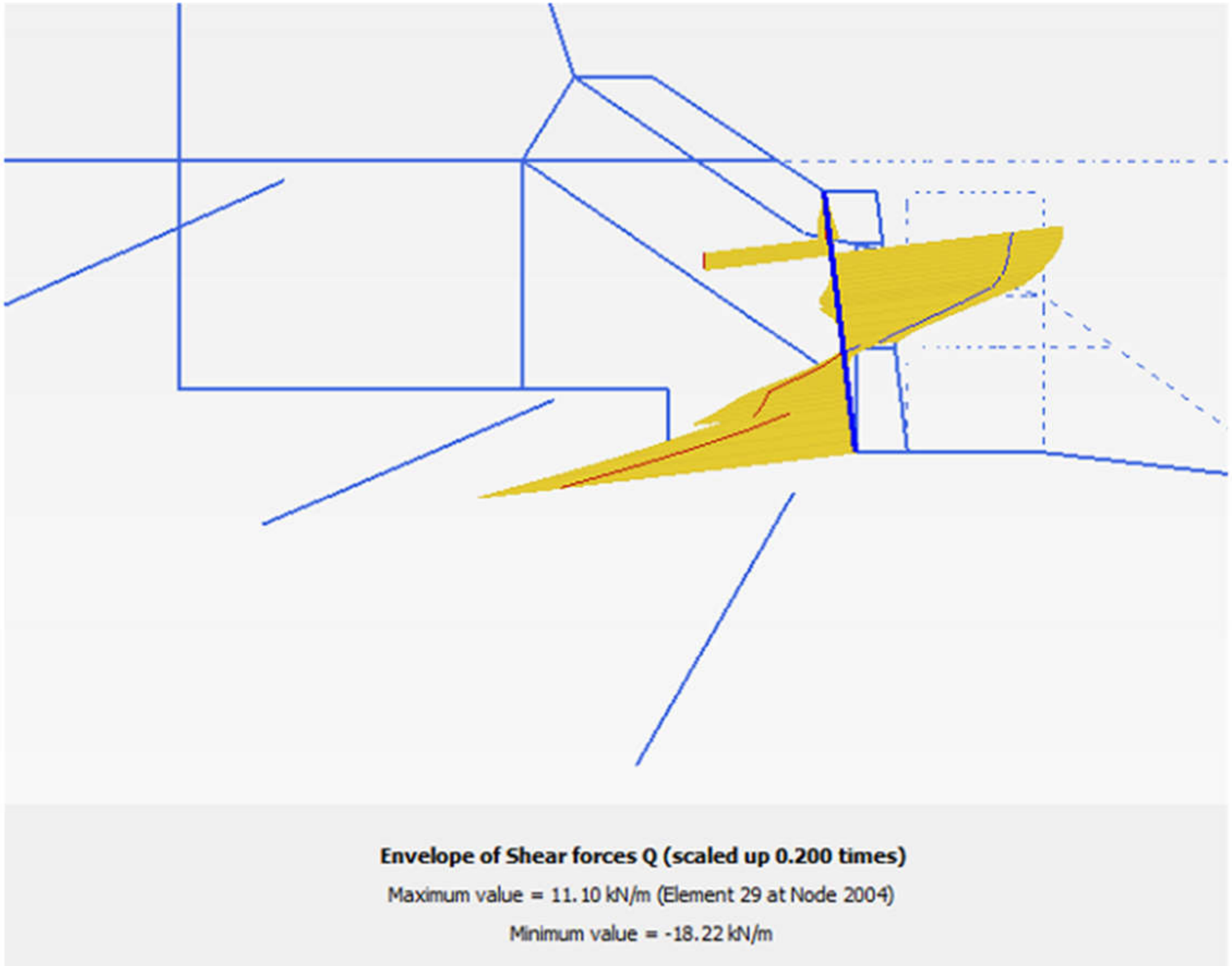
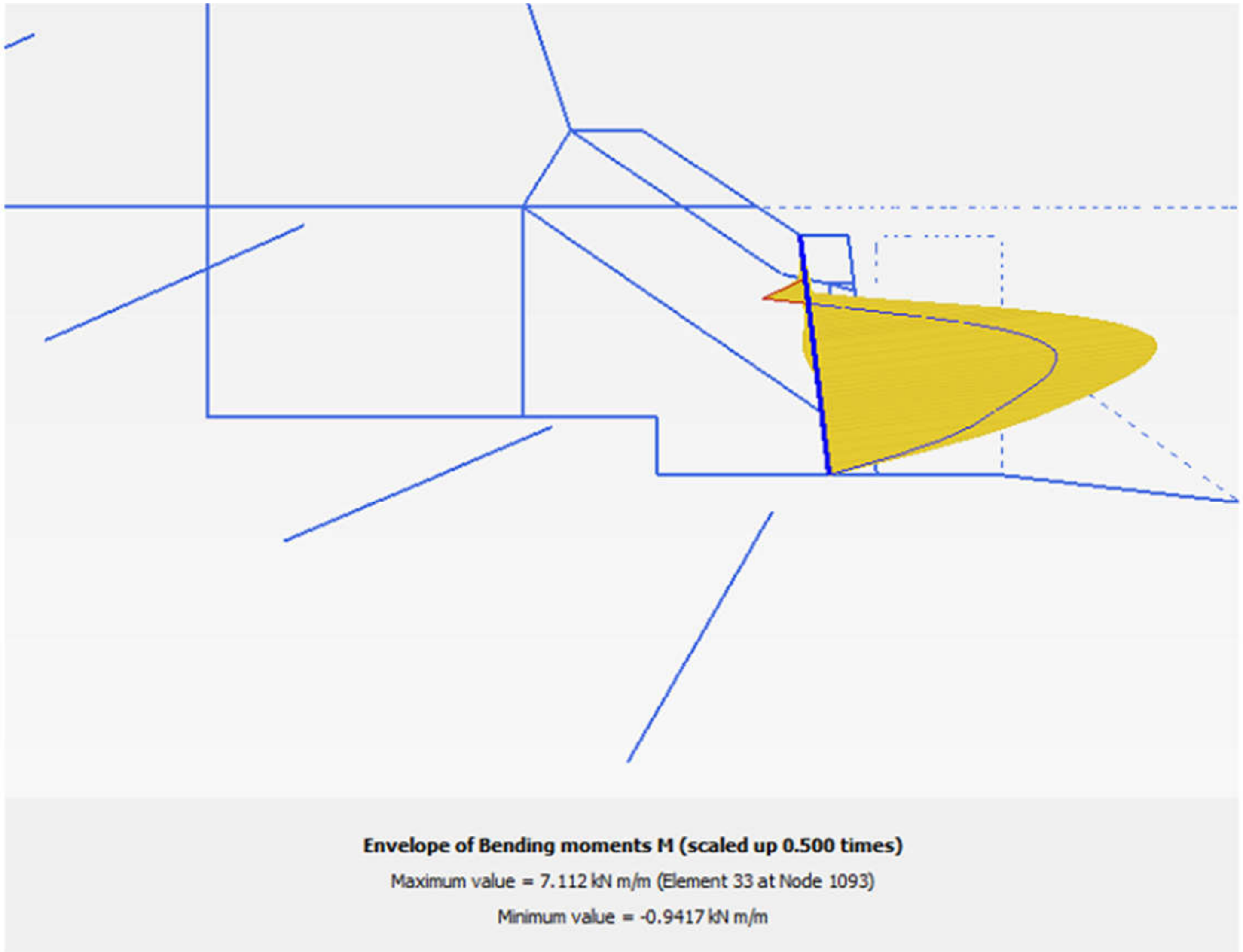
Report No. 32115RMlet2 Figure No. 5



Envelope of Axial Forces in Shotcrete



Envelope of Shear Forces in Shotcrete



Envelope of Bending Moments in Shotcrete

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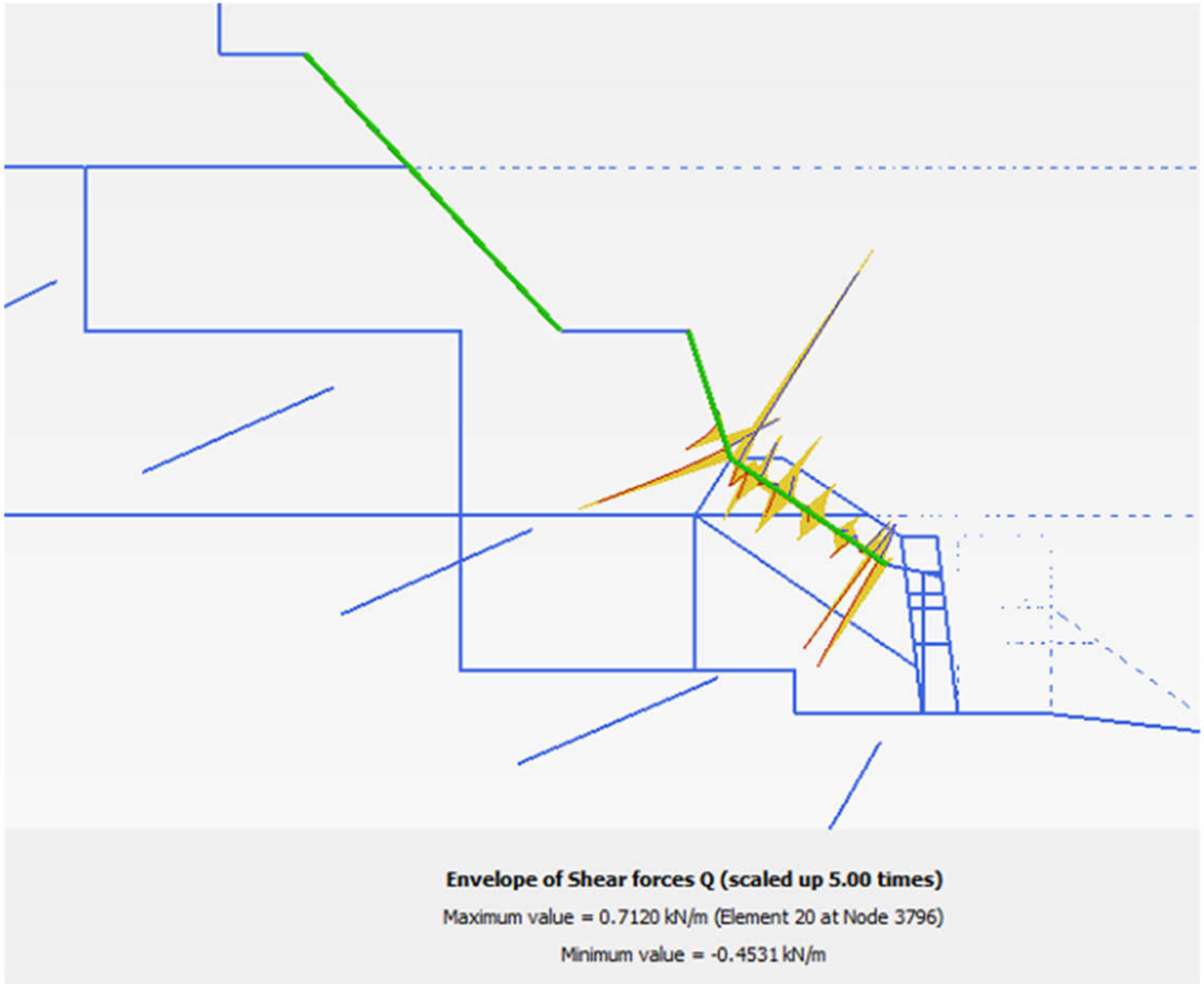
Report No. 32115RMlet2 Figure No. 8

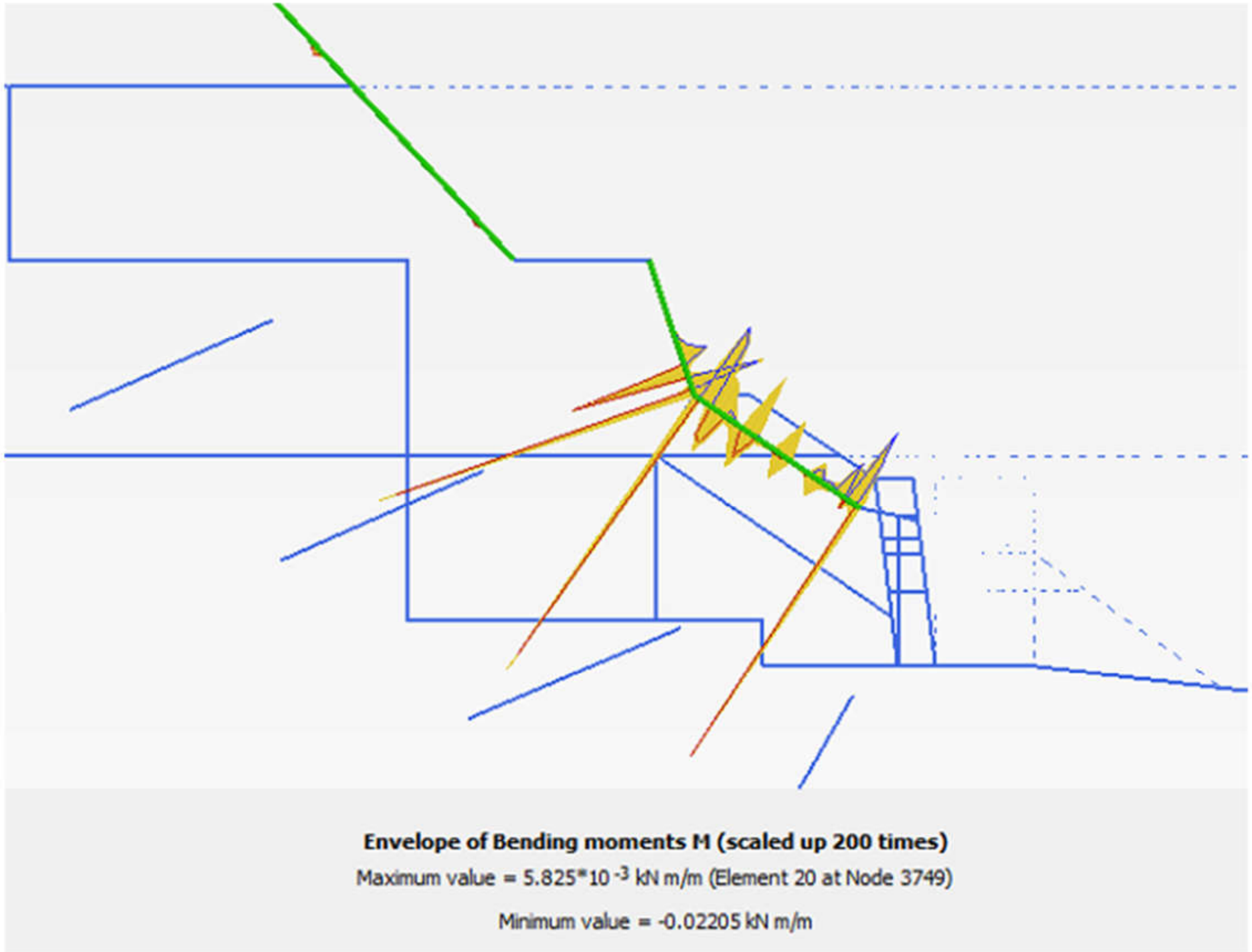
Envelope of Axial Forces in SteelGrid

JKGeotechnics

Report No. 32115RMlet2 Figure No. 9



Envelope of Shear Forces in SteelGrid



Envelope of Bending Moments in SteelGrid

APPENDIX A

Specification for Permanent Rock Bolts, Shotcrete Seawall and Steel Mesh Facing

The following specification must be read in conjunction with the following:

- 32115RMspecRev1 Figure 1: Site Plan Indicating Proposed Rock Bolts
- 32115RMspecRev1 Figure 2: Typical Section Sketch
- 32115RMspecRev1 Figure 3: Rock Bolt Head Detail Supporting Steel Mesh Facing.
- JK Geotechnics report (Ref. 32115RMlet2 rev2) dated 3 June 2021.
- The relevant drawings prepared by Royal HaskoningDHV.
- Remediation Action Plan (RAP); Ref. E32115Brpt2-RAP-rev1, dated 2 June 2021, prepared by JK Environments (JKE).
- Asbestos Management Plan (AMP); Ref. E32115Brpt3-AMP-rev1, dated 2 June 2021 prepared by JKE.

1 DRILLING HOLES, INCLUDING CLEANING HOLES

a) Rock bolt holes must be drilled at spacings and locations as set out on site by the geotechnical engineer.

b) Required hole lengths vary as shown on the drawings or as directed on site. Drill holes must be over-drilled by an additional 500mm such that incomplete cleaning does not affect bond length of bolt.

c) The minimum acceptable hole diameter must be as nominated on the drawing; 60mm.

d) Rock bolt holes must be drilled at 25° below the horizontal (Rows 1 to 4) and 45° below the horizontal (Row 5), as shown on the drawings and directed on site. Locally adjacent to the boat shed, rock bolts in Rows 4 and 5 will need to installed at a steeper downward angle as indicated on the drawings.

e) The locations of buried services must be accurately determined and any potentially intersecting rock bolts must be relocated and/or the inclination angle varied, in consultation with the Geotechnical Engineer.

f) The rock bolts must be bonded a minimum of 3m into Class V (or better) bedrock to be confirmed by the Geotechnical Engineer who must witness a representative number of the rock bolt holes being drilled. Rock bolt lengths are envisaged to be 6.8m (Row 1), 6m (Row 2, Row 3 and Row 4) and 3m (Row 5).

g) Prior to installation, all holes must be flush cleaned by using compressed air passing through a hose or delivery pipe inserted to the base of the hole. The hole will be pronounced clean once clear or almost clear air is being returned out of the hole opening. This procedure must be supervised to ensure it is being carried out correctly. If there is significant air loss then the Geotechnical Engineer must be immediately informed.

h) On completion of drilling and flushing, all holes must be plugged or otherwise protected to prevent entry of foreign matter.

i) The contractor must record for each hole, date drilled, length drilled, orientation of hole, time of compressed air clean out or water flush, details of grouting and redrilling if required. The details must be provided to the superintendent prior to installation of the rock bolt.

2 ROCK BOLTS

a) Rock bolts must consist of fully grouted BlueGeo GRP 60, fibreglass rock bolts (25mm diameter), fully threaded.

b) Bolts must be installed at 2m lateral spacings (Row 1) and 3m lateral spacings (Rows 2 to 5) and at vertical spacings of between 1.4m and 2.3m; RL8.7m, RL6.4m, RL4.4m, RL2.4m and RL1.0m as indicated on the drawings and as directed by the Geotechnical Engineer. A minimum bolt length of 50mm must protrude from the end of the nut.

c) Bolts must be locked off against a head assembly comprising the nut and plate (as per the supplier's details and the attached drawings) and engaged with the steel mesh facing or shotcrete reinforcement mesh, as indicated on the attached drawings.

d) Total in hole length of rock bolts must be as shown on the drawings or as directed on site by the Geotechnical Engineer.

e) The adopted safe working load of the rock bolts must be as directed by the suppliers.

f) Care must be taken to prevent damage, kinking or bending of bolts. Any bolts sustaining damage must not be used.

g) Bolts must be kept free from oil, grease, mud or any other deleterious substances, and stored in accordance with the supplier's recommendations.

3 INSTALLATION AND GROUTING

a) Spacers or spiders must be provided along the length of the rock bolts to maintain them centrally within the drill hole.

b) Grout mix to surround rock bolt must have a target water/cement ratio of 0.45 UNO. A target laboratory test criterion would be an average grout strength of 40MPa at seven days (no single test shall be less than 25MPa).

c) Grout must be pumped to the base of the hole through hoses or grout tubes until the consistency of the grout mix escaping at the hole openings is the same as that being pumped in. Once this is the case, the grout tube must be withdrawn slowly such that the rate of grout exiting the hole is virtually maintained. Only when the tube is completely removed from the hole must the pumping mechanism be switched off.

d) If grout level drops below drill hole opening whilst still wet, it must be topped up until loss of grout is negligible. If the grout level cannot be maintained, and/or excessive grout is required, then the rock bolt must be withdrawn and the hole grouted and then redrilled. All holes with excessive grout take and/or grout loss must be immediately identified to the Geotechnical Engineer.

e) Once grout is dry or almost dry, a thick, non-shrink topping grout must be packed into the hole until the grout completely covers the bolt/dowel up to the drill hole opening. The grout must be finished flush with the surrounding rock face.

f) The rock bolt head assembly must comprise the components recommended by the suppliers. The threaded length of the bolts will be sufficient to permit full engagement with the mesh.

4 STEEL MESH FACING OVER THE SOIL SLOPE

a) One layer of 'MASTATEX orange Hi Vis geotextile warning layer' or other similar product draped from top down.

b) One layer of Maccaferri Steelgrid HR PVC 100 with double twisted hexagonal mesh shall be draped from the top row of anchors (Row 1) downwards, overlaying the soil erosion layer and secured to the rock bolts in Rows 2 and 3.

c) The steel wire mesh shall be restrained under Blue Geo GRP bolt plates and nuts.

d) Connectivity from one roll to the next to be as per manufacturers guidelines. Overlap not necessarily required.

e) The mesh shall be kept free from oil, grease, mud or any other deleterious substances. The steel should not be visibly pitted or rusted.

5 LOAD TESTING OF ROCK BOLTS

a) All rock bolts shall be load tested before placing the steel mesh facing or reinforcement mesh. Load testing of the rock bolts shall be to 1.5 x the maximum working load (50kN), i.e. test load of 75kN and as directed by the Geotechnical Engineer. All tests to be in the presence of the geotechnical engineer. The load must be applied in three load increments; 0.5 x working load, working load then the test load. The rate of load application must not exceed 5kN/minute.

b) The load must be held for one minute at the initial two load increments (0.5 x working load and working load) and the test load must be held for 15 minutes.

c) The displacement must be recorded at the start and finish of the initial two load increments (0.5 x working load and working load). The displacement must be recorded each minute for the test load period of 15 minutes. Plots of load versus displacement must be recorded for each rock bolt, and reviewed and approved by the Geotechnical Engineer before placing the reinforcement mesh.

c) Rock bolts that fail the load testing must be replaced.

6 FIRE PROTECTION

All bolt heads including nut and plate to be protected by concrete with minimum100mm cover.

7 AUSTRALIAN STANDARDS

Wherever Australian Standards exist with regard to the materials and workmanship referred to in this Specification, then they shall be deemed to apply.

LEGEND

- BOREHOLE
- ROW 5 RL1.0m
- ROW 4 RL2.4m
- ROW 3 RL4.4m
- ROW 2 RL6.4m OR ABOVE STEP AT REAR OF SLIP
- ROW 1 RL8.7m

NOTES:

1. ACTUAL BOLT PLACEMENT DEPENDANT ON LOCALISED GEOMETRY & TO BE SET OUT BY GEOTECHNICAL ENGINEER.
2. ROW 4+5 BASED ON ELEVATIONS @ CHAINAGES: 0.6m, 2.5m, 8.0m, 9.5m, 12.25m, 15.0m, 16.5m, 19.25m AND 22.0m.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

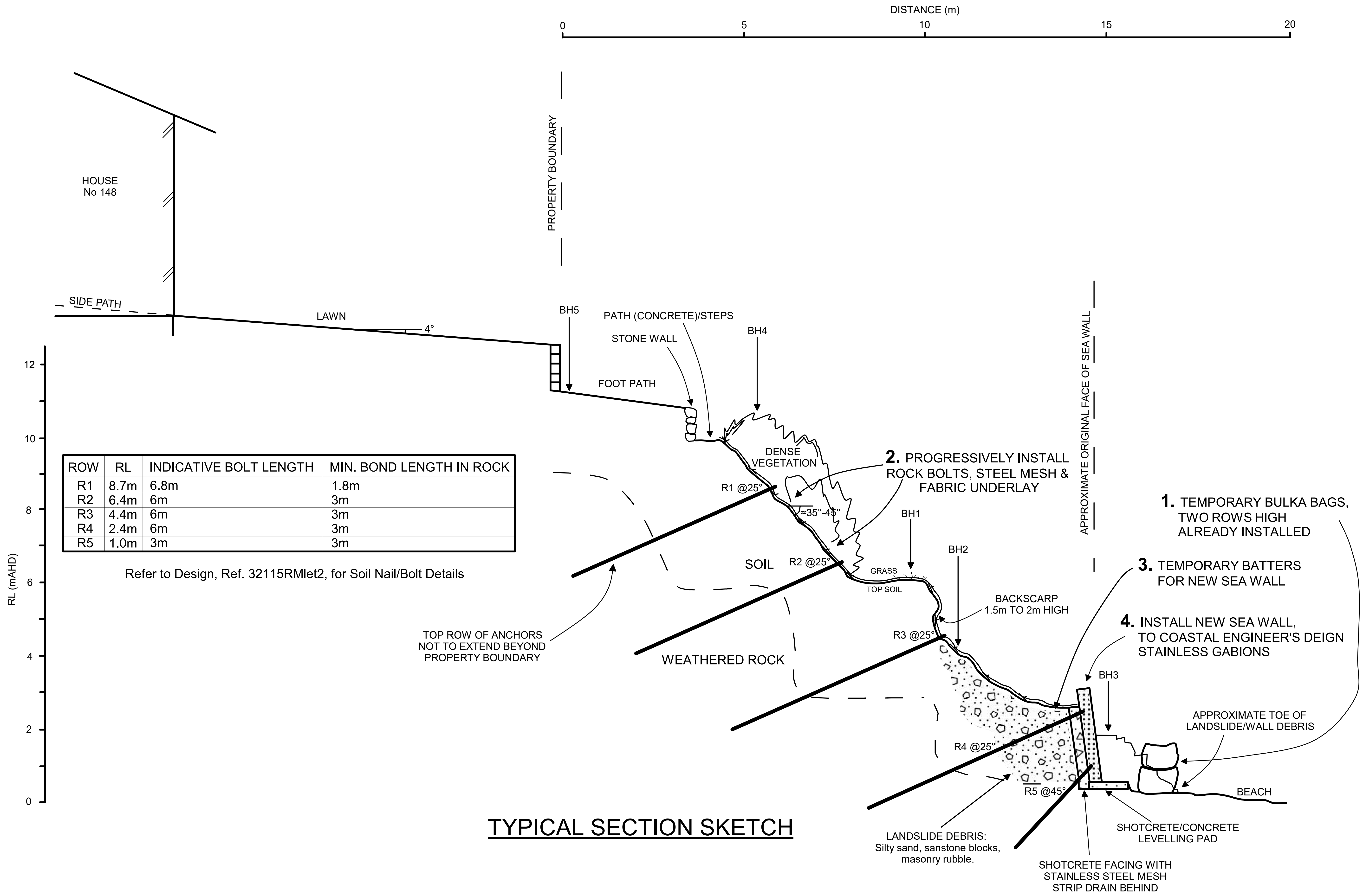
0 2 4 6 8 10
SCALE 1:200 @A3 METRES

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

Title:	SITE PLAN INDICATING PROPOSED ROCK BOLTS
Location:	148 HUDSON PARADE CLAREVILLE, NSW
Report No:	32115RMspecRev1
Figure No:	1

JKGeotechnics

PLOT DATE: 1/06/2021 11:57:58 AM DWG FILE: Y:\32000's\32115R CLAREVILLE\CAD\32115RMSPECREV1.DWG



ROW	RL	INDICATIVE BOLT LENGTH	MIN. BOND LENGTH IN ROCK
R1	8.7m	6.8m	1.8m
R2	6.4m	6m	3m
R3	4.4m	6m	3m
R4	2.4m	6m	3m
R5	1.0m	3m	3m

Refer to Design, Ref. 32115RMlet2, for Soil Nail/Bolt Details

TYPICAL SECTION SKETCH

0 5
Scale 1:100 @A3 Metres

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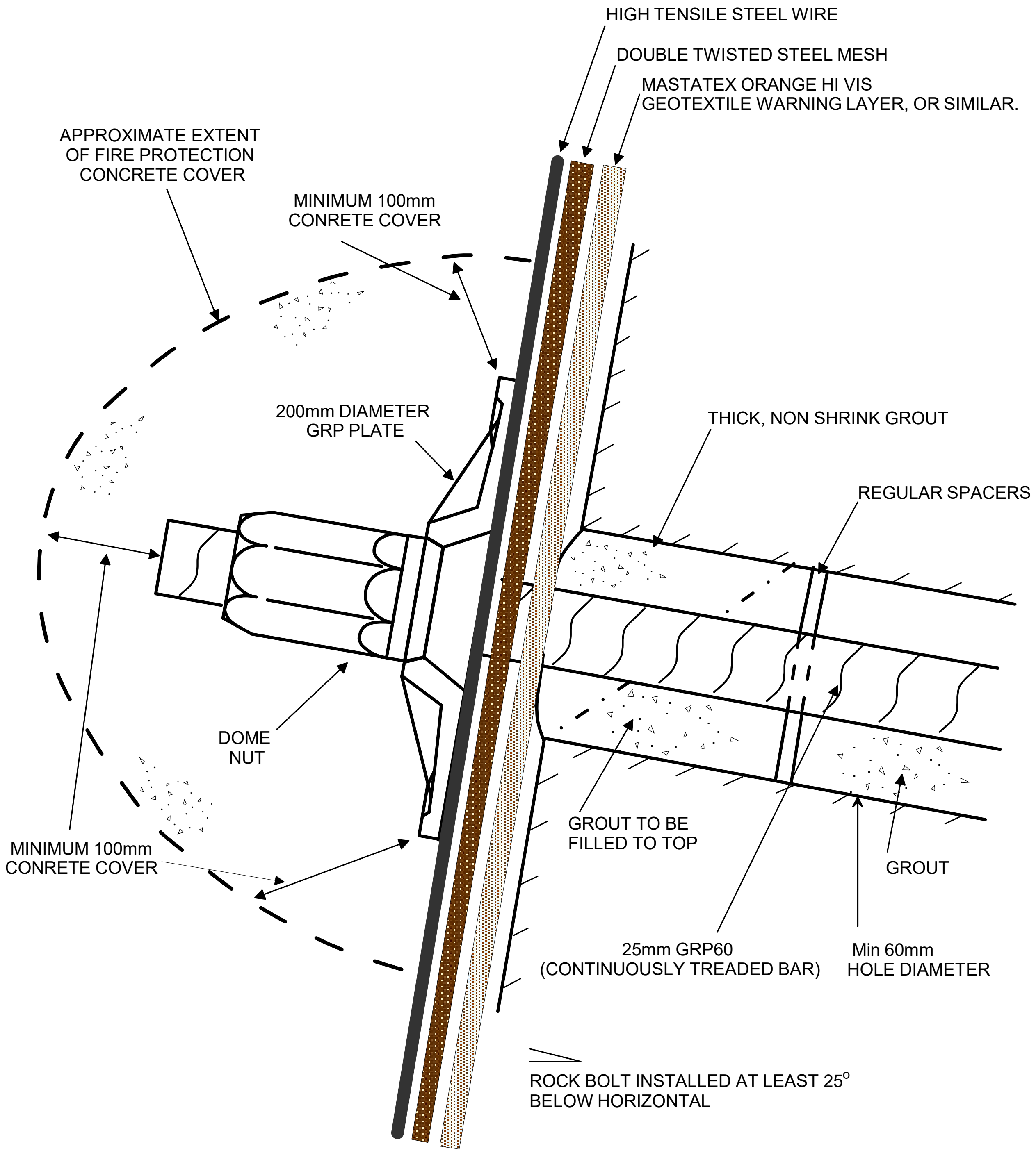
Report No. 32115RMspecRev1 Figure No. 2



TYPICAL ROCK BOLT HEAD DETAIL SUPPORTING STEEL MESH FACING

NOTE: REFER TO TEXT OF SPECIFICATIONS FOR FURTHER DETAILS.

NOT TO SCALE

JKGeotechnics

Report No. 32115RMspec Rev1 Figure No. 3

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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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:

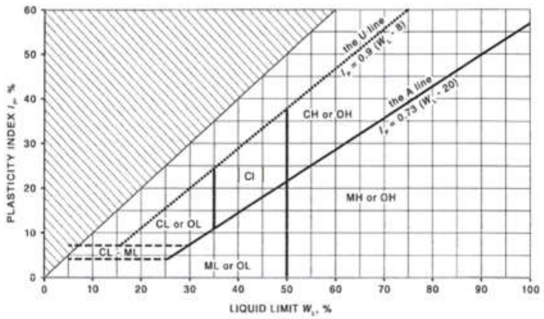
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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