GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for________________ Civia_____________________________
Name of Applicant
Address of site __1102 Barrenjoey Road, Palm Beach______________________________

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

I, ________Woodie Theunissen__________ on behalf of JK Geotechnics Pty Ltd
(Insert Name) (Trading or Company Name)

on this the ____21 June 2024____ certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million. *we*/I have:

Please mark appropriate box

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical Assessment

Report Date: 21 June 2024 Report Ref No: 33618Yrpt2 FEM
:
Author: Woodie Theunissen

Author's Company/Organisation: JK Geotechnics Pty Ltd

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

Architectural drawings prepared by Innovate (refer to drawings cover page, dated 18 June 2024)
Structural Drawings prepared by van der meer (Job No: SY200209, Drawing No: S02-01, S02-05, S02-11, S02-15 and S02-52

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

Signature …… …………………………………..

Name ……Woodie Theunissen…………………………………………..

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

Membership No. ……889807……………………………………

Company: JK Geotechnics Pty Ltd.

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for________________Civia____________________

Name of Applicant

Address of site __1102 Barrenjoey Road, Palm Beach______________________________

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

Report Date: 21 June 2024 Report Ref No: 33618Yrp2 FEM

Author: Woodie Theunissen

Author's Company/Organisation: JK Geotechnics Pty Ltd

Please mark appropriate box

☐ Comprehensive site mapping conducted ____3 November 2020_______
(date)

☐ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:250 (as appropriate)

☐ Subsurface investigation required

- ☒ No Justification …Already completed
- ☐ Yes Date conducted ………………………

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

☐ Geotechnical hazards identified

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

☐ Geotechnical hazards described and reported

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

- ☐ Consequence analysis
- ☐ Frequency analysis

☐ Risk calculation

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

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

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

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

☐ Design Life Adopted:

- ☐ 100 years
- ☐ Other ……………………………………………
specify

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

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

○ Risk assessment within Bushfire Asset Protection Zone.

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

Signature … …………………………………………
Name …Woodie Theunissen
Chartered Professional Status……CPEng………………………
Membership No. ……889807…………………………….
Company JK Geotechnics Pty Ltd.

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REPORT TO

CIVIA

ON

THREE-DIMENSIONAL (3D) NUMERICAL ANALYSIS

FOR

PROPOSED RETENTION SYSTEM

AT

1102 BARRENJOEY ROAD, PALM BEACH, NSW

Date: 21 June 2024
Ref: 33618rpt2 FEM

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DOCUMENT REVISION RECORD

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a three-dimensional (3D) numerical analysis of the proposed retention system to support the proposed mixed use development at 1102 Barrenjoey Road, Palm Beach, NSW, during excavation. The location of the site is shown on the attached Figure 1. The analysis was commissioned by Mr Mino Howard of civia.

We understand from the provided architectural drawings prepared by Innovate Architects (refer to drawings cover page, Job No: 2926 dated 18 June 2024), that the proposed development will comprise the following:

- Demolition of the existing structures on site, and,
- Construction of a three-storey building over one basement garage level.

The new building will be cut into the existing hillside with the basement will have a finished floor level of RL- 0.65m, or an assumed bulk excavation level of RL-1m. This will result in excavation to a maximum depth of about 12.5m over the rear of the site and about 3.5m over the front. While it appears that significant excavation is required, due to the site already being partially excavated, excavation over most of the site will be limited to about 3.5m, with only approximately the rear 6.5m requiring excavation to greater depths. The basement is proposed to extend to the eastern, southern and western site boundaries and to within about 0.5m of the northern site boundary. The proposed excavation will remove the existing soldier pile wall and where sandstone boulders are present along the southern site boundary, these will be trimmed back to the extent that they protrude into the building footprint.

The footprint of the proposed development is indicated on Figure 2.

A 3D model of the site, surrounds and proposed retention system was developed. The purpose of developing this model was to provide advice on the performance of the shoring system in terms of deformations, shear and axial forces, bending moments and behind wall pressures. In addition, the predicted induced settlements below adjoining structures were also reported. While we have provided the design forces and bending moments the structural engineer must make their own assessment of whether these design forces and bending moments are appropriate and that the structural integrity of the wall will be appropriate for this site, both in terms of the deflections, forces and bending moments and predicted movements induced below adjoining structures.

2 GEOTECHNICAL INFORMATION UTILISED IN THE NUMERICAL ANALYSIS

The following geotechnical information has been used to develop a subsurface model for the site and complete the 3D numerical finite element analysis:

- Geotechnical Report Prepared by JK Geotechnics Pty Ltd (Ref: 33618YJrptrev4C, dated 22 March 2024) (Appendix A1)

The assumptions, methodology and results of our analysis are presented in Sections 3, 4 and 5 of this report.

3 PROPOSED GEOMETRY AND CONSTRUCTION METHODOLOGY

The existing ground levels across the site and adjoining properties have been utilised in the development of our 3D model and are shown in the survey prepared by C.M.S. Surveyors Pty Ltd (Drawing Name: 19783detail, dated 19 November 2020). The proposed development is shown in the supplied architectural drawings by Innovate Architects (refer to drawings cover page, Job No: 2926 dated 18 June 2024) and the supplied structural drawings by van der Meer Consulting (Job No. SY200-209, Dwg. No. S02-01, S02-05, S02-11, S02-15 and S02-52, Rev. 0, dated 18 June 2024). Based on these drawings the new building will have three above ground levels over a single basement level.

The basement level will have a finished floor level at RL-0.65m, with an anticipated bulk excavation level (BEL) extending to RL-1.0m. Consequently, over much of the site the basement will require excavation to depths up to approximately 3.5m below existing surface levels, although to the rear of the site the building will be cut into the hillside and will result in excavation to a maximum depth of about 12.5m. The basement will extend to the southern, eastern and western boundaries and will be setback about 0.5m from the northern site boundary.

Construction of the basement will require installation of a 0.6m diameter secant pile wall over the front or western portion of the site and a 0.45m diameter solider pile wall with piles at 0.8m centres over the rear of the site. The eastern soldier pile wall will be buttressed by seven 0.6m diameter secant pile wall that extend out from the soldier pile wall. The middle buttresses will have a length of 4.5m while the northern and southern buttresses will form the northern and southern boundary walls over the higher portion of the proposed excavation and extend for a distance of 6m. With the exception of the retaining wall along the eastern boundary, which will be buttressed, the remaining walls will be cantilevered. The toe of the soldier pile walls will be installed to RL- 6m while the buttresses and secant pile walls will be installed to toe levels of RL-4m. The staging of the excavation comprises the installation of the retention system, excavation to bulk excavation level and in the long term tanking of the basement to RL3m. The model staging is set out in detail in Section 4.5.4 below.

It should be noted that our modelling does not take into account temporary works, such as the construction of platforms to allow the installation of piles over the upper portion of the site or any disturbance of the soils and potential settlement that may occur during the construction process of the walls. In the long term the walls will be supported by the constructed slabs and structure.

4 NUMERICAL ANALYSIS

4.1 Introduction

The finite element analysis was carried out using the 3D computer program PLAXIS. The 3D model has been prepared based on the information provided and attached in the relevant appendices to this report.

Staged 3D modelling was completed to simulate the installation of the soldier and secant pile walls and excavation to bulk excavation level. The predicted cumulative displacements, axial forces, shear forces and bending moments of the proposed shoring walls as a result of the simulated construction process were calculated. The predicted wall displacements and design shear forces, axial forces and bending moments are reported in Section 5 below. The model was extended beyond the northern, southern and western site boundaries to allow the effects of the proposed development on the adjoining buildings to be assessed.

4.2 Retention System

Based on the above referenced drawings prepared by van der Meer Consulting the following has been adopted for the modelling of the retention system:

- Over the eastern portion of the site a soldier pile wall comprising 0.45m diameter piles installed at 0.8m centres with the toe of the wall at RL-6m.
- The eastern wall is buttressed by seven buttresses comprising 0.6m diameter secant piles with a toe level of RL-4m. The northern and southern buttresses extend a distance of 6m out from the eastern wall and form the upper or eastern portion of the northern and southern walls. The central five buttresses will extend a distance of 4.5m out from the eastern wall.
- Walers running horizontally between buttresses and comprising 400WC270 and 500WC340 steel sections.
- Over the western portion of the site a secant pile wall comprising 0.6m diameter piles will be installed to a toe level of RL-4m.
- Topping the piles over the eastern side of the site are capping beams with cross sections comprising 750mm x 500mm while over the western portion they comprise 650mm x 600mm cross sections.
- The proposed BEL is at RL-1m.

4.3 Geotechnical Model

A 3D model of the site and its surrounds, as shown in the attached Figure 3, was developed based on the survey plan and the available geotechnical information obtained from the geotechnical investigations. The geotechnical model divides the subsurface profile into a number of soil, sandstone and siltstone units with specific geotechnical properties. A groundwater profile has also been adopted for the site. The assumed topography and subsurface and groundwater profiles are shown in Figure 4.

Geotechnical parameters were selected for each geological unit based on the insitu testing detailed on the borehole logs and DCP test results and empirical correlations well established in geotechnical engineering. In our selection of parameters, consideration was given to the inherent uncertainty associated with natural, non-engineered materials such as variability in relative densities/strengths and permeability.

In addition to the geotechnical parameters, the potential adversely orientated joints within the rock mass was also considered. As the Palm Beach peninsula has developed as a result of the down cutting of the plateau during the formation of Pittwater, jointing within the rock mass will have developed as a result of stress relief as the rock confining the peninsula has been eroded. This would have resulted in the development of a primary joint set running parallel to Pittwater, with joints dipping towards Pittwater and the west. Consequently, the most likely and critically orientated joints are those dipping down to the west.

To model this potentially worst-case condition a series of joints have been included in the model that dip down to the west at 45° with the lowest of these joints striking the back of the wall 0.5m above bulk excavation level (i.e. RL-0.5m). Parallel joints spaced have then been modelled that intersect the back of the wall. In addition to these 45° inclined joints, two vertical joints have also been modelled that run perpendicular to the eastern wall and just inside the northern and southern walls and form release planes for the 45° wedge of rock. While jointing was logged in the cored boreholes, it is extremely unlikely that the jointing conditions modelled will be encountered. As such, the modelling approach adopted for the eastern wall is considered to be very conservative.

This geotechnical model was then used as the basis of our analysis. PLAXIS 3D was used to analyse the soil structure interactions. Staged modelling was completed to simulate the construction staging and surrounding site conditions.

4.4 Applied Loads

The following applied surcharge loads have been adopted:

- A 20kPa surcharge over Barrenjoey Road and 5kPa over the footpath.
- A 30kPa surcharge over the footprint of Barrenjoey House.
- A 10kPa surcharge load over the footprint of the secondary dwelling to the east of the site and
- A 10kPa to 20kPa surcharge load over the footprint of the dwelling to the south.

4.5 Model Parameters and Stages

4.5.1 Geotechnical Parameters

The subsurface profile was based on the results of the geotechnical investigations. Due to the limitations of 3D modelling, some generalisation of the profile was required to prevent numerical inaccuracies and computational difficulties occurring in the analysis due to thin and tapered soil layers formed due to linear interpolation between subsurface conditions nominated at inputted borehole locations.

A small strain soil hardening model was used to model the behaviour of the soils while the sandstone and siltstone bedrock was modelled using the Mohr-Coulomb model. The table below details the parameters adopted for the soils and the bedrock.

Geotechnical Parameters Adopted for Small Strain Hardening Soil Model			
Parameter	Colluvium	Fill	Sand Very Loose
Unsaturated Unit Weight (kN/m^3)	18	17	15
Saturated Unit Weight (kN/m^3)	19	18	17
Internal Angle of Friction (°)	33	28	28
Cohesion (kPa)	1	2	0
Dilation Angle (°)	3	0	0
Modulus (E_{50}) (MPa)	15	8	8
Modulus (E_{oed}) (MPa)	15	8	8
Unload/Reload Modulus (E_{ur}) (MPa)	45	24	24
Poisson's Ratio (ν_{ur})	0.2	0.2	0.2
Shear Strain at 0.7G_o	1.5 x 10^{-4}	1.5 x 10^{-4}	1.5 x 10^{-4}
Reference Shear Modulus (G_o^{ref}) (MPa)	46.88	25	25
R_{inter}	0.67	0.67	0.67

The parameters recommended by D.A.F. Oliviera (2014) for rock masses where joints have not been explicitly modelled have been adopted to model the bedrock and are presented below. To assess the potential impact of adverse jointing dipping down to the west behind the eastern retaining wall we have explicitly modelled joints in the rock mass. Consequently, we consider this to be a conservative approach.

Geotechnical Parameters Adopted for Mohr Coulomb Model								
Parameter	*Unsaturated Unit Weight (kN/m^3)*	*Saturated Unit Weight (kN/m^3)*	*Cohesion (c) (kPa)*	*Internal Angle of Friction (ϕ)*	*Dilation Angle (ψ)*	*Elastic Modulus (E) (MPa)*	Poisson's Ratio (ν')	*R_{inter}*
Sandstone Bedrock – Class V	23	23	65	28	0	200	0.3	0.67
Sandstone Bedrock – Class IV	23	23	140	43	13	700	0.25	0.67
Siltstone Bedrock – Class V	23	23	30	26	0	100	0.3	0.67
Siltstone Bedrock – Class IV	23	23	90	33	3	350	0.25	0.67
Joint	0	0	5	28	2	80	0.3	0.67

Based on the longer term groundwater monitoring completed, which is attached as Appendix B1, the groundwater level across the level western portion of the site and Barrenjoey Road ranges from about RL0.7m to RL1.85m, with the lower groundwater table measured in the sandy soils and the higher groundwater table measured in the shallow bedrock towards the eastern side of the site. For the purpose of the modelling, we have assumed a groundwater level over the western portion of the site where sandy soils are present at RL1m. This then increases in height where shallow bedrock is encountered and has been modelled to rise steeply behind the installed eastern retaining wall, which simulates dewatering during excavation and construction. In the long term the groundwater level has been assumed to be at RL3m following tanking of the basement over the eastern portion of the site where bedrock is present at shallow depth. Where deeper sand is present over the western portion of the site the groundwater has been assumed to be at RL1m. Considering the steep nature of the hillside, the adoption of the above water levels over the eastern side of the site and beyond the eastern site boundary is considered to be very conservative.

4.5.1 Insitu Stresses

Due to the downcutting of the plateau during the formation of Pittwater, the resulting steeply sloping terrain and the relatively poor quality of the bedrock, the bedrock is considered to be stress relieved. Consequently, high horizontal insitu stresses have not been adopted for this model. In the calculation of insitu stresses, the following relationship has been adopted:

$K_o = 1- \sin\phi$

4.5.2 Structural Parameters

The structural parameters adopted for the model were provided to us by van der Meer Consulting. The retaining walls have been modelled as plate elements while the capping beams and walers have been modelled as beam elements. The buttresses have typically been modelled as volumes, with the exception of the southern and northern buttresses, which have been modelled as plates. Interface elements have been used to model the structure/soil interaction. The adopted structural parameters are outlined below.

Retaining Walls – Modelled as Plates					
	Pile Diameter (m)	***Pile Spacing (m)***	***Elastic Modulus (E) (MPa)***	***Equivalent Elastic Modulus (E_{eq}) (MPa)***	***Equivalent Plate Thickness (m)***
Secant Pile Wall	0.6	1.0	20,000	4,350	0.52
Soldier Pile Wall	0.45	0.8	20,000	10,220	0.39
Existing Soldier Pile Wall	0.6	2.5	20,000	4,350	0.52
Buttresses	0.6	0.7	-	31,426	0.6

Capping Beam and Waler Modelled as Beams				
	Cross Sectional Area (m^2)	*Moment of Inertia I_2 (m^4)*	*Moment of Inertia I_3 (m^4)*	*Elastic Modulus (E) (MPa)*
WB1 – 500WC340	0.043	2 x 10^{-3}	6.67 x 10^{-4}	200,000
WB2 – 400WC270	0.034	1.03 x 10^{-3}	3.42 x 10^{-4}	200,000
CB1 – 750mm x 500mm	0.375	7.81 x 10^{-3}	1.758 x 10^{-2}	20,000
CB2 – 650mm x 600mm	0.39	1.17 x 10^{-2}	1.373 x 10^{-2}	20,000
Existing Capping beam	0.77	7.76 x 10^{-2}	3.1 x 10^{-2}	20,000

Central Buttresses Modelled as Volumes				
	Width (m)	*Length (m)*	*Equivalent Elastic Modulus (E_{eq}) (MPa)*	Poisson's Ratio (ν)
Central Buttresses	0.6	4.5	31,426	0.2

The moduli adopted above are long term moduli.

4.5.4 Model Stages

The model was run through a number of stages in an attempt to simulate the existing conditions and the construction procedure. These are summarised below.

1. Initial phase.
2. Apply surcharge loads to Barrenjoey Road and the adjoining properties to the north, south and east.
3. Install soldier pile, secant pile wall, capping beam and walers.
4. Dewater and excavate to RL-1.0m.
5. Stop dewatering and allow the groundwater table to return to RL1m over the western side and RL3m over the western side.

Model displacements were reset to zero at the start of Stage 3. This allowed for separate assessments of the movements associated with the existing conditions and the construction itself.

While the walers will be installed progressively as the excavation progresses, the driving forces behind the wall result from a large wedge of rock sliding down on a 45° plane. Due to the strength of the bedrock the wedge of rock will generally span between the buttresses. As a result, the structural impact of not progressively installing the walers is anticipated to be limited. As a result, the staging of the installation of the walers has not been modelled

5 ANALYSIS AND RESULTS

Below we have provided the predicted displacements, design forces and bending moments. These design forces and bending moments are working loads and must be appropriately factored. The design forces and bending moments have been determined by van der Meer Consulting and are based on the modelling completed by JK Geotechnics.

The forces and bending moments reported are indicated by the numbers shown in subscript after the force/bending moment notation. These subscript numbers reference the x, y and z-axes. In this regard the axes represented are as follows:

- Subscript 1 refers to a horizontal axis parallel to the structural element,
- Subscript 2 refers to a vertical axis parallel to the structural element, and
- Subscript 3 refers to a horizontal axis perpendicular to the structural element.

The convention for compressive and tensile forces is that negative axial forces represent compressive loads while positive forces represent tensile loads. Appendix C1 presents a more detailed description of the referencing of the various forces and bending moments.

5.1 Wall Displacements

The table below presents the maximum inward displacements for the retaining walls. Figures 5 to 7 present a graphical illustration of these movements.

Maximum Horizontal, Vertical and Vector Displacements			
	Horizontal (mm)	Vertical (mm)	Vector (mm)
Eastern Wall	14.5	2.9	14.7
Western Wall	11.5	3.1	11.9
Southern Secant Pile Wall	2.1	2.0	2.8
Northern Secant pile Wall	1.8	2.5	3.0
Southern Buttress	1.5	1.8	2.3
Northern Buttress	0.9	2.2	2.3

5.2 Working Wall Forces and Bending Moments

The following tables summarise the design axial forces (N), shear forces (Q) and bending moments (M). All values are based on working (serviceability) load cases. It should be noted that the values reported below are the maximum calculated values. However, these values may not be representative of the actual forces experienced by the structures and references should be made to the height maps when deciding the magnitude of design forces and moments to adopt. These results are also presented in Figures 8 to 19.

Retaining Walls - Maximum/Minimum Axial and Shear Forces and Bending Moments			
	Axial Forces (kN/m)	Shear Forces (kN/m)	Bending Moments (kNm/m)
Eastern Wall	461.5	339.2	131.2
Western Wall	81.7	71.0	88.0
Southern Secant Pile Wall	271.1	68.8	53.4
Northern Secant pile Wall	161.1	64.0	32.5
Southern Buttress	732.4	615.8	378.1
Northern Buttress	366.4	297.9	121.0

Central Buttresses – Maximum Horizontal, Vertical and Vector Displacement		
Horizontal (mm)	Vertical (mm)	Vector (mm)
13.0	3.0	13.2

Central Buttresses - Maximum/Minimum Axial and Shear Forces and Bending Moments		
Axial Forces (kN/m)	Shear Forces (kN/m)	Bending Moments (kNm/m)
8794	3436	4850

The central buttresses have been modelled as volumes. To extract the forces and bending moments from these volumes, dummy plates have been modelled that are oriented parallel and perpendicular to the length of the volumes and run through the centroid of the volumes. These dummy plates have been modelled with a very low modulus and the same geometric shape as the buttress so that their inclusion in the model does not affect the results. The values extracted from the dummy plates have then scaled up to represent the design loads using the following relationships:

$N_{real} = N_{dummy} \times (EA)_{real}/(EA)_{dummy}$

$Q_{real} = Q_{dummy} \times (E)_{real}/(E)_{dummy}$

$M_{real} = M_{dummy} \times (EI)_{real}/(EI)_{dummy}$

Capping Beams and Waler Beams- Maximum/Minimum Axial and Shear Forces and Bending Moments			
	Axial Forces (kN/m)	Shear Forces (kN/m)	Bending Moments (kNm/m)
WB1 – 500WC340	113.0	106.5	122.1 (M_3) & 129.1 (M_1)
WB2 – 400WC270	89.0	56.5	58.0 (M_3) & 40.1 (M_1)
CB1 (south and north) – 750mm x 500mm	155.0	82.6	233.2 (M_3) & 22.7 (M_1)
CB1 (west) – 750mm x 500mm	76.4	90.3	239.3 (M_3) & 48.6 (M_1)
CB2 – 650 x 600	125.2	84.2	88.8 (M_3) & 24.1 (M_1)

5.3 Induced Movements Below Adjoining Properties

The proposed development will induce settlement below the adjoining properties to the north, south and east. The table below presents the expected maximum displacements induced below the adjoining properties. Figure 20 present heat maps indicating the predicted induced settlements. Where adjoining structures are well set back from the site and only undergo very minor induced settlements they have not been modelled or reported. Van der Meer Consulting must satisfy themselves that these settlements will not damage the adjoining building. If van der Meer Consulting is unable to satisfy themselves that damage will not result then underpinning or alternative stabilisation of the building should be carried out.

Maximum Predicted Displacements Below Adjoining Properties (mm)	
To North of Site - Barrenjoey House	1.7
To the East of the Site - 1110 Barrenjoey Road	7.1
To the South of the Site - 1100 Barrenjoey Road	1.2

5.5 Monitoring During Construction

Our model assumes that 45° joints dipping down to the west are present below the eastern retaining wall, as it is conceivable that such jointing may be present. However, it is much more unlikely that jointing dipping into the site (i.e. to the south and north) and impacting those small retaining wall sections that run along the eastern end of the northern and southern retaining walls will be present. Should it be present, the relatively short length of walls means that it is very unlikely that release planes running perpendicular the the retaining walls are present within these short wall distances and consequently, any adverse jointing will be buttressed by the eastern retaining wall. Consequently, jointing has not been explicitly modelled in these potions of the site. However, while unlikely it is possible that such jointing may be present.

While the material parameters adopted for these rock masses takes into account the presence of defects we recommend that during excavation the cut faces at the eastern ends of the northern and southern retaining walls be inspected by a geotechnical engineer every 1.5m to check for the presence of adversely orientated defects. In the unlikely event that such defects be present, the design will need to be checked to confirm that it will perform satisfactorily and allowance should be made for the installation of additional props, if necessary.

It is important that regular survey and inclinometer monitoring be completed during construction to confirm that measured deflections fall within the deflections predicted. In this regard, survey monitoring should be completed at least along the capping beam, common boundary and on the adjoining buildings. Monitoring of adjoining structures should not be limited to the closest wall to the site but should also monitor at points further back such that the magnitude of differential movements may be assessed.

Inclinometers should also be installed in the walls, which should be progressively monitored as the excavation deepens. In this regard the critical walls to monitor will be the northern and eastern walls. Ongoing monitoring of the groundwater table should also be completed.

A detailed inspection and monitoring plan will need to be developed prior to the commencement of construction. We would be pleased to assist in development of the inspection and monitoring plan and geotechnical inspections and monitoring during construction.

5.7 Comments

While we have provided the above design actions van der Meer Consulting must satisfy themselves that these design actions are reasonable. Similarly, they must confirm that the deflections of the wall and induced movements below adjoining structures will not result in damage to these buildings. Should this not be the case, underpinning or other stabilisation methods will be required.

Should the detailed design or the staging of construction be different to that modelled, we recommend that the model be updated and re-run to confirm that the design changes do not adversely impact other elements of the design and adjoining structures.

2 GENERAL COMMENTS

PLAXIS 3D has been used to model the proposed retention system at 1102 Barrenjoey Road, Palm Beach, NSW. Whilst efforts have been made to check the reasonableness of the report 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 one decimal place, it is unlikely the accuracy is to this order. Observation/measurement of displacement during the proposed construction should be used to verify the behaviour of the structures compared to these predictions, so that if the behaviour is at variance to the prediction, the reasons can be analysed and steps taken to protect structures and infrastructure.

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

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

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.

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

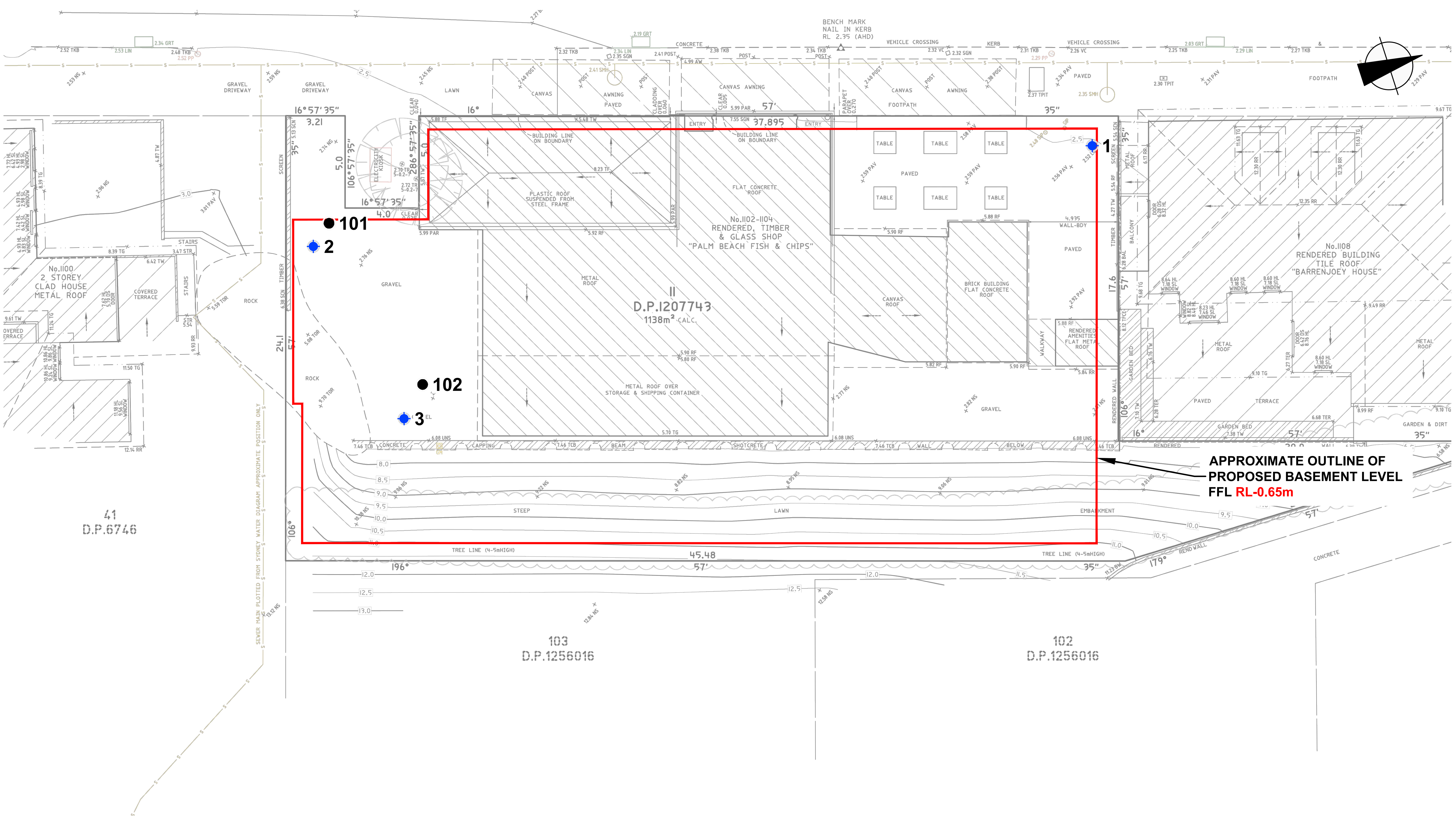
Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618Yrpt2 FEM

Figure No: 1

JKGeotechnics

PLOT DATE: 29/06/2023 1:51:43 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG



LEGEND

● BOREHOLE AND WELL (JK GEOTECHNICS, 2020)

⊕ BOREHOLE AND WELL (WITT CONSULTING, 2019)B

0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

Title:

PROPOSED DEVELOPMENT FOOTPRINT

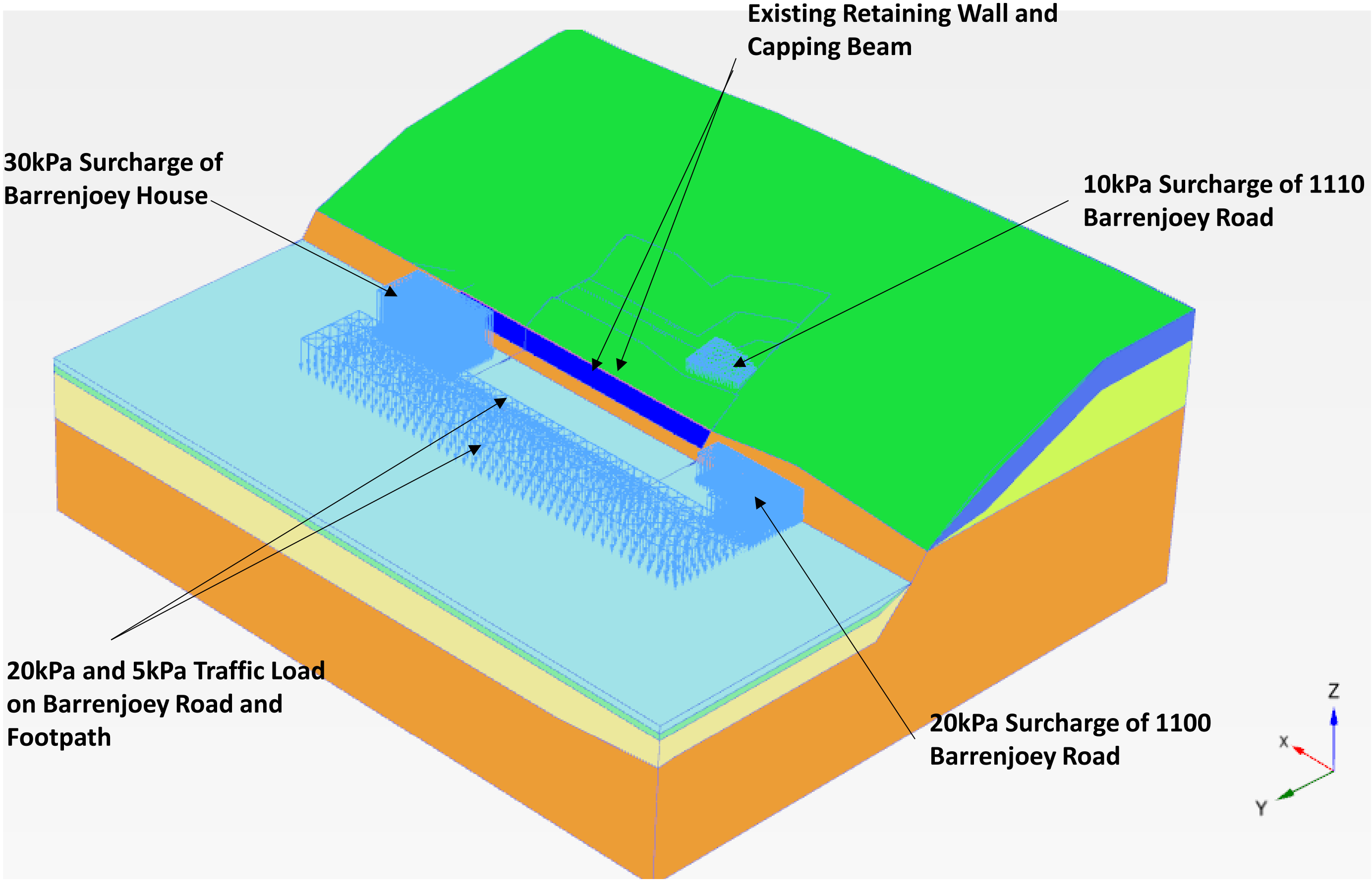
Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJrptrev4

Figure No: 2

JKGeotechnics

PLOT DATE: 25/03/2024 1:42:06 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG



Existing Site Conditions

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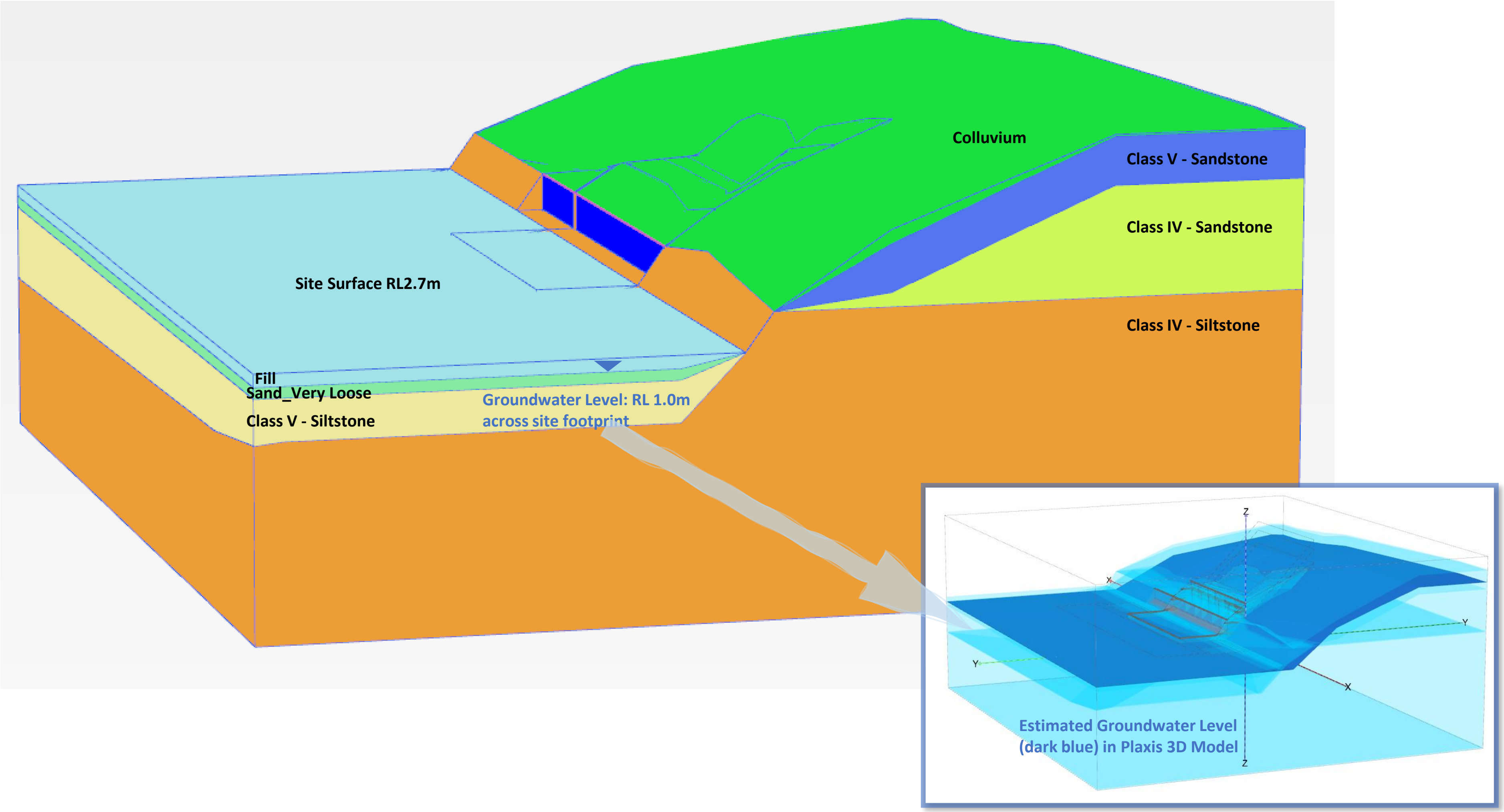
Report No. 33618Y

Figure No. 3



Assumed Topographical and Subsurface Profile

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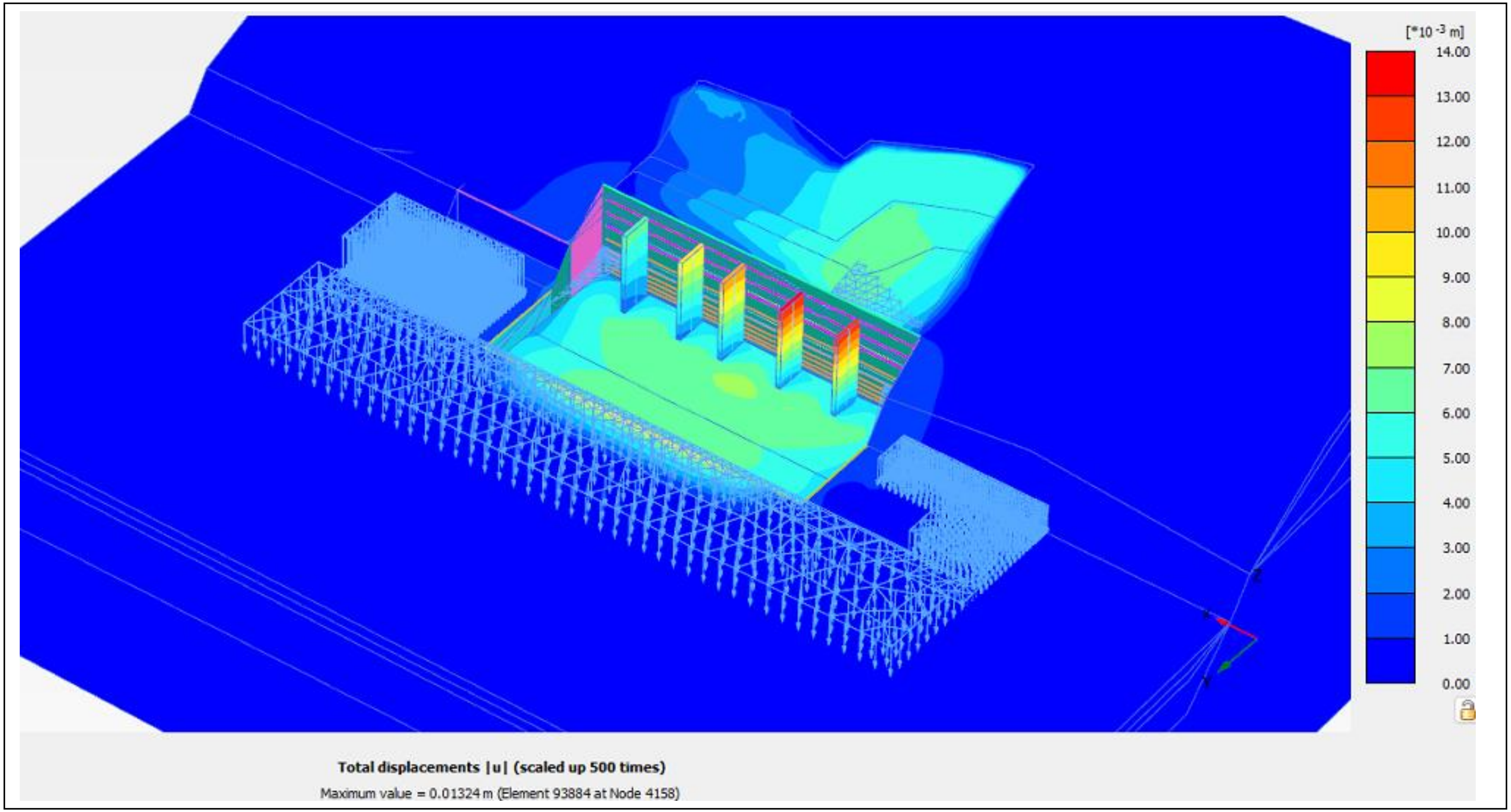
Report No. 33618Y

Figure No. 4



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01324 m (Element 93884 at Node 4158)

Maximum Displacements – Eastern and Northern Walls

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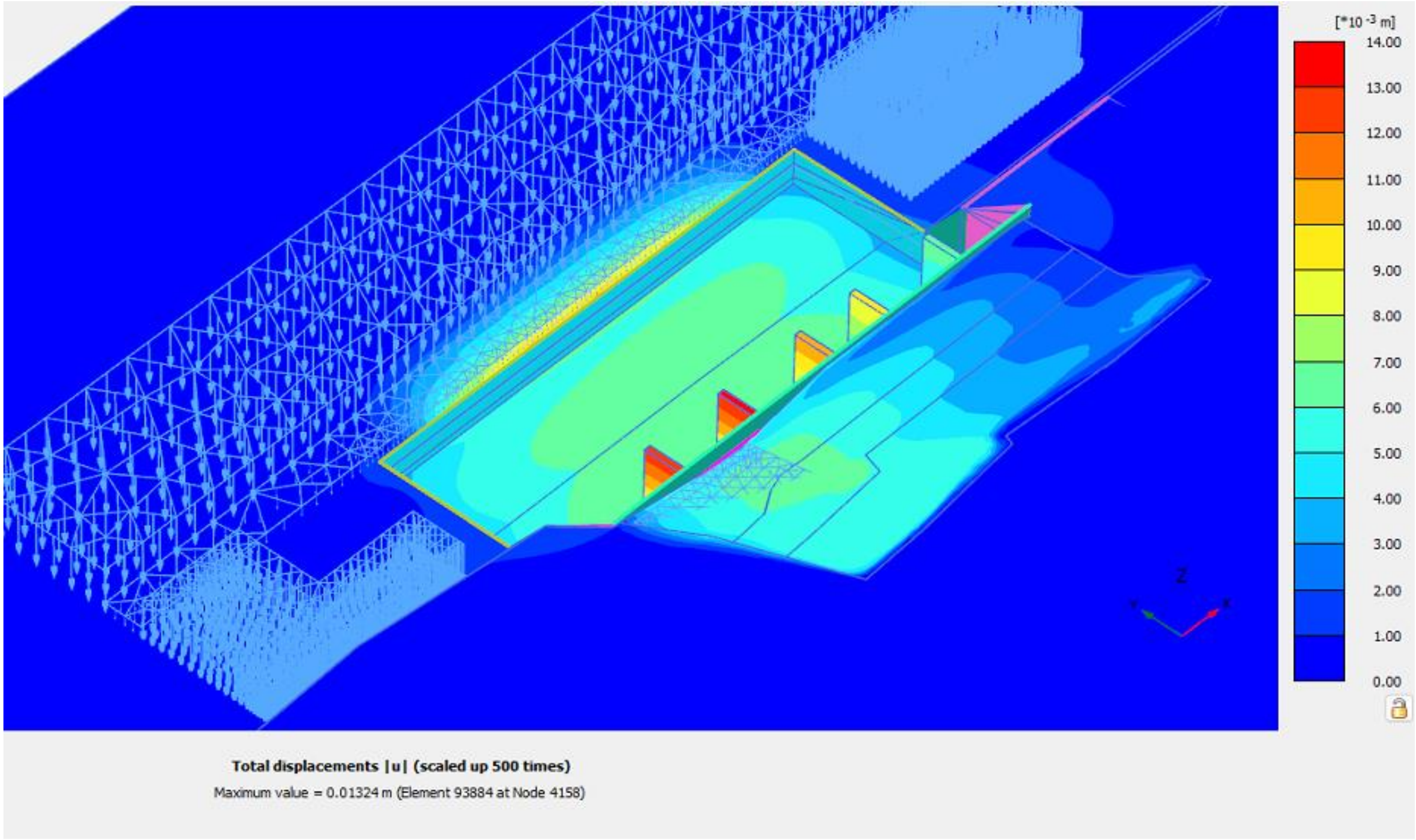
Report No. 33618Y Figure No. 5



Total displacements |u| (scaled up 500 times)

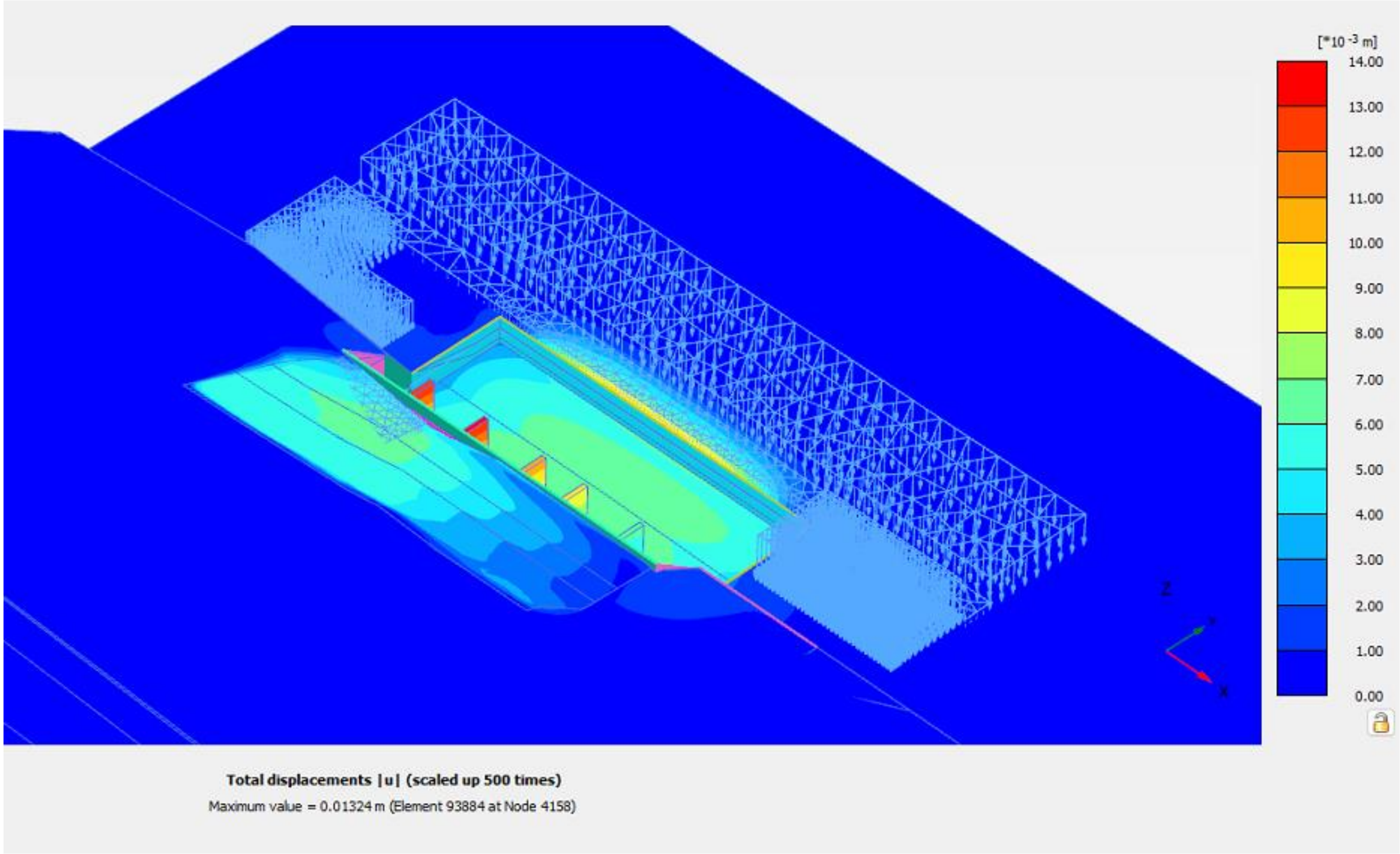
Maximum value = 0.01324 m (Element 93884 at Node 4158)

Maximum Displacements – Western and Northern Walls

JKGeotechnics

Report No. 33618Y

Figure No. 6



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01324 m (Element 93884 at Node 4158)

Maximum Displacements – Western and Southern Walls

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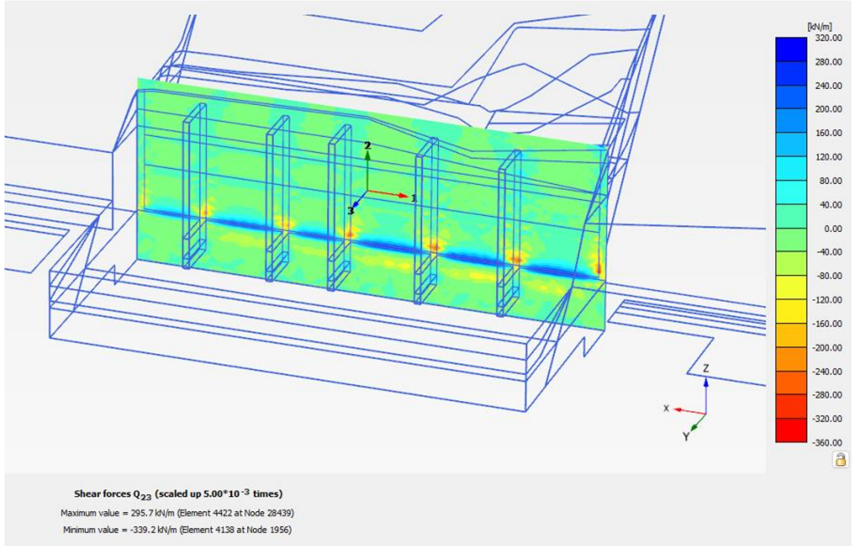
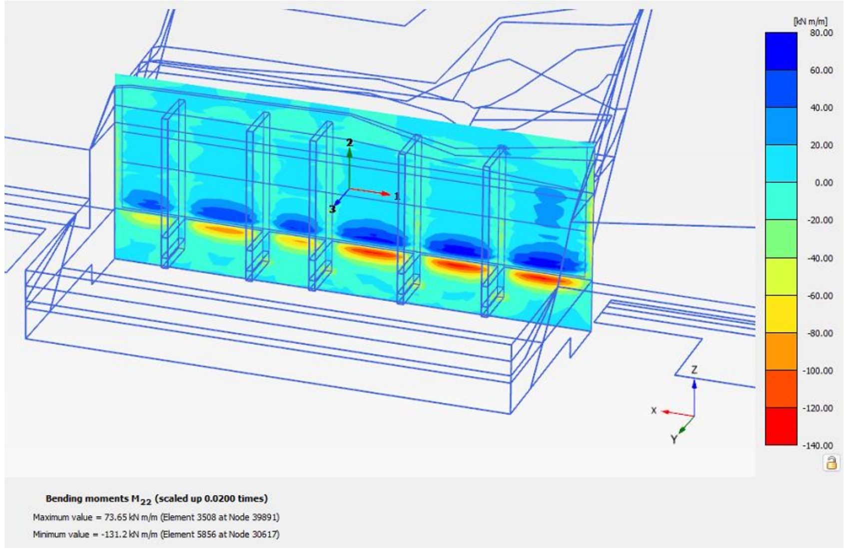
Report No. 33618Y

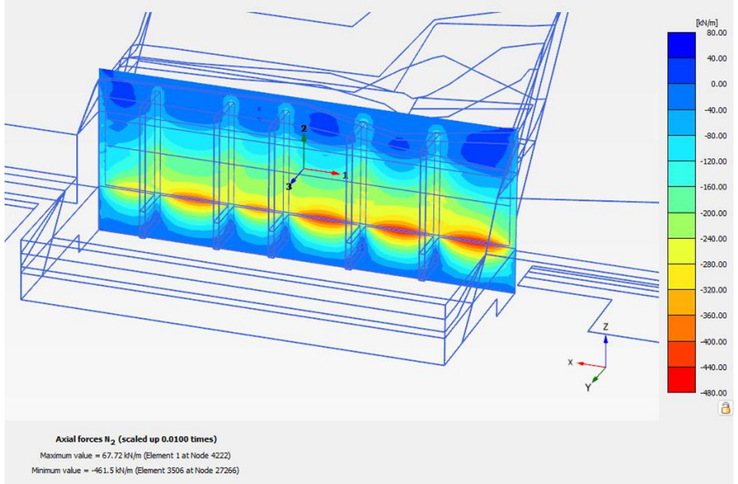
Figure No. 7



Shear forces Q_{23} (scaled up $5.00*10^{-3}$ times)
Maximum value = 295.7 kN/m (Element 4422 at Node 28439)
Minimum value = -339.2 kN/m (Element 4138 at Node 1956)



Bending moments M_{22} (scaled up 0.0200 times)
Maximum value = 73.65 kN m/m (Element 3508 at Node 39891)
Minimum value = -131.2 kN m/m (Element 5856 at Node 30617)



Axial forces N_2 (scaled up 0.0100 times)
Maximum value = 67.72 kN/m (Element 1 at Node 4222)
Minimum value = -461.5 kN/m (Element 3506 at Node 27266)

Axial and Shear Forces and Bending Moments – Eastern Wall

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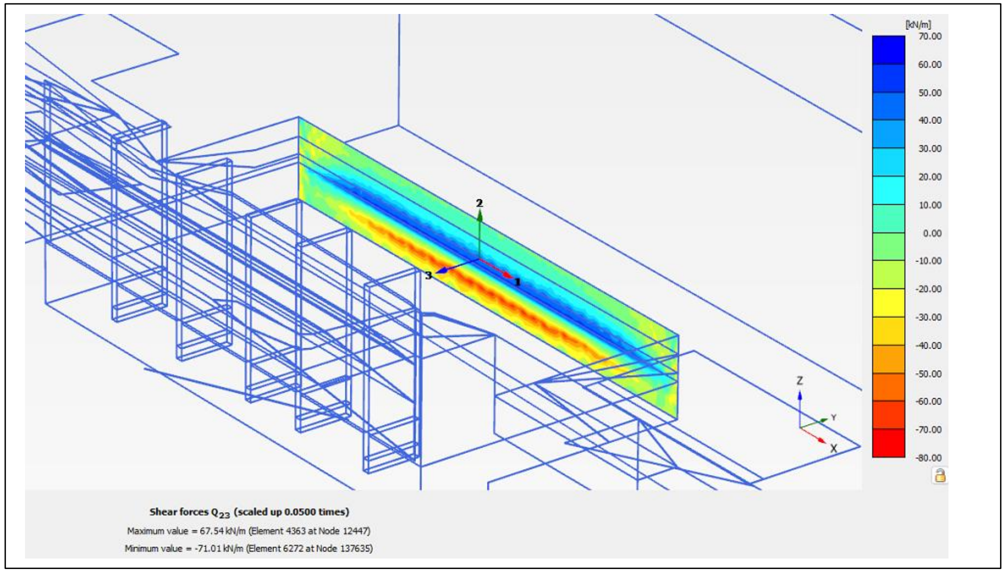
Report No. 33618Y

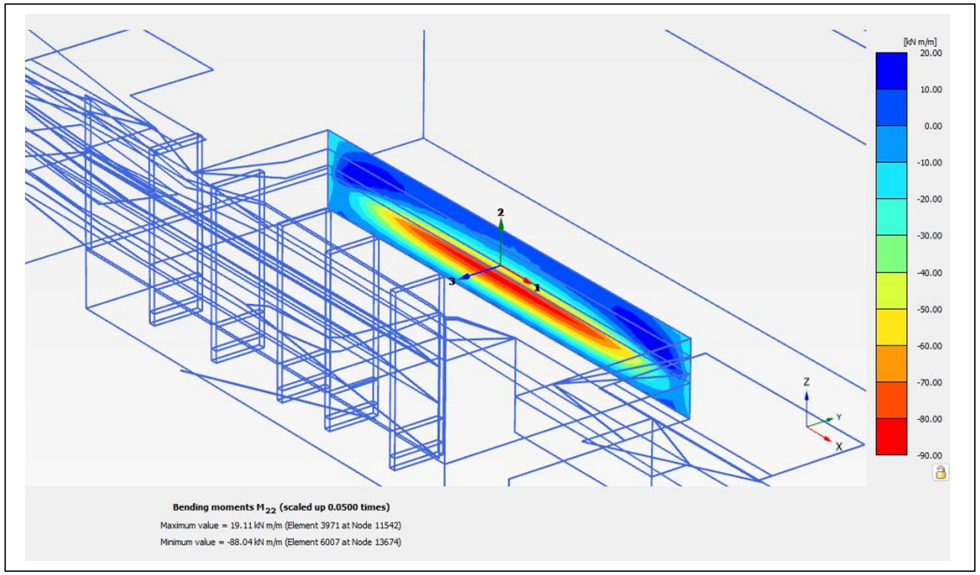
Figure No. 8



Shear forces Q_{23} (scaled up 0.0500 times)

Maximum value = 67.54 kN/m (Element 4363 at Node 12447)

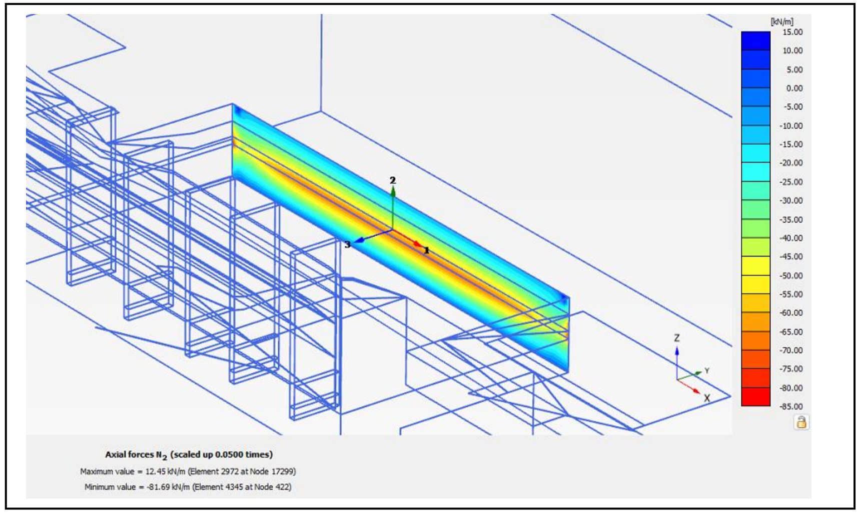
Minimum value = -71.01 kN/m (Element 6272 at Node 137635)



Bending moments M_{22} (scaled up 0.0500 times)

Maximum value = 19.11 kN m/m (Element 3971 at Node 11542)

Minimum value = -88.04 kN m/m (Element 6007 at Node 13674)



Axial forces N_2 (scaled up 0.0500 times)

Maximum value = 12.45 kN/m (Element 2972 at Node 17299)

Minimum value = -81.69 kN/m (Element 4345 at Node 422)

Axial and Shear Forces and Bending Moments – Western Wall

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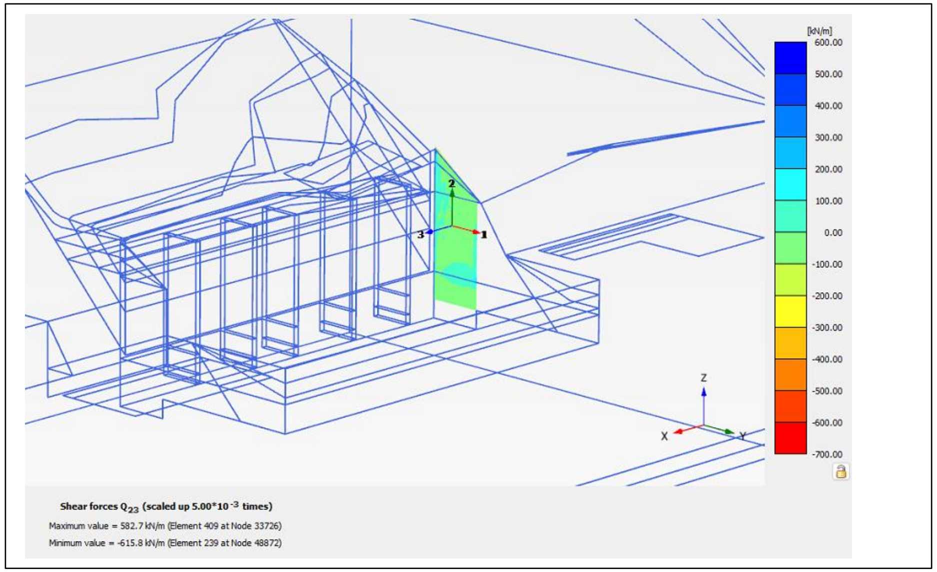
Report No. 33618Y

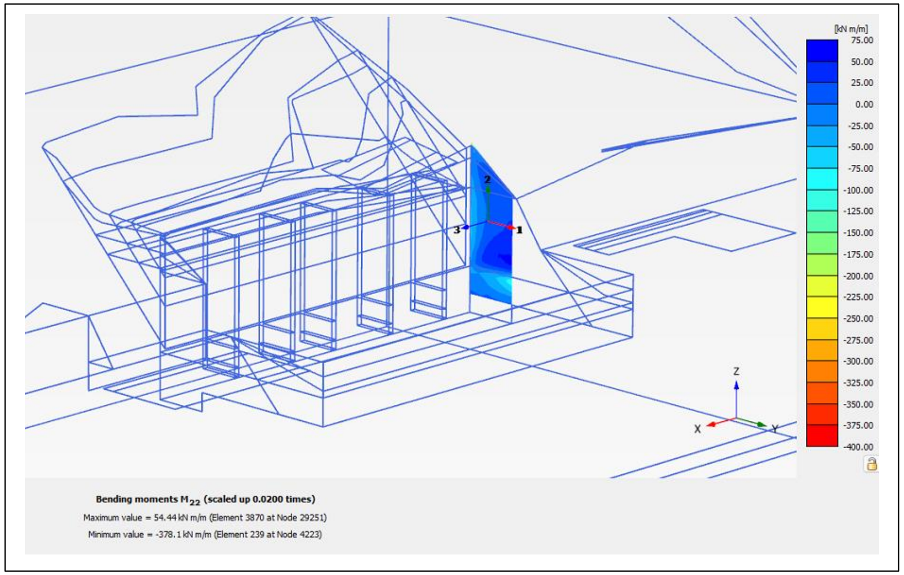
Figure No. 9



Shear forces Q_{23} (scaled up $5.00*10^{-3}$ times)

Maximum value = 582.7 kN/m (Element 409 at Node 33726)

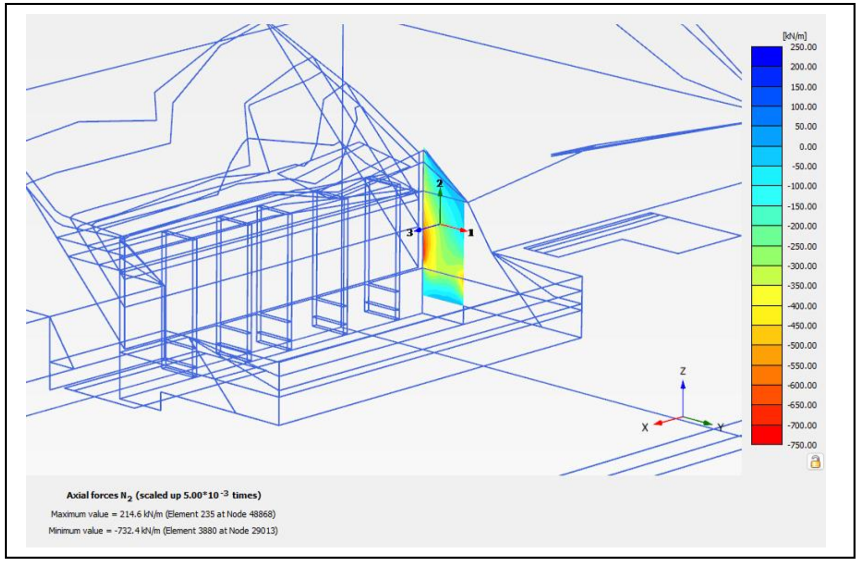
Minimum value = -615.8 kN/m (Element 239 at Node 48872)



Bending moments M_{22} (scaled up 0.0200 times)

Maximum value = 54.44 kN m/m (Element 3870 at Node 29251)

Minimum value = -378.1 kN m/m (Element 239 at Node 4223)



Axial forces N_2 (scaled up $5.00*10^{-3}$ times)

Maximum value = 214.6 kN/m (Element 235 at Node 48868)

Minimum value = -732.4 kN/m (Element 3880 at Node 29013)

Axial and Shear Forces and Bending Moments – Southern Buttress Wall

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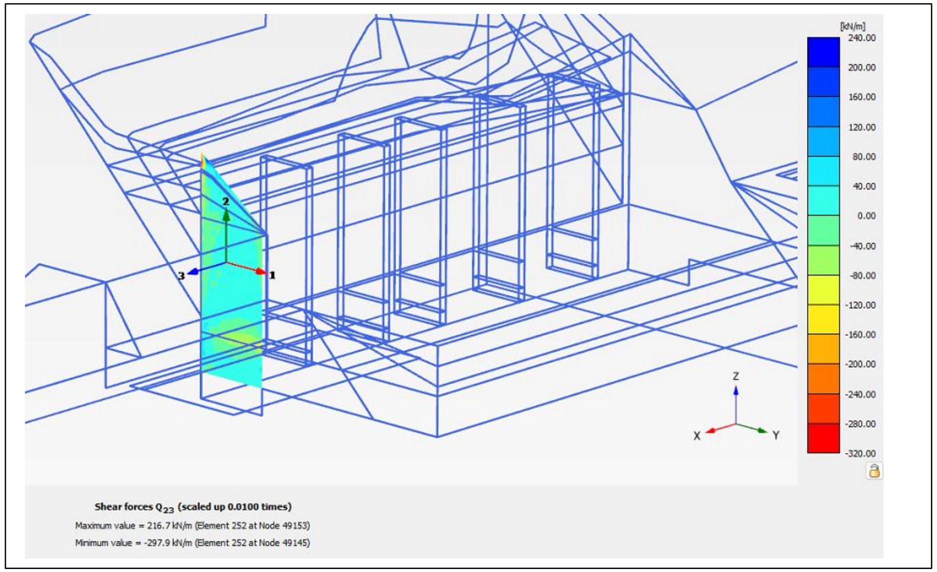
Report No. 33618Y

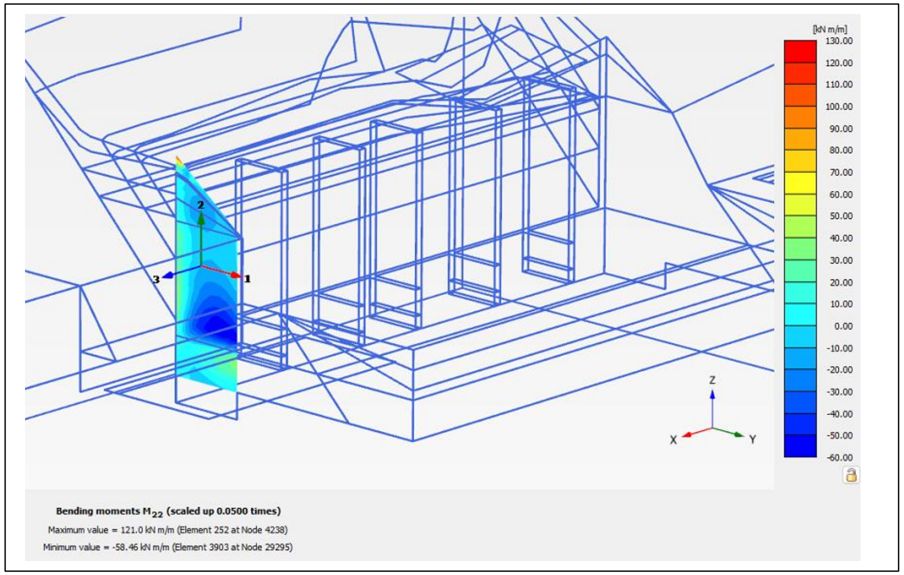
Figure No. 10



Shear forces Q_{23} (scaled up 0.0100 times)

Maximum value = 216.7 kN/m (Element 252 at Node 49153)

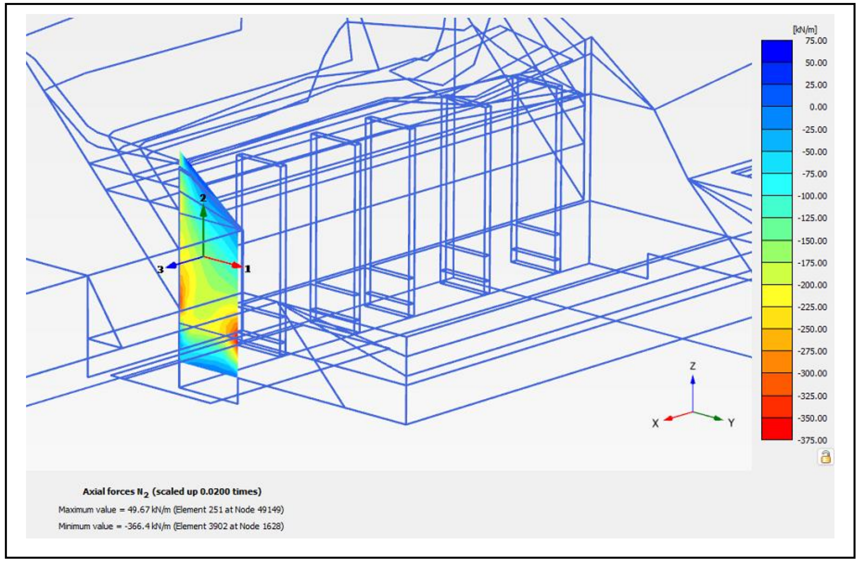
Minimum value = -297.9 kN/m (Element 252 at Node 49145)



Bending moments M_{22} (scaled up 0.0500 times)

Maximum value = 121.0 kN m/m (Element 252 at Node 4238)

Minimum value = -58.46 kN m/m (Element 3903 at Node 29295)



Axial forces N_2 (scaled up 0.0200 times)

Maximum value = 49.67 kN/m (Element 251 at Node 49149)

Minimum value = -366.4 kN/m (Element 3902 at Node 1628)

Axial and Shear Forces and Bending Moments – Northern Buttress Wall

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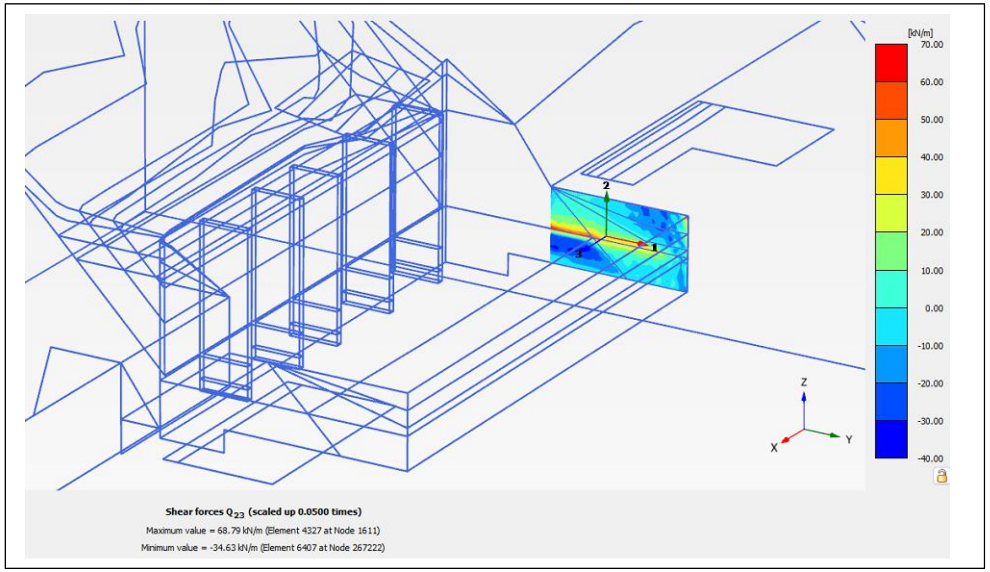
Report No. 33618Y

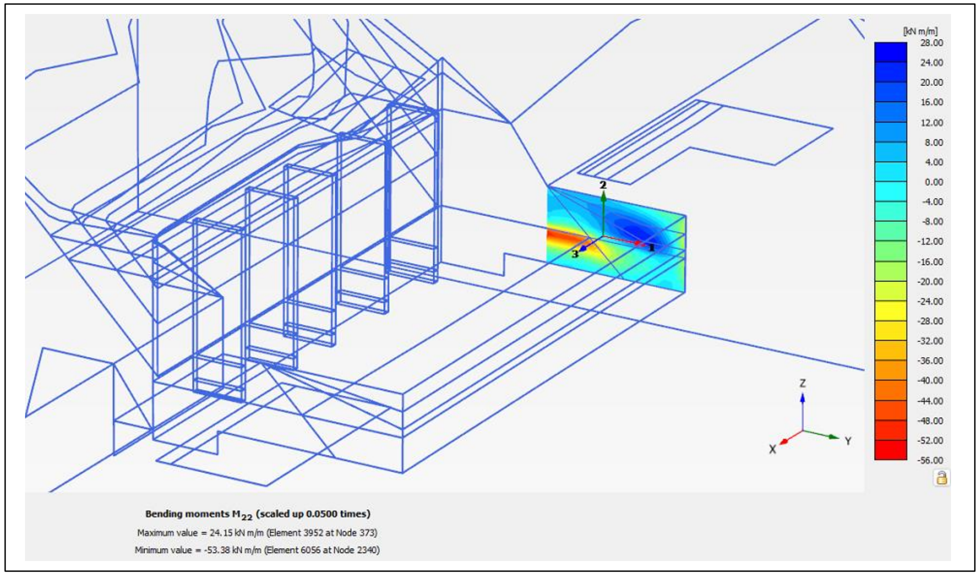
Figure No. 11



Shear forces Q_{23} (scaled up 0.0500 times)

Maximum value = 68.79 kN/m (Element 4327 at Node 1611)

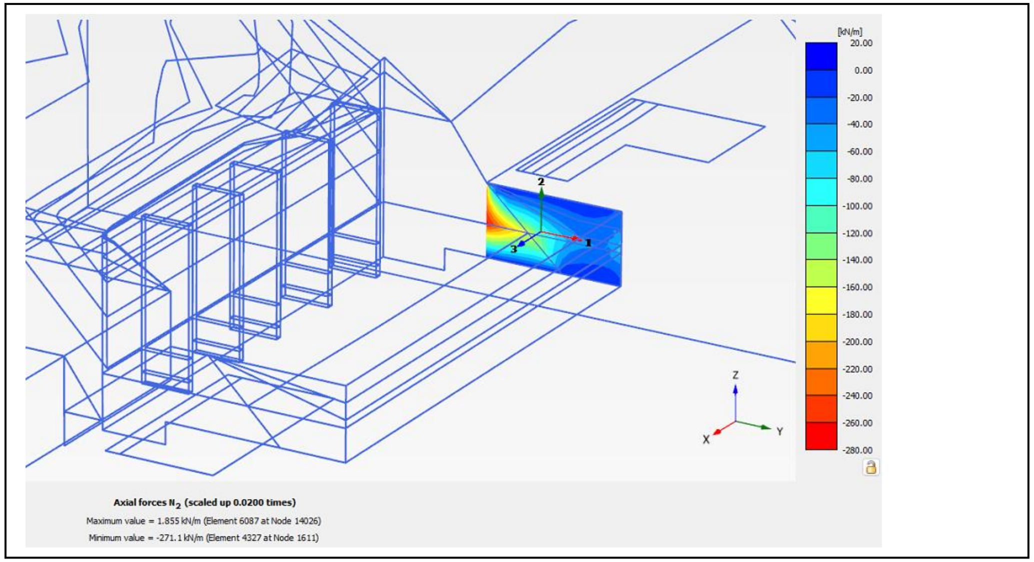
Minimum value = -34.63 kN/m (Element 6407 at Node 267222)



Bending moments M_{22} (scaled up 0.0500 times)

Maximum value = 24.15 kN m/m (Element 3952 at Node 373)

Minimum value = -53.38 kN m/m (Element 6056 at Node 2340)



Axial forces N_2 (scaled up 0.0200 times)

Maximum value = 1.855 kN/m (Element 6087 at Node 14026)

Minimum value = -271.1 kN/m (Element 4327 at Node 1611)

Axial and Shear Forces and Bending Moments – Southern Wall

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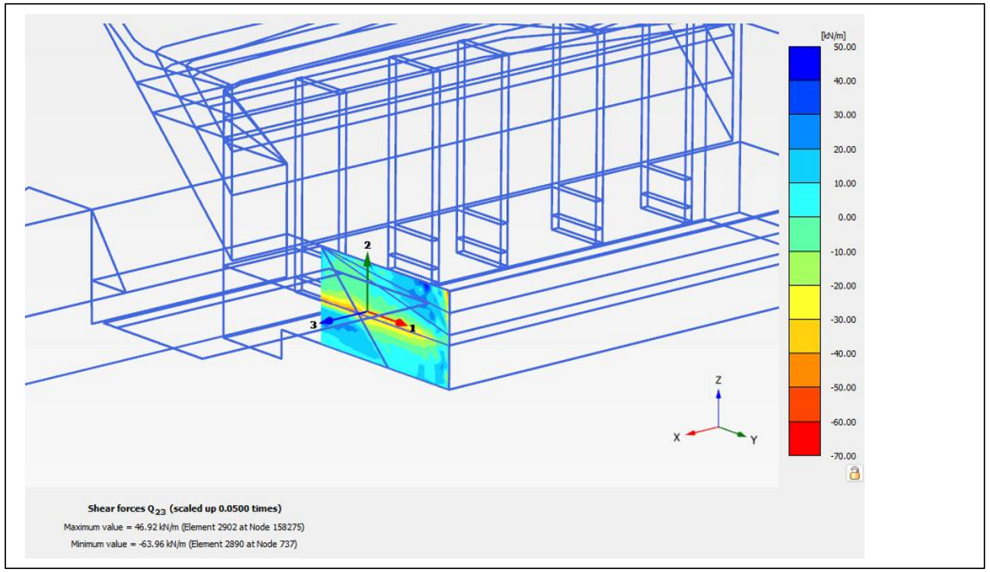
Report No. 33618Y

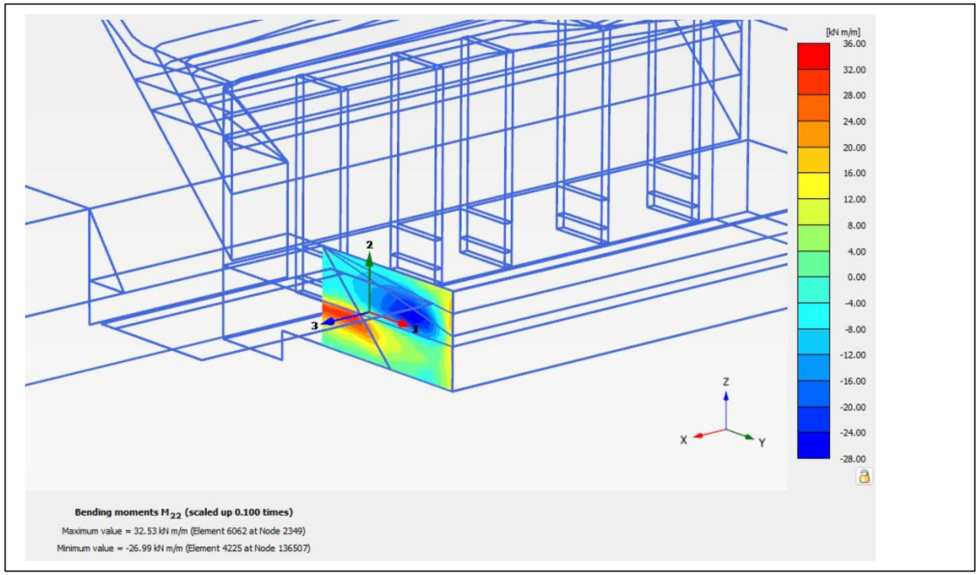
Figure No. 12



Shear forces Q_{23} (scaled up 0.0500 times)

Maximum value = 46.92 kN/m (Element 2902 at Node 158275)

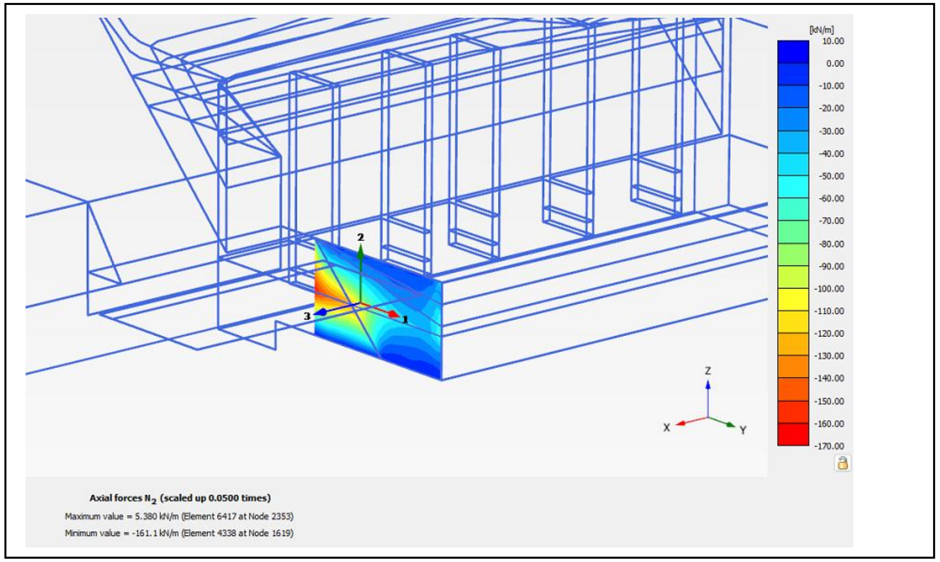
Minimum value = -63.96 kN/m (Element 2890 at Node 737)



Bending moments M_{22} (scaled up 0.100 times)

Maximum value = 32.53 kN m/m (Element 6062 at Node 2349)

Minimum value = -26.99 kN m/m (Element 4225 at Node 136507)



Axial forces N_2 (scaled up 0.0500 times)

Maximum value = 5.380 kN/m (Element 6417 at Node 2353)

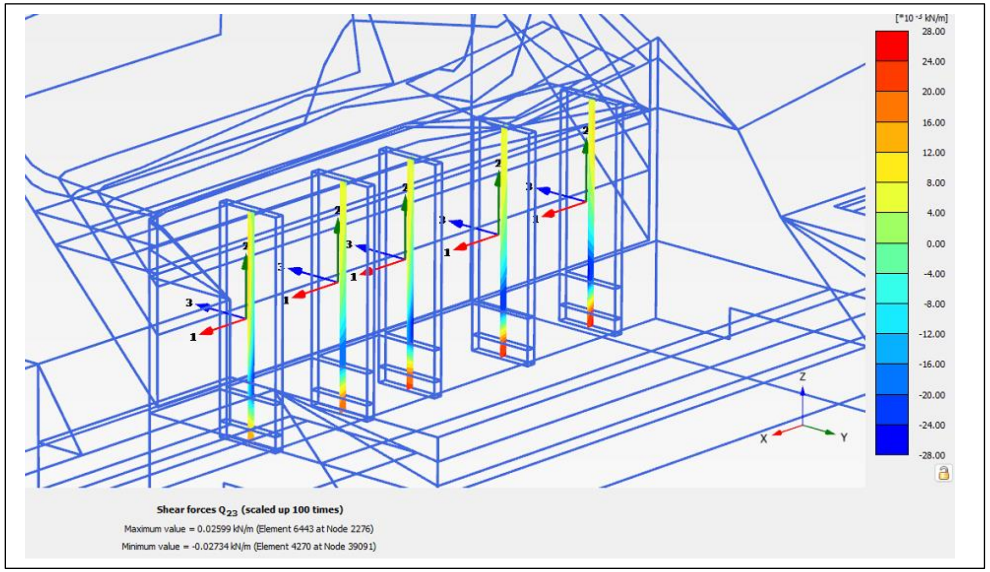
Minimum value = -161.1 kN/m (Element 4338 at Node 1619)

Axial and Shear Forces and Bending Moments – Northern Wall

JKGeotechnics

Report No. 33618Y

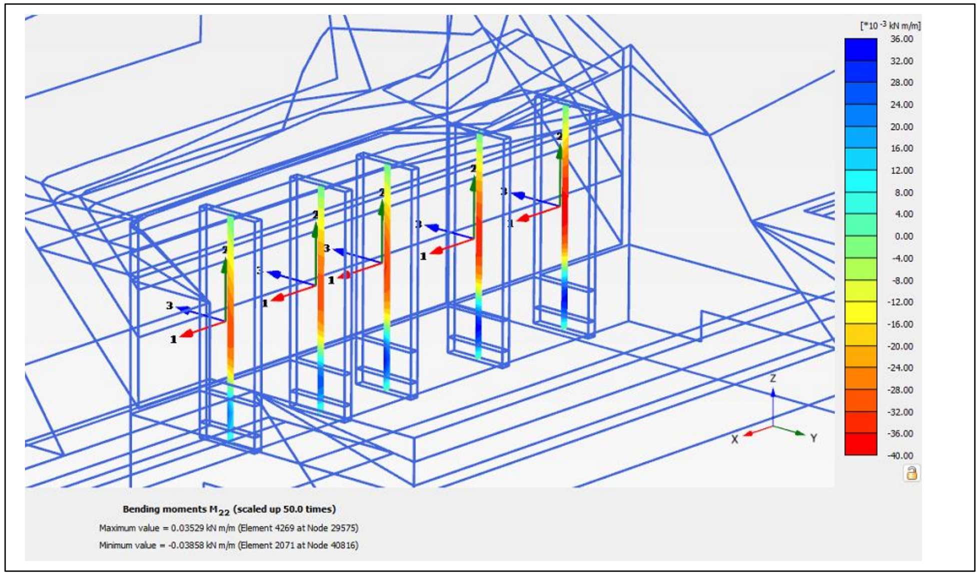
Figure No. 13



Shear forces Q_{23} (scaled up 100 times)

Maximum value = 0.02599 kN/m (Element 6443 at Node 2276)

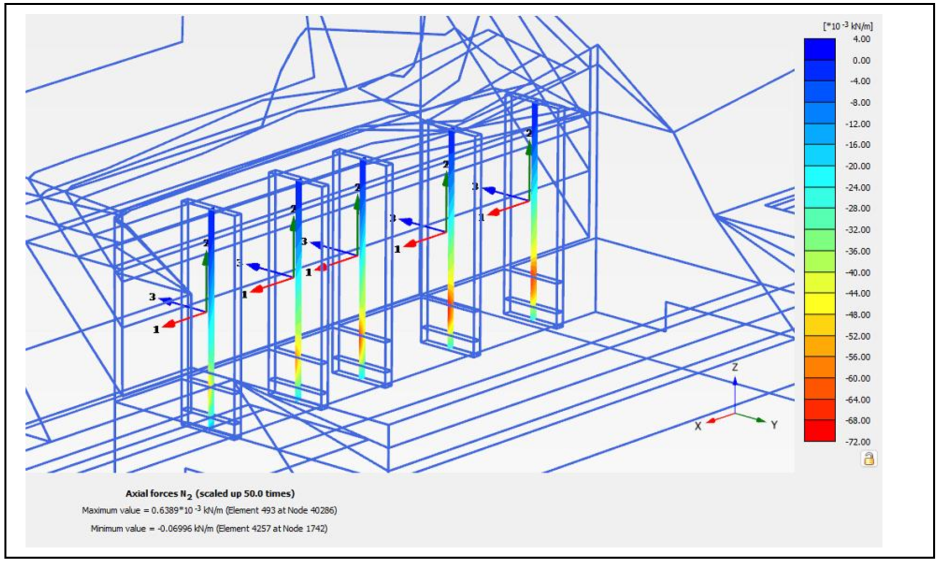
Minimum value = -0.02734 kN/m (Element 4270 at Node 39091)



Bending moments M_{22} (scaled up 50.0 times)

Maximum value = 0.03529 kN m/m (Element 4269 at Node 29575)

Minimum value = -0.03858 kN m/m (Element 2071 at Node 40816)



Axial forces N_2 (scaled up 50.0 times)

Maximum value = 0.6389*10 -3 kN/m (Element 493 at Node 40286)

Minimum value = -0.06996 kN/m (Element 4257 at Node 1742)

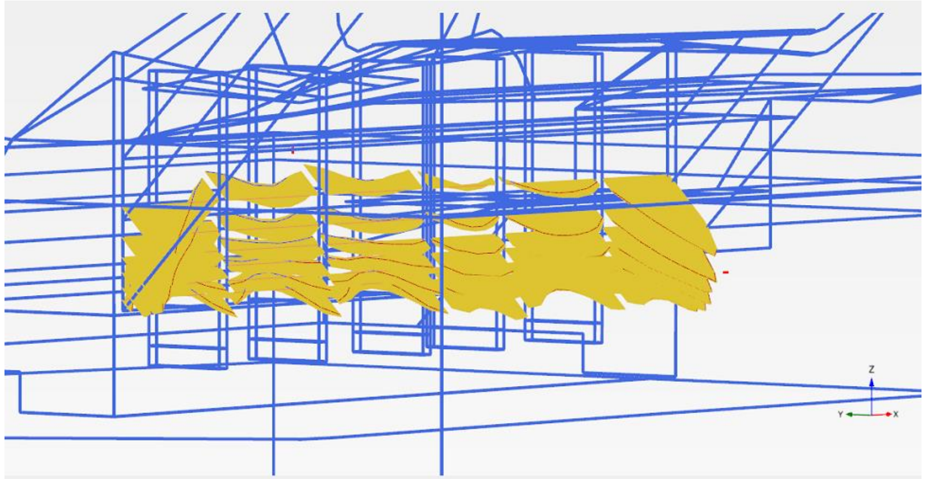
Forces and Moments in Dummy Plates of Central Buttresses

(Values have been scaled up using the ratio of $E_{real}/E_{dummy\ plate}$)

JKGeotechnics

Report No. 33618Y

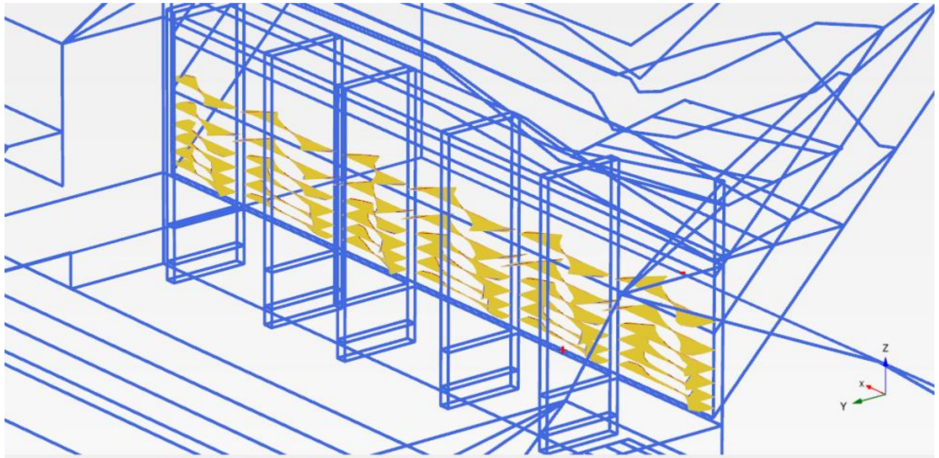
Figure No. 14



Envelope of Axial forces N (scaled up 0.0500 times)

Maximum value = 41.63 kN (Element 204 at Node 3553)

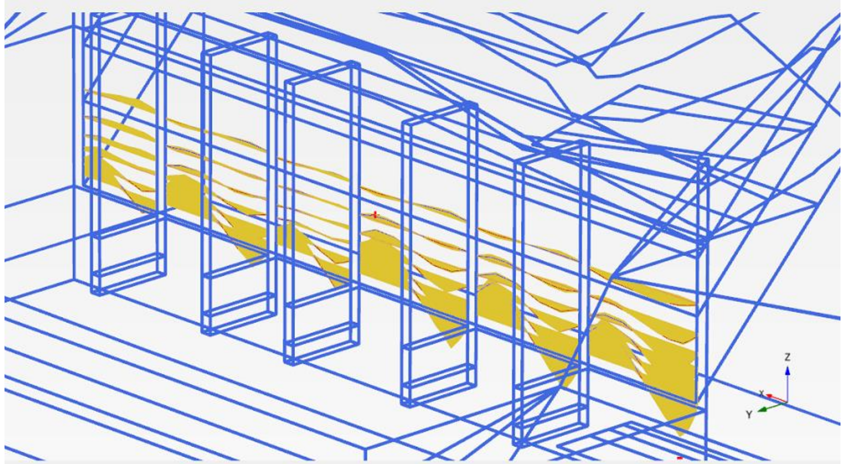
Minimum value = -113.0 kN (Element 222 at Node 43320)



Envelope of Shear forces Q_{12} (scaled up 0.0200 times)

Maximum value = 101.0 kN (Element 363 at Node 3249)

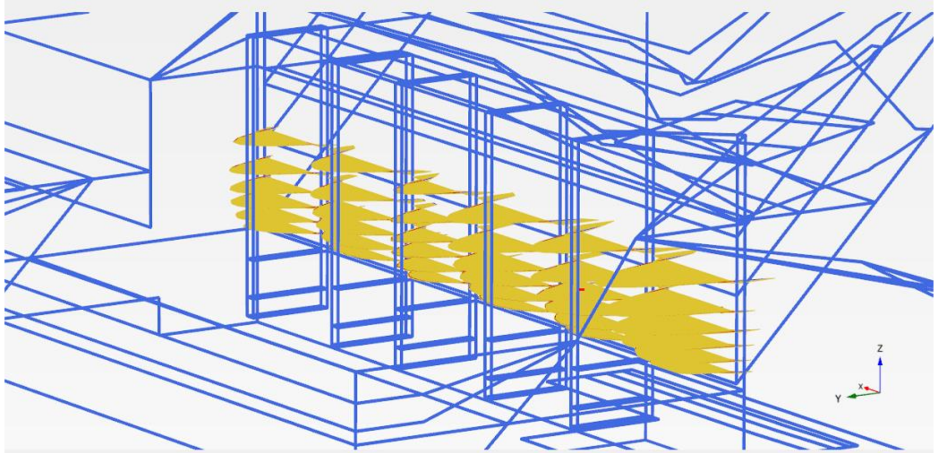
Minimum value = -106.5 kN (Element 197 at Node 41946)



Envelope of Torsion moments M_1 (scaled up 0.0200 times)

Maximum value = 116.6 kN m (Element 443 at Node 39654)

Minimum value = -129.1 kN m (Element 424 at Node 39900)



Envelope of Bending moments M_3 (scaled up 0.0500 times)

Maximum value = 62.11 kN m (Element 362 at Node 3268)

Minimum value = -122.1 kN m (Element 195 at Node 41970)

Forces and Moment - WB1

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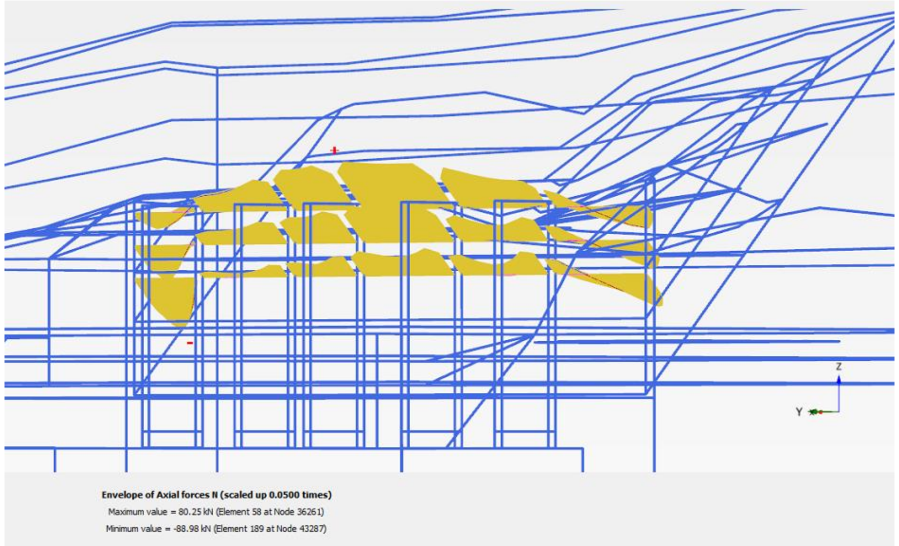
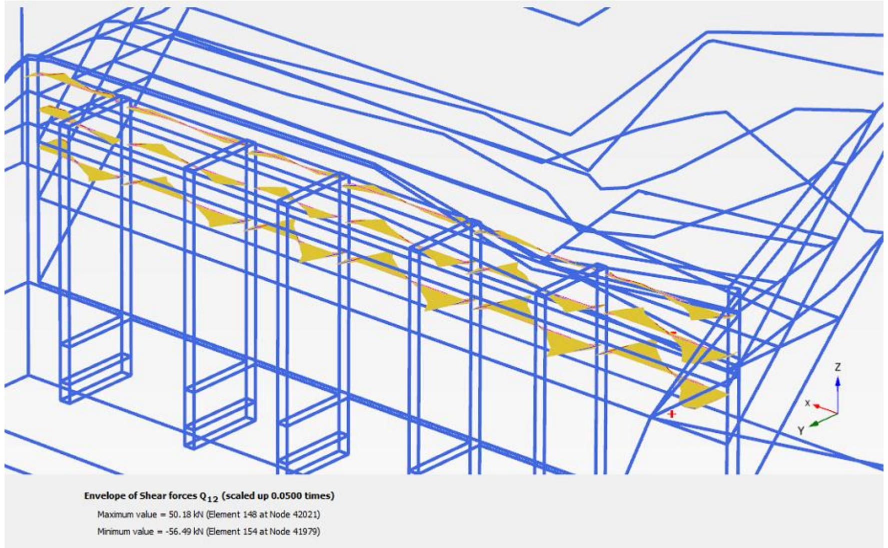
Report No. 33618Y

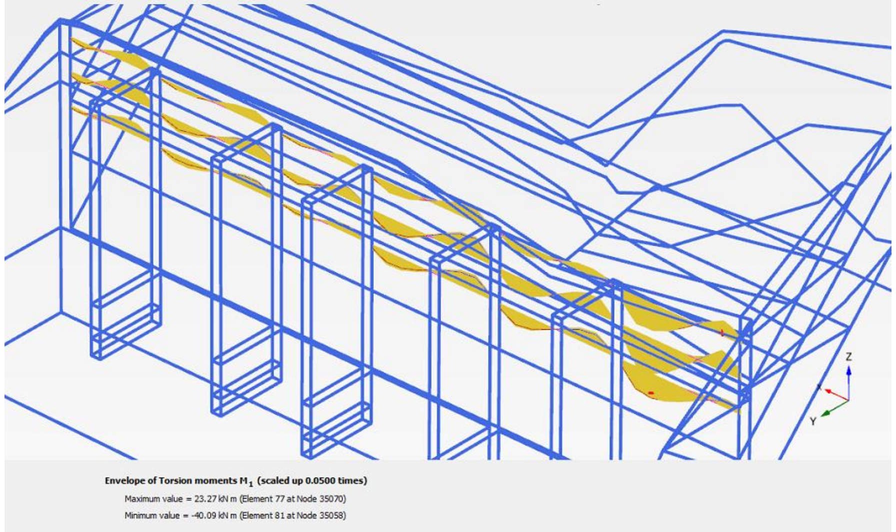
Figure No. 15



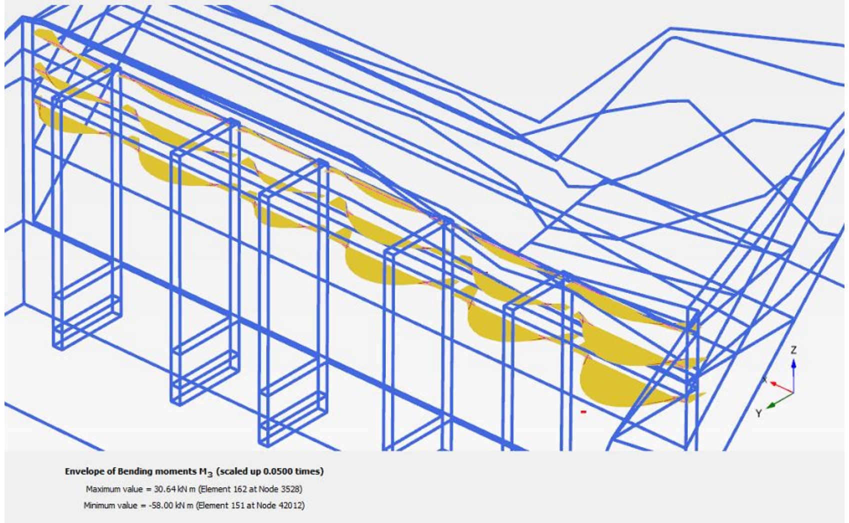
Envelope of Axial forces N (scaled up 0.0500 times)
Maximum value = 80.25 kN (Element 58 at Node 36261)
Minimum value = -88.98 kN (Element 189 at Node 43287)



Envelope of Shear forces Q_{12} (scaled up 0.0500 times)
Maximum value = 50.18 kN (Element 148 at Node 42021)
Minimum value = -56.49 kN (Element 154 at Node 41979)



Envelope of Torsion moments M_1 (scaled up 0.0500 times)
Maximum value = 23.27 kN m (Element 77 at Node 35070)
Minimum value = -40.09 kN m (Element 81 at Node 35058)



Envelope of Bending moments M_3 (scaled up 0.0500 times)
Maximum value = 30.64 kN m (Element 162 at Node 3528)
Minimum value = -58.00 kN m (Element 151 at Node 42012)

Forces and Moment – WB2

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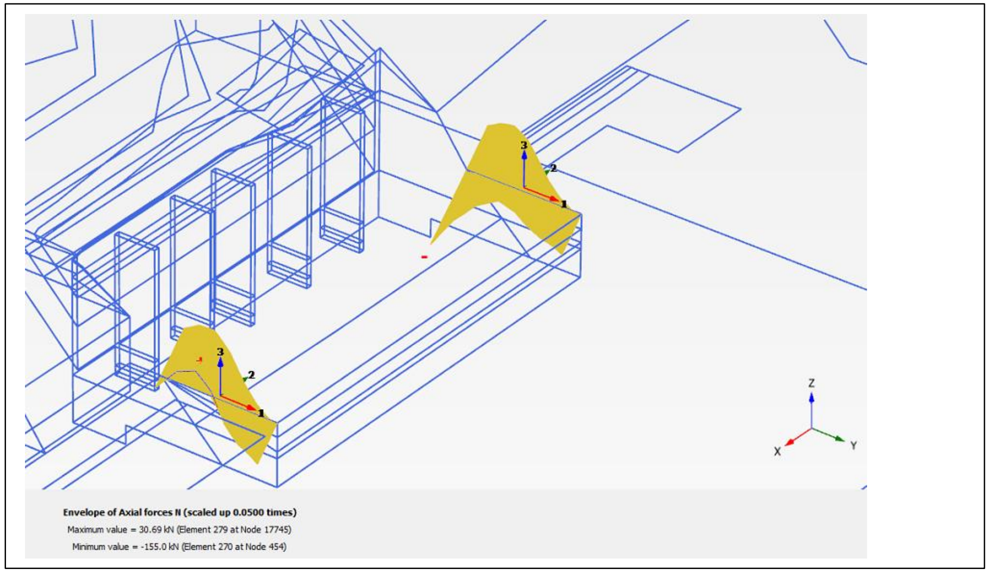
Report No. 33618Y

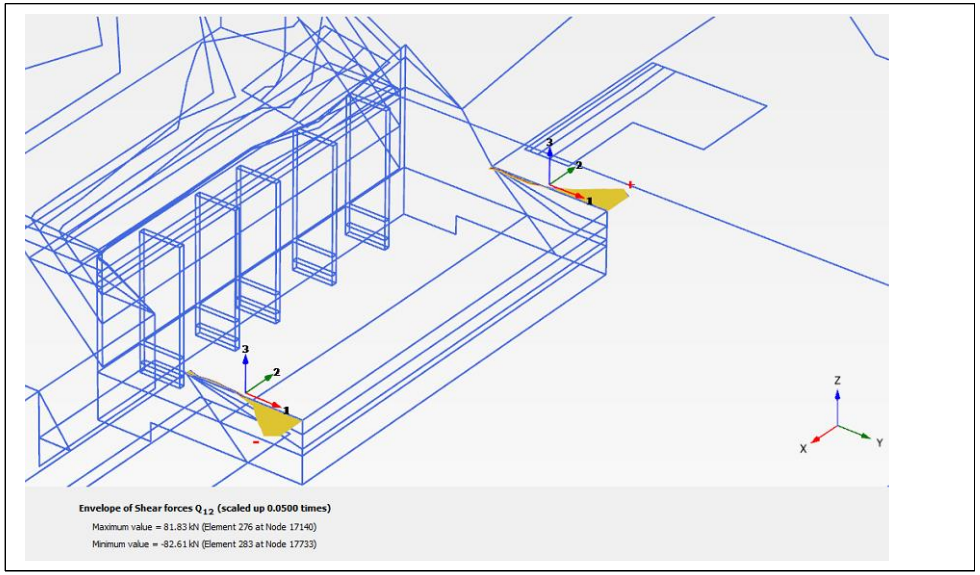
Figure No. 16



Envelope of Axial forces N (scaled up 0.0500 times)

Maximum value = 30.69 kN (Element 279 at Node 17745)

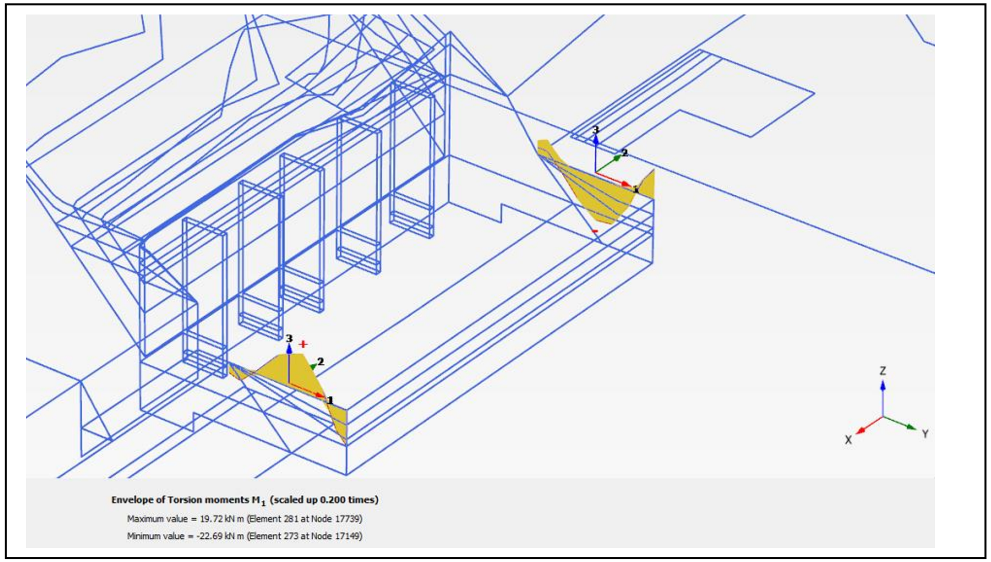
Minimum value = -155.0 kN (Element 270 at Node 454)



Envelope of Shear forces Q_{12} (scaled up 0.0500 times)

Maximum value = 81.83 kN (Element 276 at Node 17140)

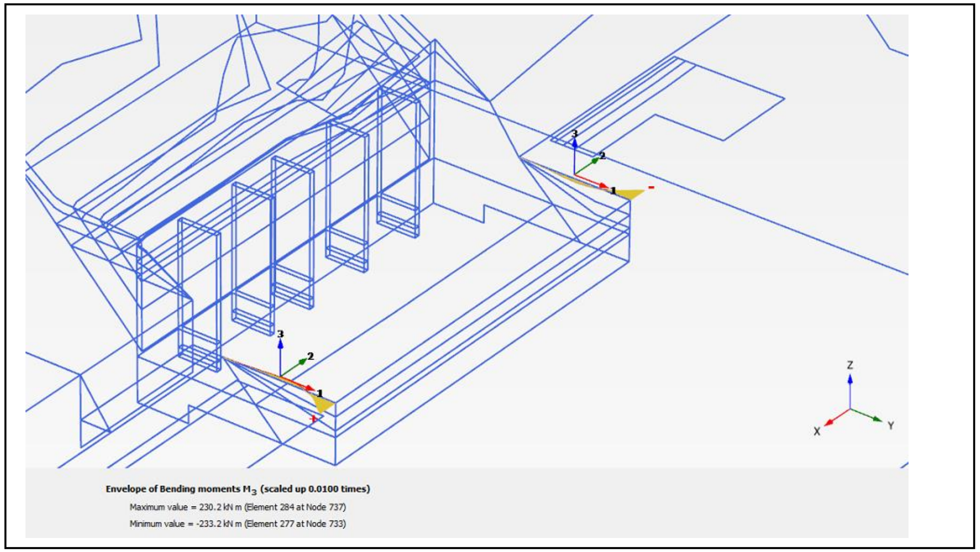
Minimum value = -82.61 kN (Element 283 at Node 17733)



Envelope of Torsion moments M_1 (scaled up 0.200 times)

Maximum value = 19.72 kN m (Element 281 at Node 17739)

Minimum value = -22.69 kN m (Element 273 at Node 17149)



Envelope of Bending moments M_3 (scaled up 0.0100 times)

Maximum value = 230.2 kN m (Element 284 at Node 737)

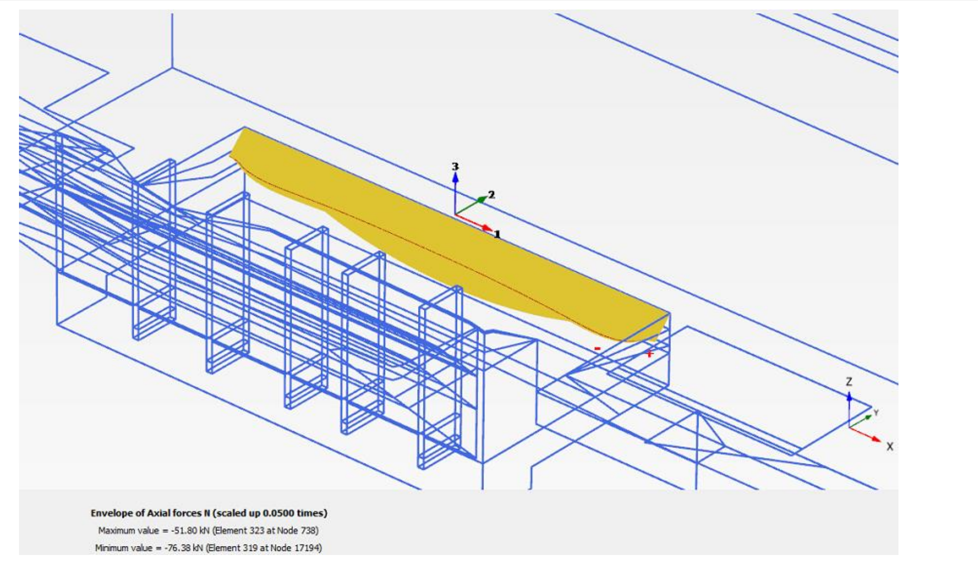
Minimum value = -233.2 kN m (Element 277 at Node 733)

Forces and Moment – Southern and Northern CB1

JKGeotechnics

Report No. 33618Y

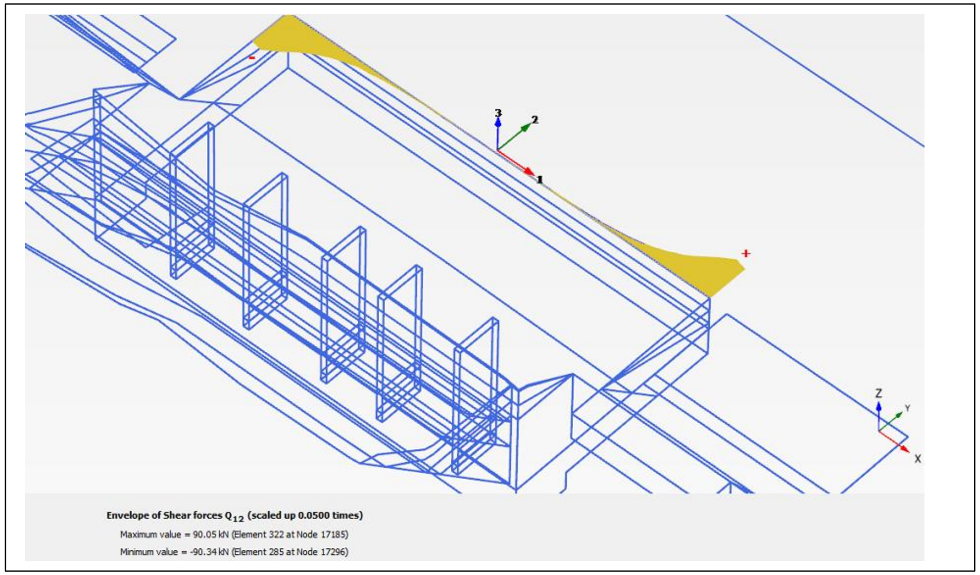
Figure No. 17



Envelope of Axial forces N (scaled up 0.0500 times)

Maximum value = -51.80 kN (Element 323 at Node 738)

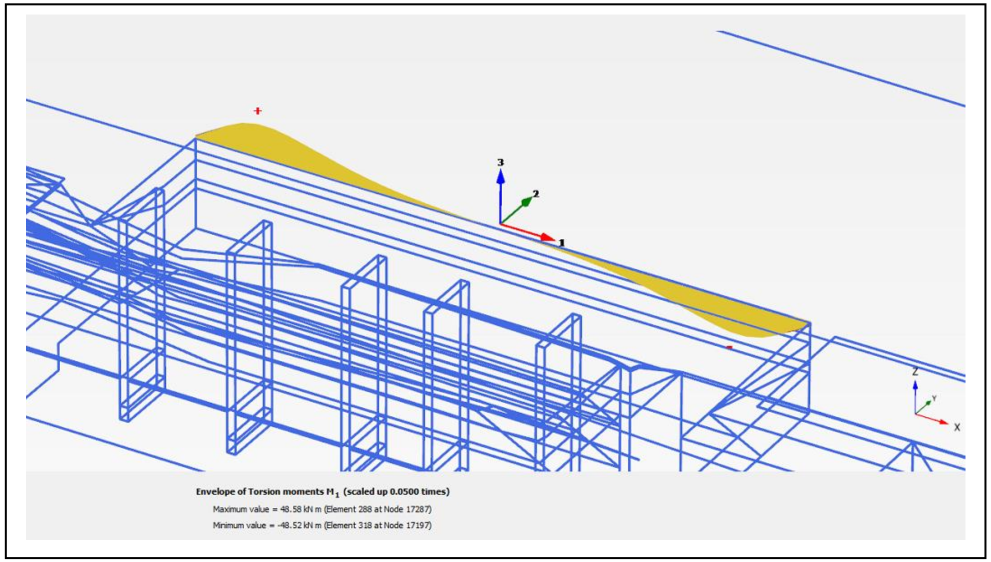
Minimum value = -76.38 kN (Element 319 at Node 17194)



Envelope of Shear forces Q_{12} (scaled up 0.0500 times)

Maximum value = 90.05 kN (Element 322 at Node 17185)

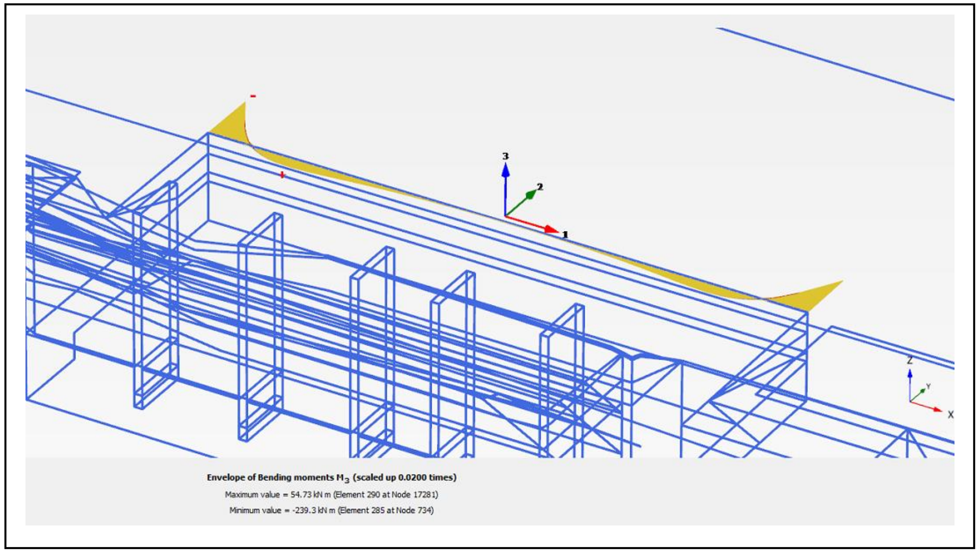
Minimum value = -90.34 kN (Element 285 at Node 17296)



Envelope of Torsion moments M_1 (scaled up 0.0500 times)

Maximum value = 48.58 kN m (Element 288 at Node 17287)

Minimum value = -48.52 kN m (Element 318 at Node 17197)



Envelope of Bending moments M_3 (scaled up 0.0200 times)

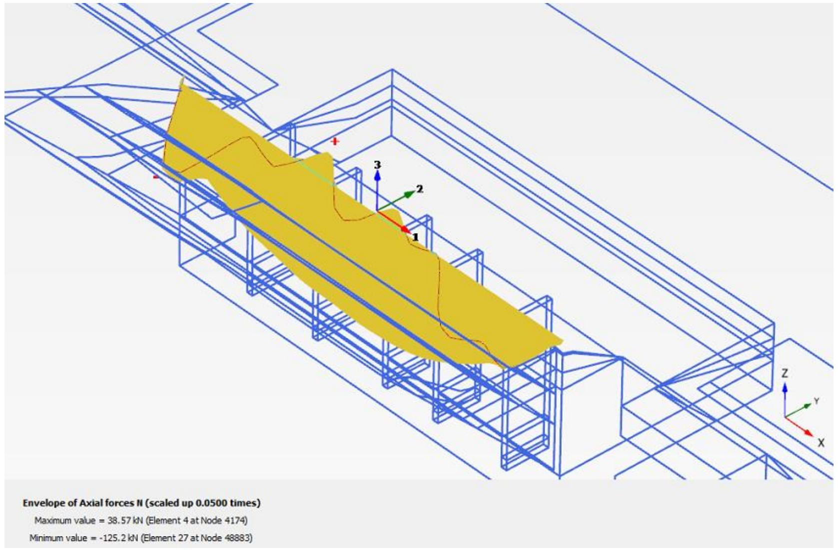
Maximum value = 54.73 kN m (Element 290 at Node 17281)

Minimum value = -239.3 kN m (Element 285 at Node 734)

Forces and Moment – Western CB1

JKGeotechnics

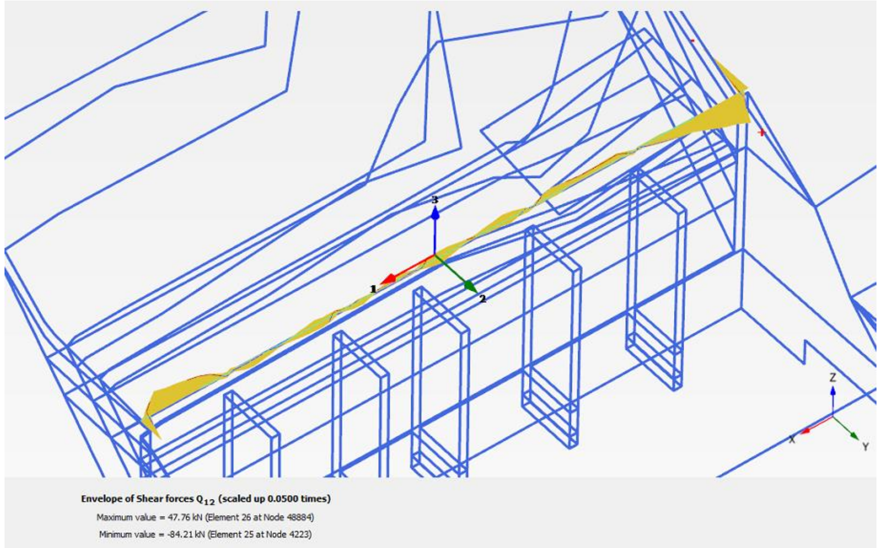
Report No. 33618Y Figure No. 18



Envelope of Axial forces N (scaled up 0.0500 times)

Maximum value = 38.57 kN (Element 4 at Node 4174)

Minimum value = -125.2 kN (Element 27 at Node 48883)



Envelope of Shear forces Q_{12} (scaled up 0.0500 times)

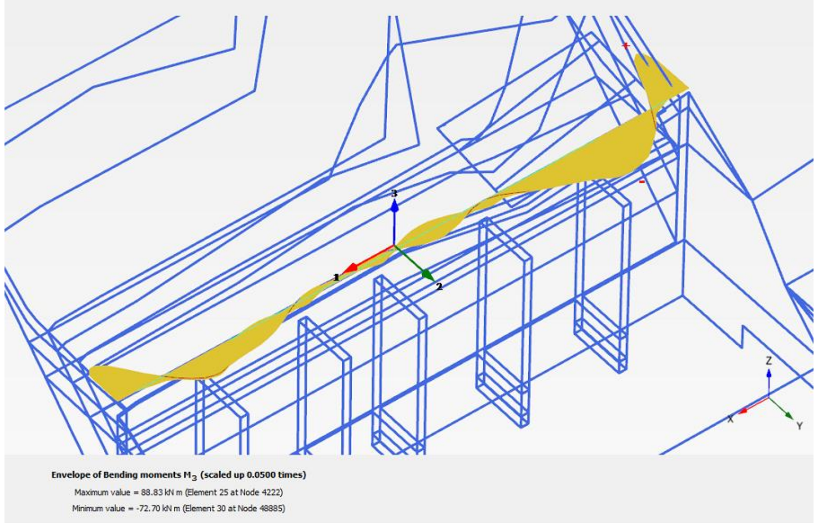
Maximum value = 47.76 kN (Element 26 at Node 48884)

Minimum value = -84.21 kN (Element 25 at Node 4223)

Envelope of Torsion moments M_1 (scaled up 0.200 times)

Maximum value = 16.64 kN m (Element 26 at Node 48884)

Minimum value = -24.10 kN m (Element 29 at Node 48881)



Envelope of Bending moments M_3 (scaled up 0.0500 times)

Maximum value = 88.83 kN m (Element 25 at Node 4222)

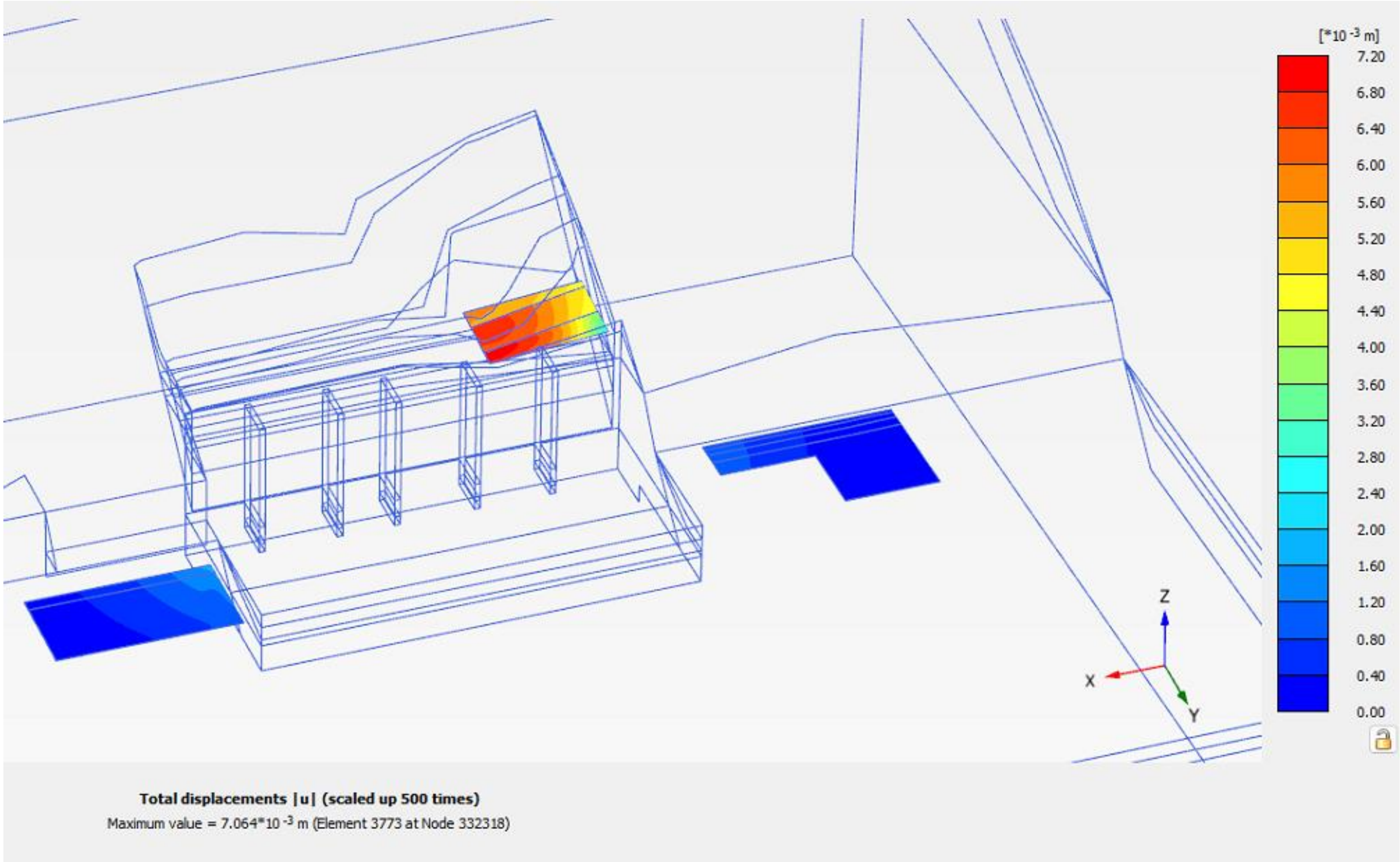
Minimum value = -72.70 kN m (Element 30 at Node 48885)

Forces and Moment – Eastern CB2

JKGeotechnics

Report No. 33618Y

Figure No. 19



Total displacements |u| (scaled up 500 times)

Maximum value = $7.064*10^{-3}$ m (Element 3773 at Node 332318)

Displacements below Neighbouring properties

JKGeotechnics

Report No. 33618Y

Figure No. 20

APPENDIX A1

JKGeotechnics

Date: 22 March 2024
Ref: 33618Ylet5rev

Reform Projects Pty Ltd
15/108 Dunning Avenue
ROSEBERY NSW 2018

Attention: Mr Alex Swiney
Email: aswiney@reformprojects.com.au

SUMMARY OF SECTIONS IN REPORT ADDRESSING THE REQUIREMENTS OF CLAUSE 6.5 OF APPENDIX 5 OF PITTWATER 21 DCP
PROPOSED MIXED USE DEVELOPMENT
1102 BARRENJOEY ROAD, PALM BEACH, NSW

This cover letter details the sections in our report (Ref: 33618Yrptrev4C, dated 22 March 2024) that address each of the requirements listed in Clause 6.5 of Appendix 5 of Pittwater 21 DCP. In addition, in response to Clause 6.5(g), we explicitly detail those parts of the report that should be "referred to by the development consent".

(a) An assessment of the risk posed by all identifiable Geotechnical Hazards that have the potential to either individually or cumulatively affect people or property upon the site or related land to the proposed development in accordance with the guidelines set out in AGS 2007(c) and in particular, in the format as outlined in Figure 1 "Framework for Landslide Risk Management" contained therein. Risk of loss of life should be determined quantitatively. Risk of loss of property can be determined quantitatively or in accordance with the qualitative terminologies and matrices presented in AGS 2007(c).

Refer to Section 5 Risk Assessment and Tables A1 and B1

(b) Plans and sections of the site and related land to a minimum scale of 1:200 from survey and field measurements with contours and spot levels to AHD. Key features are to be identified, including the locations of the proposed development, buildings/structures on both the subject site and adjoining site, storm water drainage, sub-surface drainage, water supply and sewerage pipelines. Where possible, the survey plan should be augmented by geomorphological mapping.

Refer to Figures 3, 4 and 5.

(c) Details of all site inspections and site investigations and any other information used in preparation of the Geotechnical Report. A site inspection is required in all cases. Site investigation may require subsurface investigation; appropriate investigation may involve boreholes and/or test pit excavations or other methods necessary to adequately assess the geotechnical/geological model for the site.

Refer to Section 2 Assessment Methodology.

(d) Photographs and/or drawings of the site and related land adequately illustrating all geotechnical features referred to in the Geotechnical Report, as well as the locations of the proposed development.

Refer to Section 3 Summary of Observations, Section 9.2Response to the Comments Raised by eiaustralia and Figures 2, 3, 4 and 5.

(e) Presentation of a geological model of the site and related land showing the proposed development, including an assessment of sub-surface conditions, taking into account thickness of the topsoil, colluvium and residual soil layers, depth to underlying bedrock, and the location and depth of groundwater. Hydrogeological conditions including seepage inflows and/or dewatering impacts should also be modeled and assessed where applicable. For Coastal bluff areas, the model must also include an assessment of the mechanism of bluff failure and assessment of the potential and scale of bluff failure that may affect the site.

Refer to Section 4 Geology and Subsurface Conditions, Section 6 Groundwater Modelling and Figures 4 and 5.

(f) A conclusion as to whether the site is suitable for the development proposed to be carried out. **This must be in the form of a specific statement that "The site is suitable (or can be made suitable) for the development proposed and that the site and/or the development proposal can achieve the Acceptable Risk Management required by this Policy provided that.".**

Refer to Section 5.2 Risk Analysis and 5.3 Risk Assessment.

(g) Specify all geotechnical conditions to be referred to by the Development Consent. Geotechnical conditions to achieve the management of the Geotechnical Hazard Risk for the subject site throughout the four stages of development management as follows:

Section 7 Comments and Recommendations should be referred to by the Development Consent.

(i) **Geotechnical Conditions to be provided to establish the design parameters** – these conditions are to be provided in the Geotechnical Report -

- Footing levels and supporting rock quality (where applicable)
- Degree of earth and rock cut and fill (where applicable)
- Recommendations for excavation and batters (where applicable)
- Parameters, bearing capacities and recommendations for use in the design of all structural works with geotechnical components including all footings, retaining walls, surface and sub-

surface drainage.

- Recommendations for the selection of building structure systems consistent with the geotechnical risk assessment
- Any other conditions required to ensure the proposal can achieve the "Acceptable Risk Management" level as defined in this Policy.
- Any other condition required to remove geotechnical risks that can reasonably and practically be addressed.

Refer to Section 7.1 Conditions Recommended to Establish the Design Parameters

(ii) Geotechnical Conditions applying to the detailed design to be undertaken for the Construction Certificate – these conditions are to be provided in the Geotechnical Report.

- That any structural design relating to the geotechnical aspects of the proposal is to be checked and certified by a suitably qualified and experienced Structural/Civil Engineer and Geotechnical Engineer/Engineering Geologist as being in accordance with the geotechnical recommendations.
- Any other design, excavation or construction conditions the geotechnical engineer preparing the Geotechnical Report believes are required in the design phase in order to ensure the design will achieve the "Acceptable Risk Management" level as defined in this Policy for potential loss of both property and life.

Refer to Section 7.2 Conditions Recommended to the Design Parameters to be Undertaken for the Construction Certificate

(iii) Geotechnical Conditions applying to the Construction – these conditions are to be provided in the Geotechnical Report:

- Constructed works relating to the geotechnical aspects of the proposal that require the sign off by a suitably qualified and experienced Geotechnical Engineer/Engineering Geologist. The report must highlight and detail the

inspection regime to provide the builder with adequate notification for all necessary inspections.

- Any other design, excavation or construction conditions including works methodology and temporary works that the geotechnical engineer preparing the report believes are required in the construction phase in order to ensure the design will achieve the "Acceptable Risk Management" level as defined in this Policy for the potential loss of both property and life.

Refer to Section 7.3 Conditions Recommended During the Construction Period

(iv) Geotechnical Conditions regarding ongoing management of the site/structure – these conditions are to be provided in the Geotechnical Report.

- Any conditions that may be required for the ongoing mitigation and maintenance of the site and the proposal, from a geotechnical viewpoint. Such conditions to be in the form of a recommendation for inclusion as a covenant (or similar) on the land title to ensure that any owner or future owners are clearly notified of their ongoing responsibility.

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

(v) **Geotechnical Conditions applying to the release of the Occupation/Subdivision Certificate** – these conditions are to be provided in the Geotechnical Report.

- Any conditions that may be required for the Occupation/Subdivision stage, from a geotechnical viewpoint

Refer to last paragraph in Section 7.3 Conditions Recommended During the Construction Period

(h) For bushfire prone lands, as designated in the Pittwater LGA Bushfire Prone Land Map, the Geotechnical Report is to assess the potential geotechnical impacts of any Asset Protection Zones required and mitigate landslide risk due to Bushfire management.

NA

(i) For coastal bluff areas designated on Pittwater's Coastal Risk Planning Map, a coastal engineer's report on the impact of coastal processes on the site and the coastal forces prevailing on the bluff must be incorporated into the geotechnical assessment as an appendix and the Coastal Engineer's assessment must be addressed through the Geotechnical Report and structural specification.

NA

(j) A statement with supporting information to the effect that every reasonable and practical step available has been identified to remove any foreseeable geotechnical risk from the site over and above attainment of the "Acceptable Risk Management" criterion.

Refer to Section 5.3 Risk Assessment, Section 7 Comments and Recommendations and Appendix D.

(k) A copy of Forms 1 and 1(a) bearing the original signature of the Geotechnical Engineer and/or Engineering Geologist as defined by this Policy, who has either prepared or technically verified the Geotechnical Report. Where a Coastal Engineer has been involved as required by this Policy, separate Forms 1 and 1(a) must be submitted by that Engineer.

Refer to provided Forms 1 and 1(a) which are attached to the front of the report.

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

Woodie Theunissen
Principal | Geotechnical Engineer

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for________________ Reform Projects Pty Ltd_____________________________
Name of Applicant
Address of site __1102 Barrenjoey Road, Palm Beach______________________________

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

I, ________Woodie Theunissen__________ on behalf of JK Geotechnics Pty Ltd
(Insert Name) (Trading or Company Name)

on this the ____22 March 2024____ certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million.
we/I have:

Please mark appropriate box

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical Assessment

Report Date: 22 March 2024 Report Ref No: 33618Yrptrev4C
:
Author: Woodie Theunissen

Author's Company/Organisation: JK Geotechnics Pty Ltd

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

Architectural drawings prepared by Innovate Architects (refer to drawings cover page, Job Number: 2926, dated 22 March 2024)

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

Signature …… …………………………………..

Name ……Woodie Theunissen…………………………………………..

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

Membership No. ……889807……………………………………

Company: JK Geotechnics Pty Ltd.

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

Development Application for________________Reform Projects Pty Ltd____________________
Name of Applicant
Address of site __1102 Barrenjoey Road, Palm Beach______________________________

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
Report Date: 22 March 2024 Report Ref No: 33618Yrptrev4C
Author: Woodie Theunissen
Author's Company/Organisation: JK Geotechnics Pty Ltd

Please mark appropriate box

☒ Comprehensive site mapping conducted ____3 November 2020_______
(date)

☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:250 (as appropriate)

☒ Subsurface investigation required
- ☐ No Justification …
- ☒ Yes Date conducted ………4 November 2020………………

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

☒ Geotechnical hazards identified
- ☒ Above the site
- ☒ On the site
- ☒ Below the site
- ☒ Beside the site

☒ Geotechnical hazards described and reported

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

☒ Risk calculation

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

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

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

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

☒ Design Life Adopted:
- ☒ 100 years
- ☐ Other …………………………………………….
specify

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

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

☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature … ……………………………………….
Name …Woodie Theunissen
Chartered Professional Status……CPEng………………………
Membership No. ……889807…………………………..
Company JK Geotechnics Pty Ltd.

REPORT TO

REFORM PROJECTS PTY LTD

ON

UPDATED SEEPAGE ANALYSIS AND GEOTECHNICAL ASSESSMENT (In Accordance with Pittwater Council Risk Management Policy)

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Date: 22 March 2024

Ref: 33618YJrptrev4C

JKGeotechnics
www.jkgeotechnics.com.au

T: +61 2 9888 5000
JK Geotechnics Pty Ltd
ABN 17 003 550 801

Report prepared by:

Woodie Theunissen
Principal Associate | Geotechnical Engineer

Report reviewed by:

Paul Stubbs
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33618YJrpt	Original	30 November 2020
33618YJrptrev1	Revision 1	25 May 2022
33618YJrptrev2	Revision 2	15 August 2022
33618YJrptrev3	Revision 3	16 September 2022
33618YJrptrev4	Revision 4	29 June 2023
33618YJrptrev4A	Revision 4A	11 January 2024
33618YJrptrev4B	Revision 4B	24 January 2024
33618YJrptrev4C	Revision 4C	22 March 2024

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

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:

a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

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Table of Contents

ATTACHMENTS

1 INTRODUCTION

This updated report presents the results of our seepage analysis and geotechnical assessment of the site at 1102-1106 Barrenjoey Road, Palm Beach, NSW. The location of the site is shown in Figure 1. This updated report has been prepared to:

- Reflect the updated architectural drawings referenced below,
- Report the additional investigation completed below the boulders present towards the middle of the southern site boundary
- Address the comments outlined in the correspondence prepared by Crozier Geotechnical Consultants (CGC) (email dated 13 February 2023) and eiaustralia (Ref: P21153.01_Draft, dated 16 January 2023,) and (Ref: P21153.02 REV1, dated 13 February 2023) and
- Address Point 4 of the Notice of Determination, the proposed development is inconsistent with the provisions of Clause 7.7 Geotechnical Hazards of the Pittwater Environmental Plan 2014.

We understand from the provided architectural drawings prepared by Innovate Architects (refer to drawings cover page, Job No: 2926 dated 22 March 2024), that the proposed development will comprise the following:

- Demolition of the existing buildings and structures on site, and,
- Construction of a three-storey building over one basement garage level.

The new building and garage will be cut into the existing hillside. Over the front or western side of the site the basement will have a finished floor level of RL-0.65m, or an assumed bulk excavation level of RL-1m, that will extend to the eastern, southern and western site boundaries and to within about 0.5m of the northern site boundary. At the eastern edge the excavation steps up to a proposed terrace with a finished floor level of RL6.55m. Excavation will require cuts to a maximum depth of about 12.5m. While maximum excavation depths are anticipated to be in the order of about 12.5m, excavation to these depths is over a relatively minor portion of the eastern portion of the site with the majority of the excavation occurring over the western portion of the site and limited to a depth of about 4m. The proposed excavation will remove the existing retaining structures and along the southern site boundary the sandstone boulders that are present will be trimmed back to the extent that they protrude into the building footprint.

The footprint of the proposed development is indicated on Figures 2 and 3.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 5 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 and a letter (ref: 33618Ylet5Rev3, dated 22 March 2024) that identifies the various sections of the report that have been prepared to address Clause 6.5 of Appendix 5 of Pittwater 21 DCP.

JK Environments (JKE) completed Acid Sulphate Soils testing in conjunction with the geotechnical investigation. The results of the testing have been separately reported by JKE (Ref: E33618Blet, dated 1 December 2020).

2 ASSESSMENT METHODOLOGY

The assessment methodology was completed in a staged manner and comprised the following:

- A walkover survey of the site by our Associate, Mr Jarett Mones, for the purposes of completing the stability assessment. This was completed on 3 November 2020.
- Investigation of the subsurface conditions across site which was completed over two days of investigation on 11 and 12 November 2023.
- Investigation of the materials present below the upper boulder present midway along the southern boundary.

2.1 Walkover Survey

The stability assessment was carried out by our Associate, Mr Jarett Mones and is based on 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 3 presents a plan showing notable geotechnical site features and geotechnical hazards. Figure 3 is based on the survey plan prepared by CMS Surveyors Pty Ltd (CMS, Ref: Drawing No. 19783detail, Issue 1, date of survey as 3 November 2020). We also received an older survey plan by Adam Clerke Surveyors Pty Ltd (Ref: 20688L, dated 26 April 2012), which we used to review former levels at the site. Additional features on Figure 3 have been measured by hand held clinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 6 defines the geotechnical mapping symbols adopted and used in Figure 3. Figures 4 and 5 presentw typical cross-sections (Sections A-A' and B-B', respectively) through the site based on the survey data augmented by our mapping observations. These sections include potential landslide hazards.

2.2 Subsurface Investigation

<u>JK Geotechnics Geotechnical Investigation</u>

Prior to the commencement of the fieldwork, we carried out the following:

- Review of recommendation requirements by Reform Projects and Van der Meer Consulting (Ref: emails dated 20 and 26 October, respectively);
- Review of the previous geotechnical report by Witt Consulting Pty Ltd (Witt, Ref: WittC-TMattox-R-A, dated November 2019) and Supplementary Geotechnical Investigation report by D.F Dickson & Associates Pty Limited (D.F Dickson, Ref: 28207-G6, dated 20 July 2012 and reviewed February 2013);
- Liaison with Reform Projects to agree on borehole locations;
- Meeting with JKE to discuss sampling requirements for the Acid Sulphate Soils testing;
- Completion of a Dial Before You Dig buried services search and an on-site services scan using electromagnetic induction measures completed by a buried services subcontractor; and,
- Site walkover by our Principal Associate, Mr Woodie Theunissen, to review access for the investigation locations.

Our geotechnical investigation and infiltration testing were carried out over two (2) days on 11 and 12 November 2020 and comprised the following:

- Completion of two (2) boreholes, BH101 and BH102. These boreholes were drilled using our JK205 rig in the accessible area (existing driveway) at the southern portion of the site and were initially advanced using spiral auger techniques with an attached twin pronged Tungsten Carbide (‘TC’) drill bit to depths of 3.42m (BH101) and 0.46m (BH102). These boreholes were then extended to final depths of 9.40m (BH101) and 8.98m (BH102) using rock coring techniques and a NMLC triple tube core barrel.
- Installation of monitoring wells in each of the boreholes upon completion.
- Following the above, we carried out two infiltration (‘pump out’) tests in each of the wells by pumping out the groundwater and measuring the groundwater recharge rate as groundwater flowed back into the wells. The results of each of the pump out tests are presented as Pump-Out Recovery Testing vs Time graphs and are attached as Figures 7 and 8. Following the completion of pump out testing, data loggers were left installed within the wells to allow longer term monitoring of groundwater levels.

Groundwater monitoring was carried out over about a two-week period between 5 and 18 November 2020. In addition to the above, groundwater observations were also made during, on completion and for a short period following augering and coring. A summary of measured groundwater levels is provided below in Section 3.2. Groundwater level measurements recorded on site during drilling are shown on the borehole logs. The data logger plots showing Groundwater Level and Daily Rainfall vs Time are attached as Figures 11 and 12. We note that water is introduced into the boreholes during core drilling and therefore the water levels after coring are likely to be artificially higher than actual levels. Consequently, the longer term groundwater levels measured between the 5 and 18 November 2020 and presented in Figures 11 and 12 provide a more reliable indication of ‘true’ groundwater levels.

The apparent compaction of the fill and relative density of the subsurface soils encountered in BH101 was assessed from the results of Standard Penetration Test (SPTs) ‘N’ values.

Where the bedrock was drilled using spiral auger drilling techniques the strength of the bedrock was assessed from observation of the TC bit drilling resistance, tactile examination of recovered rock chips/cuttings and correlation with the results of subsequent laboratory moisture content testing. Strengths assessed in this manner are approximate only and variations of one strength order should not be unexpected. The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with Point Load Strength Index ($I_{S\,(50)}$) testing. The results of the Point Load Strength Index tests are presented in the attached JK Geotechnics Table B and on the cored borehole logs. The Unconfined Compressive Strength's (UCS's) were estimated from the Point Load Strength Index test results and are also summarised in Table B. Photographs of the recovered core are presented to the rear of this report with the borehole logs.

The borehole location plan is included as Figure 2. Due to access constraints, boreholes were limited to areas outside the building and other structures. The locations were set out by taped measurements from apparent surface features, as shown on the survey plan prepared by Adam Clerke Surveyors Pty Ltd and referenced above. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the CMS survey drawing and are, therefore, only approximate. The datum for the levels is Australian Height Datum (AHD), as noted on the survey drawing.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd (Envirolab), both NATA accredited laboratories, for moisture content and pH, sulphate content, chloride content and resistivity testing. These results are presented in the attached STS Table A and Envirolab Certificate of Analysis No. 255075.

The investigation was carried out in the full-time presence of our Geotechnical Engineer, Mr Ben Sheppard, who set out the borehole locations, nominated the sampling (including those for the JKE Acid Sulphate Soils assessment) and testing and prepared logs of the strata encountered. The borehole logs (which include groundwater measurements), are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques adopted and their limitations, and define the logging terms and symbols used.

In Sections 3 and 4 we have provided a discussion of some of the findings from the previous investigations/assessments by Witt and D.F. Dickson. Below is a summary of the scope of works from the previous subsurface investigations/assessments carried out by those consultants:

Witt Geotechnical Investigation

The Witt geotechnical investigation comprised three augered boreholes (BH1 to BH3). BH3 was carried out to a depth of 4m, while the depth of the other boreholes is unclear as it was not recorded on the BH1 borehole log and the final page of the BH2 borehole log was not provided. However, the cross-sectional sketch annotates the end of BH1 and BH2 at RL10m depth. Wells were installed in each of the boreholes. We measured the 'groundwater' level in the well installed at BH2, whilst on site. This well indicated that the groundwater level was up to 0.6m above ground level, suggesting artesian pressures. These artesian pressures may be a result of the well intersecting confined defects within the rock mass that are charged with water that is under pressure, thus resulting in a water level that is above ground level. We could not remove the gatic cover at the BH3 location and we did not find the well at the BH1 location. The Witt borehole logs have been included as Appendix C.

No testing of the soils was carried out during the Witt investigation.

D.F. Dickson

The D.F. Dickson report comments that they had completed a series of inspections and investigations in 2002, 2004, 2009, 2010, 2011 and 2012. The report discusses previous works carried out on site and provides a slope stability assessment. The report does not include borehole logs or other investigation/inspection details but in the methods of investigation section it states that the following had been carried out:

> 'Large scale boreholes, 0.6m diameter to a depth of 8m at the retaining wall location at the rear of the property. This includes collection and examination of rock cuttings from the bored piers and inspection of drainage and ground stabilisation installation, underpinning and saw cutting of sandstone boulders.'

Crozier Geotechnical Consultants (CGC)

CGC have completed an investigation of Lot 3, 1110 Barrenjoey Road, Palm Beach which is located behind the majority of the site. Their investigation reported that the property upslope from the site is underlain by a relatively thin topsoil/colluvium layer that in turn overlies a residual sandy clay. Below this sandy clay, sandstone bedrock of variable quality was encountered. The bedrock typically increased in quality with depth, although there were still zones of poorer quality material present within this better quality bedrock. Although it is a little unclear in some of the boreholes whether they have encountered sandstone boulders or sandstone bedrock, it appears that even with the most adverse interpretation of the borehole logs that sandstone bedrock is present at depths ranging from 2.15m to 4.25m.

2.3 Investigation of Materials Present Below Upper Boulder on Southern Site Boundary

The investigation was completed on 31 January 2023 and comprised the drilling of eight horizontal core holes through the shotcrete and mesh facing. All holes were initially drilled using a 0.1m diameter diatube. Following the penetration of the shotcrete and mesh panel, a hand auger was then used to horizontally penetrate the soils present behind the panel. Where bedrock was encountered and could not be penetrated with the hand auger, a screw driver was used to obtain a sample of the bedrock.

Test locations were positioned on three section lines and were progressively drilled from below the underside of the upper sandstone boulder (B1) on the southern boundary. Where bedrock was not encountered immediately below B1, another hole was drilled at a slightly lower level. This process was repeated until bedrock was exposed. Due to the elevated nature of B1, scaffold was used to drill the upper holes. Figure 3 shows the position of the section lines on which the test locations were positioned while Figure 5 presents sections showing the soil and bedrock profile present behind the shotcrete and mesh panel. The table presented below in Section 3 of this report, details the materials encountered at each of the test locations.

The investigation was completed in the full time presence of our Principal Associate, Mr Woodie Theunissen. Where clayey soils were present behind the shotcrete and mesh panel, hand penetrometer tests were completed to assess the strength of the soils. For more information on the nature of the materials present and their strength, reference should be made to the table below and Figure 5.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figure 1 (Site Location Plan) and Figure 3 (Plan of Notable Geotechnical Features and Geotechnical Hazards).

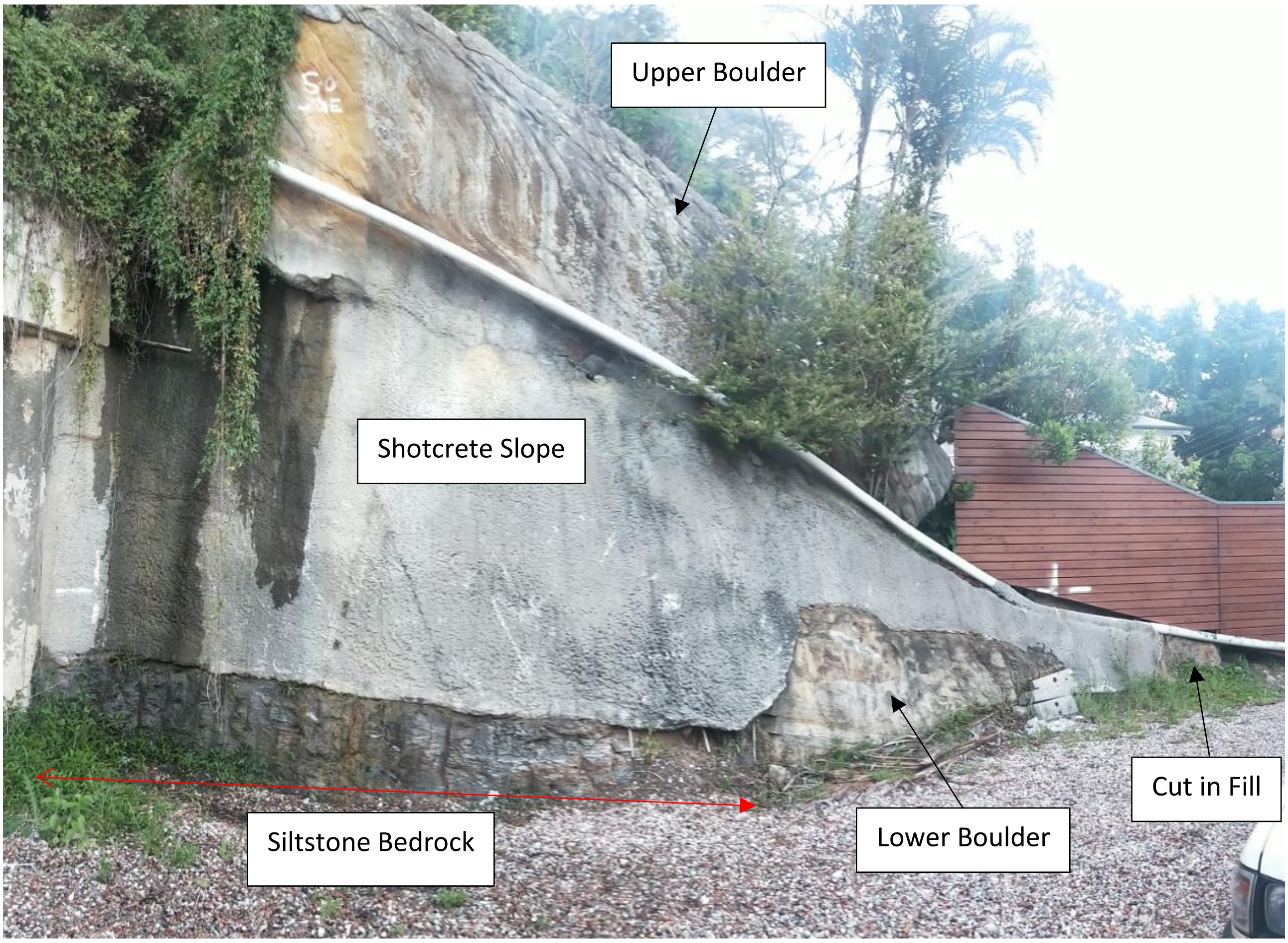
The site is located east of Pittwater, below and along the base of a north-south trending ridgeline. Original site slopes have previously been modified with the western portion of the site cut into the hillside to a soldier pile wall that is set-in about 6.5m from the eastern boundary. The site slopes up steeply to the eastern site boundary from the back of the piled retaining wall at about 45° and is covered with thick grass. Based on the D.F. Dickson report the slope has been reinforced by geogrids and there is a ‘drainage system piped to the street’.

To the west of the soldier pile wall the site has been cut, is relatively level and has ground levels that vary between about RL2.5m and RL3m. This portion of the site is currently unoccupied. The solider pile wall comprised 0.6m diameter piles spaced at about 2.5m, with mesh and shotcrete infill panels between the piles to about 0.8m to 0.9m above the base of the excavation. Mesh and shotcrete has not been placed here and the materials exposed were assessed to comprise moderately weathered siltstone of low to medium strength. Water seepage was observed in this face, both over the siltstone cut face and through the shotcrete panels. Some cracks varying from ‘hairline’ to 2mm in width were observed in the shotcrete panels but overall, the soldier pile wall appeared to be in good condition. At the top of the piles was a concrete section that was about 1.1m high and appears to be the capping beam. Above the capping beam was a mesh fence about 1m high. The architectural section drawing by Mills Architecture & Interiors (Ref: Drawing No. DA.01) details the base of the soldier piles at RL-0.25m. We have extracted a photograph from the Witt geotechnical investigation report, as shown below in Plate 1, of the soldier pile wall prior to being shotcreted. It appears the portion below the capping beam comprises weathered rock.

Plate 1: Photograph of Soldier Pile Wall Prior to being Shotcreted (Figure 3 from Witt Report)

The site description that follows should be read in conjunction with Plate 2, which includes a photograph of the southern boundary area. Extending into the site near the southern boundary is a large sandstone boulder. This boulder is located in the upper portion of the profile and where excavation has extended below it the cut face has been protected with shotcrete. The boulder is approximately 2m to 3m high and extends for a length of about 7.5m. The shotcreted cut has a maximum height of approximately 3.5m and extends for a length of about 10.5m. The boulder overhangs the eastern end of the cut by about 1m to 1.5m over a length of 1.5m. Water seepage was observed below the boulder along the eastern 2m length of the shotcrete facing. A second sandstone boulder is exposed below the western end of the upper boulder. This boulder extends for a length of about 2.5m over the lower 0.7m to 1.3m of the cut. Between the eastern soldier pile wall and this lower boulder, siltstone bedrock was exposed below the shotcrete for a length of about 6m to 7m over the lower 0.8m of the cut. The siltstone bedrock was assessed to be moderately weathered and of low to medium strength. A 0.8m high vertical cut in fill was observed over a length 1m at the westernmost end of the southern cut.

Plate 2: Photograph 1 of Large Boulders and Shotcreted Cut Face At/Near Southern Boundary (Facing South)



The site is bounded by residential development to the south (1100 Barrenjoey Road). It appears that the upper boulder visible in the southern cut face extends an additional 2.5m to the south and overlies another large boulder that extended further to the south. This can be seen below in Plate 3. While this lower boulder appears to be one boulder, investigation by Davies Geotechnical Consulting Engineers has established that this lower boulder is actually two discreet boulders that are positioned adjacent to one another with the southern (lower) boulder located entirely within 1100 Barrenjoey Road. Beyond these boulders is a two-storey timber clad house. To the east of the site is a flat grass area (driveway) and then a single-storey shed. To the north is a one and two storey rendered building ('Barrenjoey House', which is heritage listed) while Barrenjoey Road bounds the site to the west. All adjoining structures appeared in good condition when viewed from the site.

Plate 3: Photograph 2 of Large Boulders and Shotcreted Cut Face At/Near Southern Boundary (Facing East)

4 GEOLOGY AND SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by the Newport Formation, 'Narrabeen Group', which comprises interbedded laminite, shale and quartz to lithic quartz sandstone.

Since the eastern portion of the site has previously been cut, the investigations disclosed subsurface conditions generally comprising shallow weathered bedrock over the eastern portion of the site transitioning to fill overlying sands and then weathered bedrock over the western portion of the site. The weathered bedrock comprised interbedded sandstone, siltstone and laminite.

Some of the more pertinent details of the strata encountered are described below. For further details of the conditions encountered at each borehole location, reference should be made to the attached borehole logs.

Information from the subsurface investigation by Witt, which included 3 boreholes, BH1 to BH3, has been included in our summary below.

Pavements

Prior to the clearing of the site there were pavers around the buildings. In BH1, which was carried out at the northern portion of the site, the pavers were about 0.05m thick.

Fill

From the JKG and Witt investigations, fill was encountered to depths ranging from 0.05m to 1.6m. The upper 50mm of fill in the driveway was gravel. At BH101, underlying the gravel discussed above, the fill comprised silty sand and clayey sand with brick and concrete fragments and was assessed to be well compacted. At BH1, underlying the pavers, the fill comprised gravelly sand, silty sand and sandy clay with sandstone fragments.

Marine Soils

Underlying the fill, the marine soils at BH101 comprised silty sand that was assessed to be of very loose relative density and extended to a depth of 3.0m. Traces of clay and ironstone and quartz gravel were noted within the silty sand. The Witt investigation encountered sand of an assessed loose relative density that extended to depths between 2.6m and 2.7m. In BH1, a silty clay was encountered below the sand and extended from a depth of 2.6m to 2.8m and was assessed to be of firm to stiff strength.

We assessed the natural sands and clays in the Witt investigation to be marine soil, but the clay may be residual soil. As testing of the soils was completed as part of our investigation, we have more confidence in our assessment of relative density/strength for the underlying soils than we do for the Witt boreholes, where there is no record that of the subsurface testing that has been completed.

Bedrock

Bedrock was encountered across the western level portion of the site at depths ranging from 0.05m (RL2.85m) to 3.0m (RL-0.3m). At BH101 and BH102, the bedrock comprised interbedded siltstone and sandstone, laminite and siltstone and was typically of very low strength, although some harder bands were present. At BH101, the bedrock included thick zones (up to about 0.9m thick) of extremely weathered material with properties of a hard clay and a 0.35m thick ‘No Core’ zone at a depth of 7.78m. The ‘No Core’ zone likely represents an extremely weathered band that has washed away during coring.

Based on our observation of the bedrock exposed in the cut faces, the supplied photos and the results of the CGC report, it appears that bedrock over the eastern portion of the site is present at relatively shallow depth and, where exposed appears to be of at least low to medium strength. This apparent poorer quality bed of rock exposed over the eastern portion of the site below about RL2.85m is not unusual for rock within the 'Narrabeen Group', which can be of quite variable quality and does, in places, become poorer in quality with depth.

Defects within the rock mass generally comprised numerous thin extremely weathered seams (typically 5mm to 30mm thickness but some up to 70mm to 140mm thickness) and some sub-horizontal bedding partings. Some steeply inclined joints (generally 30° to 90°) were also observed.

Based on the photograph provided in the Witt report included in Section 3, it appears bedrock is exposed below the capping beam along the southern portion of the existing soldier pile wall, so the top of bedrock is estimated to be at RL6.4m or higher at this location.

Materials Present Below Upper Boulder (B1) on Southern Boundary

The table below presents the results of the investigation. This table details the location of the test holes below the underside of the upper boulder, the thickness of the shotcrete, whether drainage was present behind the panel and a description of the soil and bedrock present. The drainage was in the form of prefabricated strip drains.

Thickness of Shotcrete and Mesh Panel and Nature of Materials Present Behind the Panel				
Test	Height Below Underside of B1 (mm)	Thickness of Shotcrete and Mesh Panel (mm)	Is Drainage Present Behind the Shotcrete and Mesh Panel	Description of Materials Present Behind Shotcrete and Mesh Panel
1 Section C-C	0.25	90	Yes	Silty Clay: Medium to high plasticity, grey and orange brown (MC>PL) (VSt) (HP=240kPa)
2 Section C-C	0.5	120	Yes	Siltstone: Grey and Orange Brown (DW) (VL-L)
3 Section B-B	0.5	110	No	Silty Clay: Medium to high plasticity, grey and orange brown (MC>PL) (St) (HP=120kPa)
4 Section B-B	1.0	140	No	Silty Clay: Medium to high plasticity, yellow brown (MC>PL) (VSt) (HP=160kPa)
5 Section B-B	1.4	140	Yes	Sandy Clay: Medium plasticity, grey and yellow brown (MC>PL) (VSt) (HP=180kPa)
6 Section B-B	1.8	160	No	Sandstone: fine grained, yellow brown (H) (EL)
7 Section A-A	1.05	150	No	Sandstone: Fine to medium grained, yellow and orange brown (DW) (M)
8 Section A-A	0.75	130	No	Sandstone: Fine to medium grained, yellow and orange brown (DW) (M)

Groundwater

In BH101 and BH102, groundwater observations were made on completion or a short period after augering/coring. Subsequent groundwater readings in the wells installed in BH101 and BH102 were carried out the day following the well installation, with continuous groundwater monitoring carried out between 5 and 18 November 2020. Below is a summary of the depths and RLs for these measurements and the groundwater levels recorded by Witt.

Test Location (Surface ~RLm)	Date of Measurement	Note on measurement	Groundwater Depth and Level (AHD)
BH101 (2.7)	4 November 2020	Seepage during augering	2.3m (RL0.4m)
	4 November 2020	On completion of augering	2.4m (RL0.3m)
	4 November 2020	On completion of coring	2.6m (RL0.1m)
	5 November 2020	Following day prior to 'pumping out'	1.96m (RL0.74m)
	5 to 18 November 2020	Continuous readings with loggers following 'pumping out'	2.02m to 2.18m (RL0.68m to RL0.52m)
	18 November 2020	Following groundwater monitoring	2.16m (RL0.54m)
BH102 (2.7)	4 November 2020	After 3hrs of coring	0.82m (RL1.88m)
	5 November 2020	Following day prior to 'pumping out'	0.84m (RL1.86m)
	5 to 18 November 2020	Continuous readings with loggers following 'pumping out'	0.85m to 1.13m (RL1.85m to RL1.57m)
	18 November 2020	Following groundwater monitoring	1.16m (RL1.54m)
BH1 (2.5)	23 October 2019	Understood to be during drilling	2m (RL0.5m)
BH2(2.9)	23 October 2019	Understood to be during drilling	2m (RL0.9m)
BH3 (2.9)	23 October 2019	Understood to be following drilling	0.6m above ground (RL3.5m)
	3 November 2020	Approximately 1yr following drilling	0.2m above ground (RL3.1m)

Near the southern boundary, water seepage was observed below the boulder. Along the soldier pile wall, water seepage was observed both through the shotcrete and mesh panels and over the lower siltstone bedrock exposed in the cut face below the shotcrete and mesh.

4.4 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores from each borehole correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s\ (50)}$) tests ranged from 0.01MPa to 0.6MPa. These are also plotted on the attached borehole logs. Estimated unconfined compressive strength (UCS), which are based on the relationship of UCS = 20 x $I_{s(50)}$, ranged from <1MPa to 12MPa.

The moisture content test results on samples of the weathered rock showed reasonably good correlation with our field assessment of rock strength in the augered portion of the boreholes.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below and results are attached in the Envirolab Certificate of Analysis No. 255075. Refer to Section 7 for an interpretation of the results.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH101	1.0-1.2	Fill: Clayey Sand	7.2	33	20	15,000
BH101	2.6-2.8	Silty Sand	7.4	29	32	20,000
BH102	3.0-3.15	Laminite (bedrock)	7.0	<10	10	31,000

4.5 Infiltration Testing and Calculated Permeabilities

To assess realistic coefficients of permeability for the soils and the weathered bedrock, two pump-out tests were completed in both BH101 and BH102. Data loggers were installed in both boreholes during testing to allow accurate measurement of groundwater recharge. The results of these tests are presented in Figures 7 and 8. At the time of testing, the infiltration zone encompassed the soils (generally silty sand and clayey sand including fill) and extremely to moderately weathered bedrock in BH101 and generally highly weathered to moderately weathered bedrock in BH102.

Based on the testing completed, the coefficient of permeability for the strata was calculated. Since the permeability of the sands is significantly greater than that of rock, the testing in BH101 assumed that the seepage predominantly occurred through the sands. Three alternative methods of analysis were used. These included the Basic Time Lag method, a method by WJ Neely (Ground Engineering, Vol 7, 1974) and the method outlined in the Naval Facilities Engineering Systems Command (NAVFAC) DM7 Design Manual. We note that the Neely method includes studies from larger sized test holes and some interpretation was made beyond the limits of chart the method provides. The table below sets out the calculated values. A range of values was provided as two pump out tests were completed at each location and the results varied slightly.

Calculated Insitu Permeability's (k) (m/s)		
	Borehole BH101	Borehole BH102
Method 1 – Basic Time Lag	8.7 x 10^{-7} to 9.1 x 10^{-7}	2.4 x 10^{-7} to 2.5 x 10^{-7}
Method 2 - Neely	1.5 x 10^{-6} to 1.6 x 10^{-6}	3.2 x 10^{-7} to 3.8 x 10^{-7}
Method 3 – NAVFAC DM7	1.4 x 10^{-6} to 1.5 x 10^{-6}	3.3x 10^{-7} to 3.6 x 10^{-7}

The values were fairly consistent between the three calculated methods. However, it is possible that the calculated values above are unrealistically high, particularly in the bedrock in BH102, because testing was completed over a relatively short period of time and consisted of the removal of relatively small amounts of groundwater between subsequent tests. As has been observed in basement excavations completed both in the vicinity of the site and throughout the Sydney basin more generally, groundwater inflows are higher initially before reducing to a steady state or ceasing all together as drawdown occurs. The limited time over which the pump-out tests were completed is likely to represent this initially higher inflow rate rather than the steady long-term seepage rate.

5 GEOTECHNICAL ASSESSMENT

5.1 Potential Landslide Hazards

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

A Stability of existing large boulder near/at the southern boundary (requires shotcrete slope below to fail, assumed as engineered);
B Stability of the soldier pile wall (assumed as engineered);
C Stability of the reinforced hillside upslope (east) of the soldier pile wall (assumed as engineered);
D Stability of the localised 0.8m high vertical cut in fill along a 1m length at the southern boundary; and,
E Stability of new engineered retaining walls

These potential hazards are indicated in schematic form on the attached Figure 3.

5.2 Risk Analysis

The attached Table A1 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 A1 indicates that the assessed risk to property is Very Low, which would be considered 'acceptable' in accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B1 together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 3×10^{-8}. This would be considered to be 'acceptable' in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

5.3 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, its design and the 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, reinforced earth embankment etc. (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.

Our risk assessment has considered Hazards A, B and C to have been previously engineered and certified during construction. While we observed that some remedial works have been carried out from our site inspection, the design and as-built records were not available to confirm the design and construction details. According to the D.F. Dickson report, they were involved during the construction period and have provided certification for these elements. In this regard we recommend that the D.F. Dickson reports, design drawings and as-built records are obtained so that our assessment of the likelihood of instability of these Hazards can be confirmed. While most of Hazard C will be removed, if these records cannot be obtained we recommend that further investigation of Hazard C be carried out as discussed further in Section 7 during construction. We understand that Hazard B will be demolished during construction and that Hazards A and D will be provided with long term support by the construction of a retaining wall.

Where boulders are present upslope in 1110 Barrenjoey Road, the responsibility of managing this risk such that it poses an acceptable risk to 1102-1106 Barrenjoey Road is the responsibility of these upslope owners.

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 site and existing and proposed development 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.

6 GROUNDWATER MODELLING

6.1 Analysis Methodology

Steady state seepage analysis has been carried out on one section, Section A-A', which is shown on the attached Figure 3. The purpose of the modelling was to provide an estimate of extraction rates required during construction. The section is based on the survey plan by CMS and was chosen as it represented a 'typical' section through the site in terms of the proposed boundary set-backs, depth of proposed excavations, gradient of the existing ground and groundwater and subsurface conditions.

The seepage analysis was carried out using a 2D finite element computer program, SEEP/W (from GEO-SLOPE). The methodology used for the seepage analysis was:

- Develop a subsurface model for the site using the information obtained from our investigations, supplied architectural and survey drawings, results from the groundwater monitoring, pump-out testing and from the review of published information on permeability rates for sands, siltstone and sandstone, and,
- Carry out a steady state seepage analysis of the proposed basement excavation, as shown in Section A-A'.

6.2 Review of Published Information and Discussion on Permeability Rates

In a paper by Paul Hewitt, *Groundwater Control for Sydney Rock Tunnels*[2], the range of typical 'bulk permeability' for the weathered Ashfield Shale was given as 10^{-6} to 10^{-9} m/s while for the Hawkesbury Sandstone ranged from 10^{-6} m/s near the surface to about 2 x 10^{-8} m/s at 50m depth. While the 'Narrabeen Group' bedrock is generally more weathered and weaker than the above formations, these permeabilities provide a guideline for comparison purposes.

The permeability of the underlying bedrock will be governed by fracturing within the bedrock itself, in particular the continuity, tightness and interconnectedness of this fracturing. As a result, the groundwater ingress experienced by the excavation will depend on the frequency (spacing), continuity and openness of the joints intersected.

Table 8.1 in AS1726 (1975) provides permeability values for different types of soils. The permeability of 'clean sands, clean sand and gravel mixtures' ranges between 10^{-2} m/s and 10^{-5}m/s and for 'very fine sands, organic and inorganic silts and mixtures of sand' ranges between 10^{-5} m/s and 10^{-9}m/s. The soils on site contained fines and are expected to have permeabilities that fall within the latter permeability ranges.

6.3 Subsurface Conditions and Groundwater

Generally, the excavation will be predominately through interbedded siltstone, sandstone and laminite bedrock over the eastern portion of the site and sands over the western portion of the site. East of the existing soldier pile wall clayey soils are anticipated to be encountered overlying the bedrock. In the western portion of the site, a 'cut-off' wall will be required to prevent soil and water flowing into the excavation both during construction and in the long term and provide both temporary and permanent support. In this regard the cut-off wall has been assumed to be impermeable. As a consequence, seepage inflows must flow around the toe of the cut-off wall and come up into the excavation through the floor of the basement excavation. Over the eastern portion of the site where bedrock is at shallow level, groundwater inflow will be through the rock mass exposed in both the cut faces and base of the excavation. The geotechnical model is shown in Figure 9.

Groundwater was encountered in each of the boreholes drilled by JKG and Witt. Groundwater measurements were presented in Section 3.2. A groundwater level of RL0.7m was adopted from BH101 to the west of the site and a groundwater level of RL0.7m to RL2.2m was adopted from BH102 to the solider pile wall. Beyond the soldier pile wall a groundwater gradient was estimated to slope upwards as shown in Figure 9. The groundwater levels adopted were based on the highest groundwater levels recorded in BH101 and BH102 over the monitoring period. We recommend continued periodic groundwater monitoring in the wells be carried out prior to construction to review that the above groundwater levels are appropriate. Further modelling may be required to assess extraction volumes due to groundwater level rises.

The table below sets out the coefficients of permeability adopted for the analysis. These values are based both on an assessment of the published literature, as presented above, experience on similar projects and the results of the pump-out tests. The permeability of the sands was considered to be towards the lower bound of the expected range (i.e. slower rate for sands) and this is likely due to the percentage of fine-grained materials present within these soils. The tested permeability of the bedrock corresponded well with the range of values presented above. The coefficients of permeability adopted in our model are considered to be realistic and representative values for the site.

Calculated and Adopted Coefficients of Permeability

Strata	Calculated Range for Coefficient of Permeability (k) (m/sec)	Adopted Coefficient of Permeability (k) – Realistic Case Note 1 (m/sec)	Ratio of Vertical to Horizontal Permeability (k_v/k_h) Note 2
Soils (Silty Sands, Clayey Sands, Clay)	8.7×10^{-7} to 1.6×10^{-6}	2×10^{-6}	1
Weathered Bedrock (Interbedded Siltstone, Sandstone and Laminite)	2.4×10^{-7} to 3.8×10^{-7}	4×10^{-7}	0.5

Note 1 – We carried out a sensitivity of the permeability for the soils and weathered bedrock using a k = 7×10^{-6}m/sec and 9×10^{-7}m/sec, respectively.

Note 2 - We carried out a sensitivity of the ratio of vertical to horizontal permeability in the weathered bedrock using a $k_y/k_h = 1.0$

While we consider the coefficients of permeability adopted in the table above to be both realistic and accurate, they are still only approximate. Consequently, these values should be considered to be a "reasonable estimate" and have not been selected to be conservative or optimistic. It must be recognised that there are natural variations in the permeability rates of rock or soils that may not have been captured in the pump-out tests and that variations of at least one order of magnitude should not be considered unexpected. Sensitivity checks of the permeability of the soils and weathered bedrock and ratio of vertical to horizontal permeability, K_v/K_h, of the weathered bedrock have been carried out to show the effect of these values on extraction rates.

6.4 Model Geometry

The model geometry assumes that over the eastern portion of the site where excavation is through bedrock that these faces will be fully drained and that over the western portion of the site where sandy soils and a high groundwater table are present that a ‘cut-off wall’ will be installed to a depth of 3m below bulk excavation level, (i.e.RL-4m). It is anticipated that in the long term the basement will be tanked.

In this case we have modelled the basement excavation with a vertical cut on the eastern side and a ‘cut-off wall’ on the western side. Total head boundary conditions have been adopted at the sides and base of the model. Infinite elements have been utilised at the sides and base of the model to represent the total head conditions beyond the model boundaries. The model geometry adopted is presented in Figure 9.

6.5 Analysis Assumptions

The following assumptions have been made in the seepage analysis:

- Initial boundary groundwater levels vary between RL6m and RL0.7m at the eastern and western sides of the site respectively. These groundwater levels should be checked prior to, during and on completion of construction.
- Bulk excavation level for the basement is taken as RL-1m, which is 0.35m below the proposed basement level.
- A cut-off wall is installed at the western face of the excavation, will extend 3m below bulk excavation level and form an impervious barrier. The eastern cut face is pervious and the excavation will be maintained in a dry state by draining the excavation by means of sump and pump in the base of the excavation.
- The horizontal permeability is twice that of the vertical permeability for the weathered bedrock but in the soil the horizontal and vertical permeability is equal.

6.6 Predicted Extraction Rates

The table below indicates the predicted pump out rates required to maintain the basement in a dry condition. It should be noted that higher pumping rates will be required initially until steady state conditions are achieved.

Predicted Daily and Annual Extraction Rates with 'Realistic Case' Adopted Coefficient of Permeability Values	
Pumping Rate (Litres/Day)	**Pumping Rate (Megalitres/Annum)**
18,200	6.7

Based on the modelling completed using the 'Realistic Case' coefficients of permeability, annual pumping rates are in the order of about 6.7ML. Figure 10 presents a graphical representation of the seepage.

Sensitivity analysis was also completed and considered two alternative models. These were:

- Increasing the coefficient of permeability of the soil from 2 x 10^{-6}m/s to 7 x 10^{-6}m/s,
- Increasing the coefficient of permeability of the rock mass from 4 x 10^{-7}m/s to 9 x 10^{-7}m/s
- Assuming that the horizontal permeability is ten times that of the vertical permeability for the weathered bedrock.

The tables below present the pump out rates for these cases.

Sensitivity of Soil Permeability – **Predicted Daily and Annual Extraction Rates with *k = 7 x 10^{-6} m/sec* and k_y/k_h = 1 (for soil), and,** **k = 4 x 10^{-7} m/sec and k_y/k_h = 0.5 (for rock)**	
Pumping Rate (Litres/Day)	**Pumping Rate (Mega Litres/Annum)**
18,500	6.8

Sensitivity of Rock Permeability – **Predicted Daily and Annual Extraction Rates with k = 2 x 10^{-6} m/sec and k_y/k_h = 1 (for soil), and,** ***k = 9 x 10^{-7} m/sec* and k_y/k_h = 0.5 (for rock)**	
Pumping Rate (Litres/Day)	**Pumping Rate (Mega Litres/Annum)**
40,400	14.8

Sensitivity of Ratio of K_y/K_x for Rock – **Predicted Daily and Annual Extraction Rates with k = 2 x 10^{-6} m/sec and k_y/k_h = 1 (for soil), and,** **k = 4 x 10^{-7} m/sec and *k_y/k_h = 0.1 (for rock)***	
Pumping Rate (Litres/Day)	**Pumping Rate (Mega Litres/Annum)**
6,200	2.3

Further comments on hydrogeology are discussed in *Section 7 Comments and Recommendations* below.

6.7 Predicted Water Take

The results of the analysis predict that the water take will be 18,200 litres/day. As it is estimated that temporary dewatering will be required for a period of no longer than 9 months (270 days), the total water take is estimated to be about 5.0 ML.

6.8 Water Take Measuring and Recording

Measurement of pump out volumes should be completed using calibrated flow meters. These readings will be completed and recorded daily to confirm cumulative discharge values and variations in pump out rates.

7 COMMENTS AND RECOMMENDATIONS

We consider that the proposed development 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. The intent of these recommendations should be adopted by the structural and hydraulic engineers in the development of their design drawings and should be referred to by the Development Consent. These recommendations address geotechnical issues only and other conditions may be required to address other aspects of the proposed development.

7.1 Conditions Recommended to Establish the Design Parameters

7.1.1 Prior to the commencement of excavation, retention systems will need to be installed around the proposed basement excavation. Over the eastern portion of the site, where bedrock is shallow and present above the design groundwater table (assumed to be about RL2.85m), a soldier pile wall may be adopted. Where cut depths are less than 3m the wall may be designed as cantilevered while where excavation is greater than 3m the wall must be designed as anchored or propped. Alternatively, consideration could be given to the adoption of a soil nail wall that is progressively installed as the excavation deepens. The existing soldier pile retaining wall to the east of the site will be removed as part of the excavation and careful consideration must be given to how this will be achieved while continuing to maintain support to the excavation. Over the western portion of the site where bedrock is present below the design groundwater table (assumed to be about RL2.85m), a cut-off wall, such as a secant pile wall will need to be installed. It is assumed that this wall will extend a minimum of 3m below bulk excavation level. Advice on retaining wall options is provided below in Section 7.1.5.

7.1.2 Upslope of the existing soldier pile wall is a steep soil bank that slopes down towards the west at about 45°. The new retention system will typically extend to the eastern boundary resulting in its removal however, at the northern end of the site a thin strip will be left in place. While it is understood that this slope is an engineered slope that has been reinforced with horizontal geogrids, this must be confirmed during construction. If this is not the case and the slope is not reinforced it is over-steep and prone to failure without warning. If as-built records cannot be obtained and it cannot be confirmed that the design has an acceptable factor of safety, further investigation is required to confirm the design of the slope. To this end we recommend that in the initial stage of construction a number of test pits be carried out within the proposed excavation footprint to expose the reinforcement that has been installed in the slope and the subsurface conditions. Where geogrids are present these should be cut prior to excavation. The geogrids will not require reinstatement as the test pits will be limited in width to no greater then 1m, will be orientated up and down the slope and will be located in a portion of the site where support will be provided by the installation of a soldier pile wall.

7.1.3 The upper and lower sandstone boulders located approximately midway along the length of the southern boundary will be trimmed back to the extent that they protrude into the building footprint. This will also result in the removal of the shotcrete and mesh facing present below the upper sandstone boulder. To trim these boulders back and provide both temporary and permanent support the works will need to be completed in a carefully staged manner. Details on the proposed excavation and retention staging for this portion of the site, including hold points are presented in Appendix D.

7.1.4 Should temporary batters be adopted for localised excavations within the basement excavation, subject to inspection by a geotechnical engineer, temporary batters should be formed at no steeper than 1 Vertical (V) in 1.5 Horizontal (H) within the soil profile above the groundwater table and 1V:1H within the extremely weathered rock. Temporary batters in soils below the groundwater table should not be attempted. If steeper batters are required, further advice will be necessary from the geotechnical engineer and some form of temporary support will be necessary. All surcharge loads should be kept well clear of the crest of the temporary batters. As a guide, surcharge loads should be positioned no closer than 2H from the top of any batter, where H is the vertical height of the batter. Where ramps are required to provide access for machinery across the site, they will need to be formed from good quality granular material. Further advice on the formation of ramps and working platforms should be sought from this office. Based on the proposed basement footprint, sufficient space does not exist for the adoption of temporary batters for the proposed basement excavation.

7.1.5 As temporary batters will not be suitable for the proposed basement excavation, a retention system will be required and must be installed prior to excavation commencing. We recommend the retention system comprise the following:

- Anchored/propped (retained heights of greater than 3m) or cantilevered (retained heights of less than 3m) soldier pile wall with reinforced shotcrete infill panels over the eastern portion of the site where bedrock and overlying clayey soils are present above the design groundwater table (i.e. approximately RL2.85m). The infill panels must be progressively installed as excavation proceeds (i.e. at maximum 1.8m depth intervals). Where anchored or propped retaining walls are adopted, the anchors or props should be installed progressively as the excavation deepens.
- Secant pile walls could be adopted over the western portion of the site where the design groundwater table (i.e. approximately RL2.85m) is above the bedrock level. The secant pile wall should be socketed into the underlying bedrock below bulk excavation level and act as a ‘cut-off wall’ to prevent groundwater from flowing into the basement excavation. Secant pile

walls can sometimes be problematic when the verticality of the piles is not within tolerance such that they become misaligned and no longer interlock to form a 'watertight' barrier. Where this occurs, it may then be necessary to jet grout behind the wall to seal the wall. However, considering the limited depth to which the piles will be installed this is less likely to be a problem on this site provided competent piling contractors are engaged. Cutter soil mix (CSM) walls are less prone to this problem due to the rectangular shape of the constructed panels and also have a relatively good finish. CSM walls can be problematic where they are required to be socketed into bedrock. In this regard, further advice should be sought from suitably experienced piling contractors on the suitability of the adoption of such a wall on this site.

- We recommend that in the early stage of construction, test pits be excavated along the southern and northern boundaries to investigate the ground conditions to confirm where the transition between the soldier and secant pile walls should be.
- Anchors or props will be required for the soldier pile (eastern boundary and portions of the northern and southern ends), where retained heights exceed 3m. Elsewhere cantilevered retaining walls may be considered.
- Secant piles through sands will need to be drilled using grout injected piling techniques (i.e. Continuous Flight Auger (CFA) techniques). Elsewhere, should piles be adopted as foundation piles, CFA piles are also recommended due to the high groundwater table and underlying weak bedrock which is prone to softening on contact with groundwater. If bored piles are adopted for the soldier pile wall, it is likely that these will need to be tremie poured immediately following piling, as they will extend below the groundwater table. The use of pumps or a cleaning bucket to remove all the water in the pile holes will be difficult and may not be feasible.
- Attention will need to be given during secant pile construction through the water charged sand to ensure that excess spoil is not removed from the pile holes during installation (termed decompression). Decompression can cause a settlement bowl in the ground around the piles, and in extreme cases this settlement bowl can extend some metres from the pile location. The site superintendent should monitor ground settlement during piling to ensure that adverse settlements do not occur. Particular care will be required when piling adjacent to Barrenjoey House that this does not occur. This risk can be managed using a double rotary system which rotates in temporary casing in conjunction with the auger.

7.1.6 The proposed new retaining walls should be designed using the following parameters:

- For cantilever walls, adopt a triangular lateral earth pressure distribution. Where movement sensitive structures are not present within the zone of influence of the excavation (which is defined as everything above a line drawn upwards from bulk excavation level at 1 Vertical(V):2 Horizontal(H)), a coefficient of active pressure, K_a, of 0.35 may be adopted for the retained soils and weathered bedrock, assuming a horizontal surface behind the wall. Where movement sensitive structures are located within the zone of influence of excavation, a coefficient of lateral earth pressure, K, of at least 0.55, for the retained soils and weathered bedrock should be adopted, assuming a horizontal surface behind the wall.

- Propped or anchored walls supporting soils and weathered bedrock at the eastern portion of the site should be designed to resist a trapezoidal earth pressure distribution. Where movement sensitive structures are not present within the zone of influence of the excavation, a lateral pressure of 6H kPa may be adopted. Where movement sensitive structures are present within the zone of influence of the excavation, a lateral earth pressure distribution of 8H kPa should be adopted. An appropriate surcharge load must be added to the above pressures to represent a sloping backfill, should it be present, as detailed below.
- All surcharge loads, such as from the sloping ground above the wall at the eastern portion of the site, construction equipment, stockpiles, structures (including 'Barrenjoey House' footings), etc. and appropriate hydrostatic pressures should be added to the above pressures.
- Bulk unit weights of $20kN/m^3$ and $22kN/m^3$ should be adopted for the soil and weathered bedrock profiles, respectively.
- The soldier pile retaining walls should be provided with complete and permanent drainage of the ground behind the walls and exit at the base of the shotcrete for controlled discharge to Council's stormwater system. The subsoil drains should incorporate a non-woven geotextile fabric (eg. Bidim A34), to act as a filter against subsoil erosion.
- Toe resistance of the wall may be achieved by keying the footing into the weathered bedrock below bulk excavation level. In this regard an allowable lateral stress of 200kPa may be adopted. This assumes that full passive restraint can be mobilised in the rock and that features such as excavations in front of the wall do not reduce the available capacity. In this regard we recommend that when calculating the required depth of embedment needed for lateral restraint the first 0.5m of the socket below bulk excavation and all known localised excavations be ignored. This allows for disturbance during tracking or for accidental over-excavation. All retaining wall designs should be reviewed by a geotechnical engineer prior to construction to confirm that appropriate design values have been adopted.
- Anchors or bolts may be designed based on an allowable bond strength of 70kPa in weathered bedrock. Temporary anchors used for lateral support should be bonded beyond a line drawn up at 45° from the bulk excavation level. All anchors should be proof stressed to at least 1.3 times their working load and then locked off at about 80% of the working load.
- Where temporary anchors extend below adjoining properties permission from the respective property owners must be obtained before installation.
- Long term support is understood to be provided by the built structure. Once constructed temporary anchors will then be destressed.

We have found that detailed retaining wall designs using geotechnical software such as PLAXIS can produce more economical wall designs than by using the apparent earth pressure recommendations above. However, WALLAP, does not have input parameters to review scenarios where there are adverse joints, large wedge failures, etc, and therefore specific checks need to be carried out to review these. Otherwise, these can be reviewed using PLAXIS. We consider that the following preliminary geotechnical design parameters could be adopted for shoring wall design using such software packages. However, as noted above, only designers with experience in modelling jointed rock masses should undertake this design.

Preliminary Shoring Wall Design Parameters					
Material Type	**Unit Weight (above/below GW) (kN/m^3)**	**Effective Friction Angle (degrees)**	**Effective cohesion (kPa)**	**Elastic Modulus (MPa)**	**Poisson's Ratio**
Soils (including fill)	18/20	27	0	8	0.3
Weathered Bedrock Note 1	22/22	26	30	75	0.3

Note on table above:

Note 1: The above assumes an intact rock mass free from adverse defects, such as adversely orientated joints forming large wedge failures, etc. Therefore, we recommend that the designer consider the support requirements necessary should adverse defects be present, detail additional support measures that may be necessary should such defects be present and ensure that the cut faces between the soldier piles be progressively inspected during excavation by a geotechnical engineer so that should adverse defects be present they may be identified and additional support measures installed. Alternatively, the designer can assume that such defects will be present.

7.1.7 Excavation will encounter predominately weathered bedrock over the eastern portion of the site. Some soils will be present in the upper hillside east of the soldier pile wall and soils for the full depth of excavation are anticipated over the western portion of the site. Excavation of the soils and weathered bedrock of up to low strength should be achievable using conventional excavation equipment, such as medium sized excavators (say 15 to 20 tonnes) with buckets and "tiger teeth" attached. Where the bedrock is of low or higher strength, rock excavation techniques will be required.

Rock excavation techniques may consist of percussive or non-percussive equipment. Percussive techniques comprise the use of rock hammers, while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting etc. Where percussive excavation techniques are adopted there is the risk that transmitted vibrations may damage nearby movement sensitive structures such as the 'Barrenjoey House' building to the north and the residential buildings to the east and south. Therefore, where percussive excavation techniques are used continuous vibration monitoring will be necessary. The prescribed vibration limits that should be adopted are set out in the Vibration Emission Design Goals attached to the rear of this report. Since the 'Barrenjoey House' is a heritage building, and since nearby buildings may be supported on loose granular soils, particular care will be required to keep vibrations as low as possible to reduce the risk of damage to these structures. Consequently, it is our recommendation that percussive rock excavation techniques not be used in proximity of these structures. Where rock is unable to be ripped, saw cutting techniques in conjunction with ripping can be adopted.

Reference should be made to the Acid Sulphate Soils assessment by JK Environments and any other environmental reports prior to excavations.

7.1.8 Groundwater monitoring wells have been installed during our investigation in BH101 and BH102. During the time of our investigation and subsequent monitoring, the results indicate that the groundwater level gradient is falling towards the west, typically ranging from depths between about 0.85m (BH102, RL1.85m) and 2.0m (BH101, RL0.7m). However, we anticipate that the groundwater level may fluctuate by at least 1m, especially following extended periods of wet weather and climatic changes, and may rise to the approximate level of Barrenjoey Road (i.e.

approximately RL2.85m). According to the Northern Beaches Council database the western portion of the site is within a flood zone. Therefore, we recommend the design groundwater level be adopted at or above RL2.85m.

With regards to climate change related groundwater level rise, expert advice should be obtained. As a preliminary guide, global sea levels are anticipated to increase by between 45cm and 88cm by 2090 (Ref: CoastAdapt, Information Manual 2, National Climate Change, 2016), although it is likely that local variations will occur along the coastline.

Based on monitoring to date and the Council database, groundwater will be above bulk excavation level. As such, dewatering will be required to enable excavation and construction to be completed in a 'dry' condition. Furthermore, the basement structure will need to be tanked and designed to resist uplift forces in the long term. The structure must be designed to resist the hydrostatic uplift pressures that will result from the anticipated design groundwater table. Where the building has insufficient mass to resist uplift pressures, vertical anchors or piles will be required and must be bonded into the underlying weathered bedrock to resist the uplift pressures. Design of these anchors will be governed by both the bond resistance of the rock and structure/strength of the underlying rock mass. Once anchor details, such as spacing, location and design loads are available these should be reviewed by the geotechnical engineer. As a preliminary guide, anchors socketed into weathered bedrock may be designed for an allowable bond stress of 70kPa, but as discussed above, the structure and strength of the rock mass must also be considered in the design.

Temporary dewatering during construction will be necessary following installation of the shoring system. A dewatering licence will need to be obtained from WaterNSW for all temporary dewatering activities. Estimated extraction volumes based on the existing groundwater levels are provided above in Section 6.6.

7.1.9 All proposed footings must be founded in the underlying bedrock. Footings founded in bedrock of very low strength or better may be designed for an allowable bearing pressure of 700kPa, subject to inspection by a geotechnical engineer prior to pouring concrete. Where footings are located close to the edge of localised excavations, they must be wholly founded below a line drawn upwards from the base of the excavation at 1V:1H.

While pad footings should be suitable for most of the site, there is some uncertainty regarding the depth of bedrock closer to Barrenjoey Road. Where bedrock is deeper than 1m, pile footings may be more appropriate. Where pile footings are adopted and are socketed a minimum of 0.3m into weathered bedrock of at least very low strength an ABP of 700kPa may be adopted. Where piles extend below a nominal 0.3m socket into rock, allowable shaft adhesions of 10% and 5% of the ABP may be adopted for compressive and tensile (uplift) loads, respectively. This assumes that the rock socket is suitably roughened. Due to the presence of a high groundwater level and sandy soils, CFA grout injected piles will be required.

7.1.10 The surface water discharging from the new roof and paved areas must be diverted to outlets for controlled discharge to Council's stormwater system. In addition, we recommend a drain east of the proposed building be provided to intercept surface water run-off from upslope and connected to the stormwater system.

7.1.11 At bulk excavation we anticipate that bedrock will be exposed over the eastern portion of the site and soil will be exposed over the western portion. We anticipate that where soil is present at bulk excavation level that bedrock will be at shallow depths.

Since there are some uncertainties regarding the depth of fill and depth of rock closest to Barrenjoey Road, where soils are exposed at bulk excavation level, we feel it prudent to carrying out some subgrade preparation, but recognise that this is not strictly necessary as the basement slab will be designed for full uplift pressures and will be fully suspended on the underlying bedrock. In this regard, where soils are exposed, the subgrade may be proof rolled. All proof rolling should be carried out without vibration. The purpose of proof rolling is to increase the near surface density of the subgrade. Care should be taken not to use rollers that are excessively large. We suggest a roller of only about 5 tonnes be used.

Where bedrock is exposed at bulk excavation level, no proof rolling is required, although a coarse granular de-bonding layer should be placed over the surface of the rock. This same coarse granular layer may be placed over the remainder of the site and rolled in to form a sound working platform, particularly where the exposed subgrade is not particularly trafficable.

Where pavements are located outside of the basement excavation and will be supported on the natural soils, we recommend that all topsoil and root affected soils first be stripped from site. Following stripping, the exposed subgrade should be proof rolled using a minimum 5 tonne smooth drum roller. The purpose of proof rolling is to identify any loose or unstable zones. Where unstable zones are identified they should be excavated down to a sound base and replaced with engineered fill. Should such zones be identified, further advice will be provided on site and insitu density testing will be required to confirm that the fill has been placed to the required specification.

7.1.12 According to AS1170.4-2007 'Structural Design Actions – Part 4: Earthquake Actions in Australia', including Amendment No. 1 (August 2015) and Amendment No. 2 (February 2018) the site classifies as Class D_e. This is based on soil with an SPT N-value of less than 6, but with a soil layer less than 10m thick.

The Hazard Factor, Z, for Sydney is 0.08.

7.1.13 The laboratory test results indicate that for buried concrete structures the soils have a 'Mild' exposure classification in accordance with Table 6.4.2 (C) of AS2159-2009 "Piling Design and Installation''. For buried steel structures the soils have a 'Non-Aggressive' exposure classification in accordance with Table 6.5.2 (C) AS2159-2009.

7.1.14 The guidelines for Hillside Construction given in Appendix B should also be adopted.

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

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

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

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

7.2.4 Dilapidation surveys must be carried out on the neighbouring buildings and structures to the north and south. A copy of the dilapidation report must be provided to the relevant property owners who should be asked to review and sign the reports to confirm that they represent an accurate record of existing conditions. These reports may also be required to be submitted to Council or the Principle Certifying Authority.

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

7.2.6 The excavation/retention methodology must be reviewed and approved by the geotechnical engineer.

7.3 Conditions Recommended During the Construction Period

7.3.1 The approved excavation/retention methodology must be followed.

7.3.2 With regards to the sloping ground to the north of the proposed shoring system, if as-built records cannot be obtained we recommend that in the initial stage of construction a number of test pits be excavated to review the subsurface conditions. Since geogrids have likely been installed these should be cut prior to excavation.

7.3.2 Where percussive excavation techniques are used, continuous vibration monitoring will be necessary. The prescribed vibration limits set out in the Vibration Emission Design Goals attached to the rear of this report should be adopted. Since the ‘Barrenjoey House’ is a heritage building, and may be supported on loose granular soils, we recommend that a peak particle velocity (PPV) of 3mm/s be adopted as the trigger level.

7.3.3 The geotechnical engineer is to witness the installation of the retaining wall piles to confirm that the design criteria are satisfied.

7.3.4 All rock anchors must be proof-tested to 1.3 times the working load. In addition, the anchors must be subjected to lift-off testing no sooner than 24 hours after locking off at the working load. The proof-testing and lift-off tests must be witnessed by the geotechnical engineer. The anchor contractor must provide the geotechnical engineer with all field records including anchor installation and testing records.

7.3.5 The bedrock exposed between the soldier piles and for localised excavations below bulk excavation level (i.e. lift pits, etc.) must be progressively inspected by the geotechnical engineer as excavation proceeds. We recommend inspections at 1.5m vertical depth intervals in rock and on completion of the bulk excavation works/localised deeper excavations. The purpose of this inspection is to check for the presence of adversely orientated defects within the rock mass. Where the retaining wall has been designed to support the most adverse case of adversely orientated defects these inspections will not be necessary.

7.3.6 We recommend that the groundwater levels around the site be carefully monitored prior to and during the construction period to review the groundwater levels that were modelled and establish exactly what effect dewatering is having, respectively. All monitoring results should be reviewed by the geotechnical and hydraulic engineers and, where the measured groundwater levels or pump out rates differ markedly from those predicted in our analysis, modifications to the proposed dewatering system may need to be made.

7.3.7 The geotechnical engineer must inspect all footing excavations including piled footings (if carried out) prior to placing reinforcement or pouring the concrete.

7.3.8 Proposed material to be used for backfilling behind retaining walls, if carried out within the building structure, must be approved by the geotechnical engineer prior to placement.

7.3.9 Compaction density of any fill placed must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer.

7.3.10 If they are to be retained, the existing stormwater system, sewer and water mains must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer or architect, and repaired or replaced if found to be leaking.

7.3.11 The geotechnical engineer must inspect all subsurface drains prior to backfilling.

7.3.12 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).

7.3.13 The geotechnical engineer must confirm that the proposed works 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 surface (including roof) and 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 No cut or fill in excess of 0.5m (eg. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Council.

7.4.3 Where the structural engineer has indicated a design life of less than 100 years then the structure and/or structural elements must be inspected by a structural engineer at the end of their design life; including a written report confirming scope of work completed and identifying the required remedial measures to extend the design life over the remaining 100 year period.

9 Response to the Comments Raised by CGC, eiaustralia and Notice of Determination

9.1 Response to the Comments Raised by CGC

In the email prepared by CGC, the following comments were made. We have addressed each in turn:

Following the site investigations to date no geotechnical data is available over the eastern side of the site

The subsurface conditions in Palm Beach are well known and consistent and comprise Hawkesbury Sandstone over the upper or higher portions of the peninsula with rocks of the Narrabeen Group present over the lower portions. A relatively thin layer of soil typically overlies the bedrock. The purpose of the subsurface investigation is to confirm that there are no subsurface conditions peculiar to the site.

While it is true the no investigation has been completed above the retaining wall in the eastern portion of the site, the distance from the existing soldier pile wall to the eastern site boundary is about 6m. Consequently, this area forms a relatively small proportion of the site. Notwithstanding this, information is available on the subsurface conditions present behind the retaining wall with the supplied photographs indicating that bedrock is present at the underside of the pile cap, which is at RL6.4m. This means that bedrock in this portion of the site is at least at this level, if not higher which means that in this area bedrock is at a depth of no greater than 1.1m below ground level. When this information is combined with the boreholes from the previous subsurface investigations completed over the western part of the site, a good understanding of the subsurface conditions is obtained across the site as a whole.

In addition, the report prepared by CGC for 1110 Barrenjoey Road provides subsurface information on the conditions immediately to the east of the site. This indicates that sandstone bedrock is present at relatively shallow depth and extends to at least RL8.6m.

Anchoring across the boundary may impact the approved development at 1110 Barrenjoey Road, Palm Beach

We agree that there is the potential for the installation of anchoring below 1110 Barrenjoey Road, Palm Beach to impact the proposed development. Anchors are normally installed inclined downwards from the horizontal at between about 15° to 30°, but can be installed at steeper angles where necessary. Where they are installed inclined down at between about 15° and 30°, for every horizontal meter of anchor it increases in depth by between about 0.25m and 0.6m.

The retaining wall capping beam along the common boundary with 1110 Barrenjoey Road will be at about RL11.5m. It is likely that where anchors are used they will be installed about 1.5m below the top of the wall, or at about RL10.0m. The main building has a first floor level of RL20.1m and even considering the depth to which piles may be installed, any anchors that extend below the house are likely to be at no higher than about RL7.5m. It is highly unlikely that these anchors will impact construction of the proposed main residence. The same is true for the garage, which although it has a finished floor level of RL12.15m, it is setback about 11m from the boundary and will be cut into the hillside with maximum depths of excavation in the order of about 6m. Sandstone bedrock is expected to be exposed over much of the base of the excavation and similarly, it is highly unlikely that these anchors will impact construction of the proposed garage.

The granny flat is proposed to be constructed in the south-western corner of the site setback about 1m from the common boundary with 1102-1106 Barrenjoey Road and has a finished floor level of RL12.27m. Consequently, where anchors are installed below the granny flat they will pass below the closest side of this building at about RL9.75m, it is possible that there could be some interaction, although from the sections prepared by CGC it appears likely that footings will be founded no deeper than about RL9m. Notwithstanding this, this risk could be mitigated in this area by increasing the angle of the anchors or the depth below the top of the wall at which the anchors are installed so that they pass below any piles installed to support the granny flat.

Where a soldier pile wall is adopted there is the possibility that collapse of soils may occur between the piles prior to the installation of the shotcrete and mesh panels

While it is possible that some loss of soil may occur between the soldier piles the spacing of piles is typically controlled such that this risk is mitigated. For clayey soils, such as those found onsite, the installation of a soldier pile wall is commonly adopted and few issues are experienced, particularly where the shotcrete and mesh are placed in controlled and staged manner. They are such a common and proven retention system that we note that CGC have also recommended the adoption of a soldier pile wall at 1110 Barrenjoey Road as a suitable shoring system.

Without the completion of a shoring design there is no way of assessing the potential impact of the excavation on 1110 Barrenjoey Road

While it is true that at this stage in the project a shoring design has not been completed for the above site, the recommendations provided in this report provide the required information for the design of such a system to limit displacements within acceptable limits, depending on the sensitivity of the surrounding structures. We also note that as recommended in the Geotechnical Report prepared by CGC, all structures at 1110 Barrenjoey Road are to be uniformly founded on the underlying bedrock, as is good hillside practice. Consequently, while a detailed assessment of the performance of the proposed shoring system cannot be completed at this stage, where the recommendations of this and CGC reports are followed with regards to the design of retaining walls and with the uniform founding of all structures on bedrock, the potential impact of the excavation on 1110 Barrenjoey Road should not have an adverse impact to the proposed buildings on 1110 Barrenjoey Road.

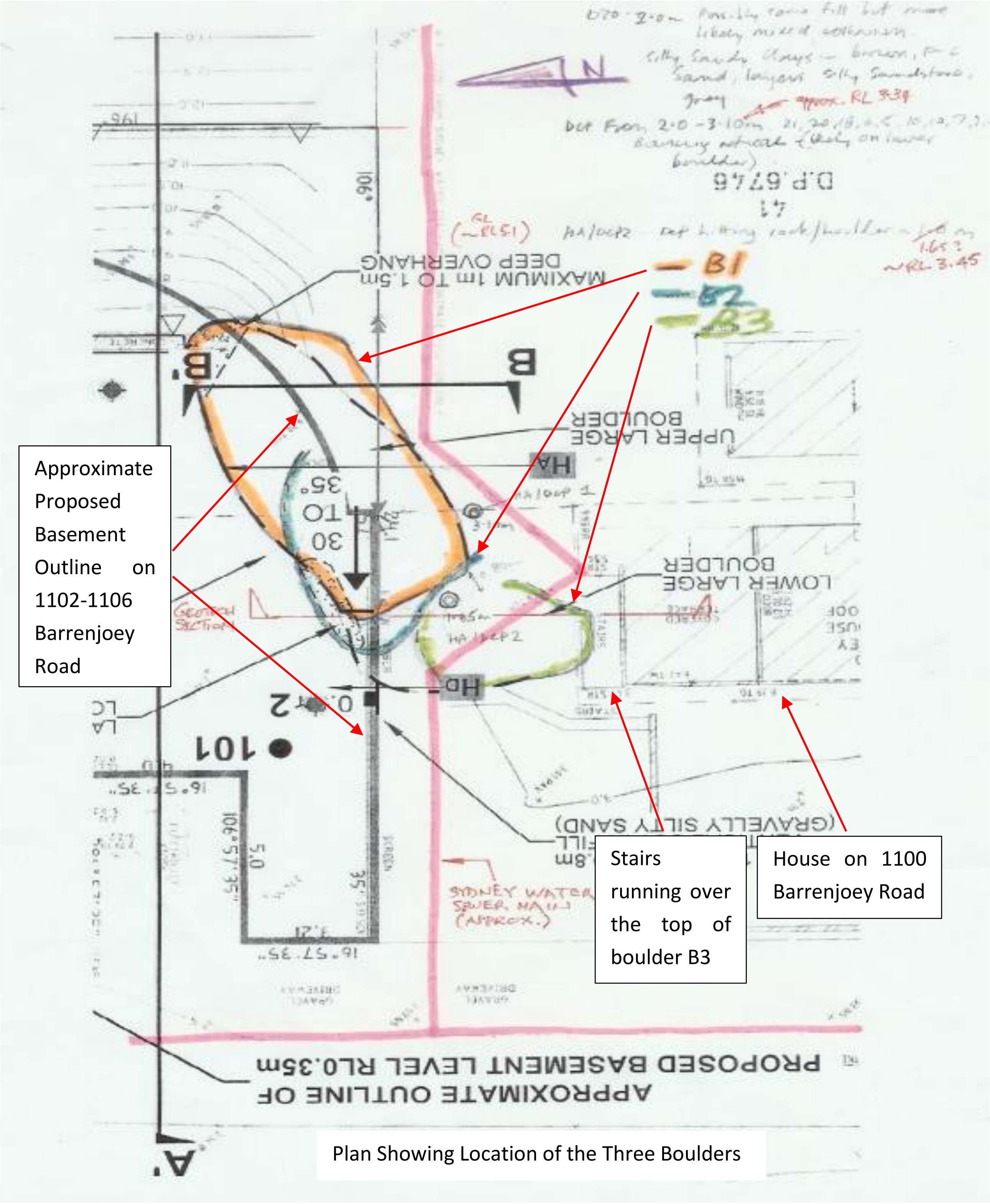
9.2 Response to the Comments Raised by Eiaustralia

While the reports prepared by eiaustralia contain many assertions, it appears that the key concerns relate to the following. We have addressed these comments below.

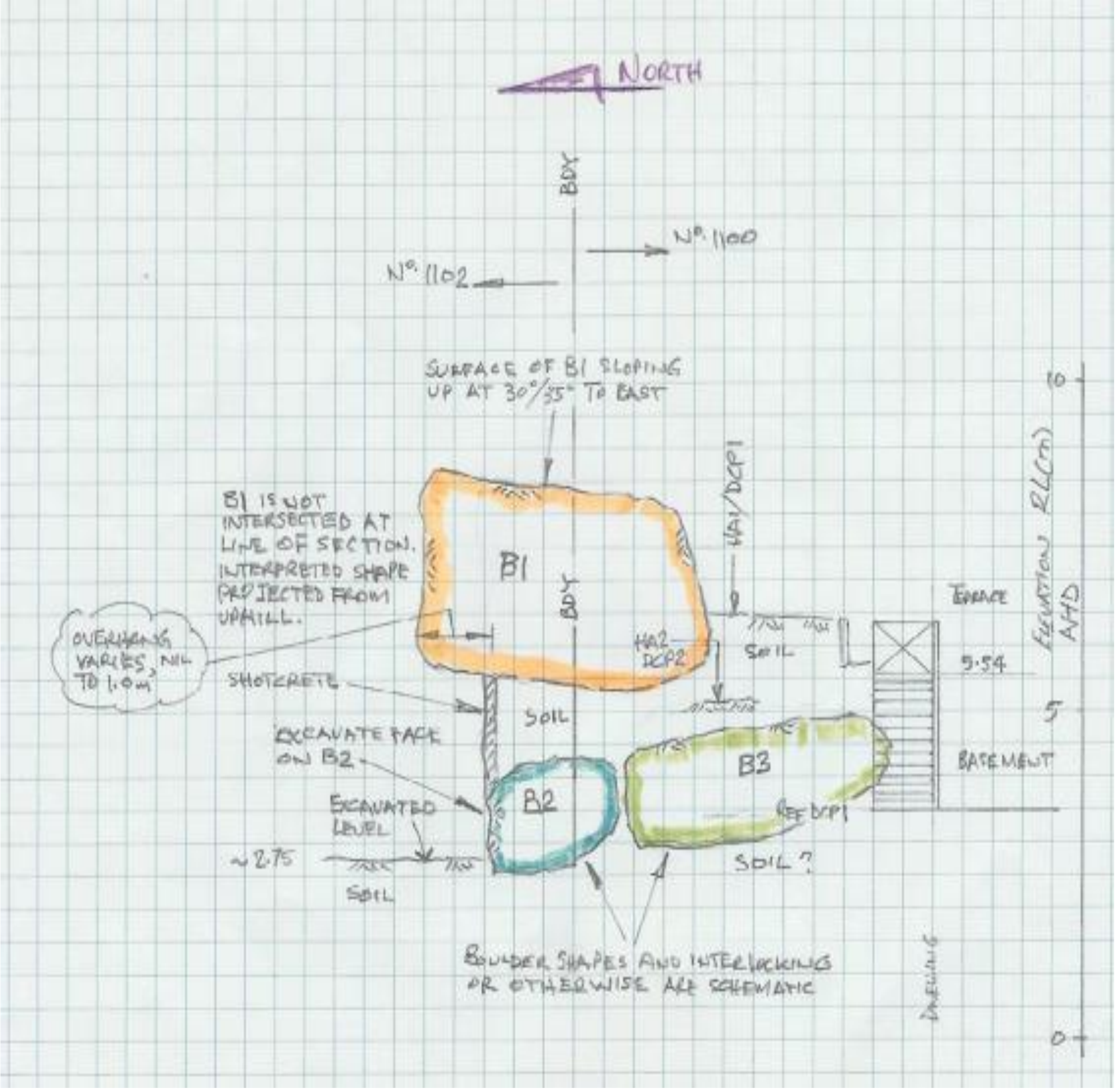
That the current geotechnical reports lack information or any recognition of the critical element of site stability with regards to the boulders that are present approximately midway along the southern boundary

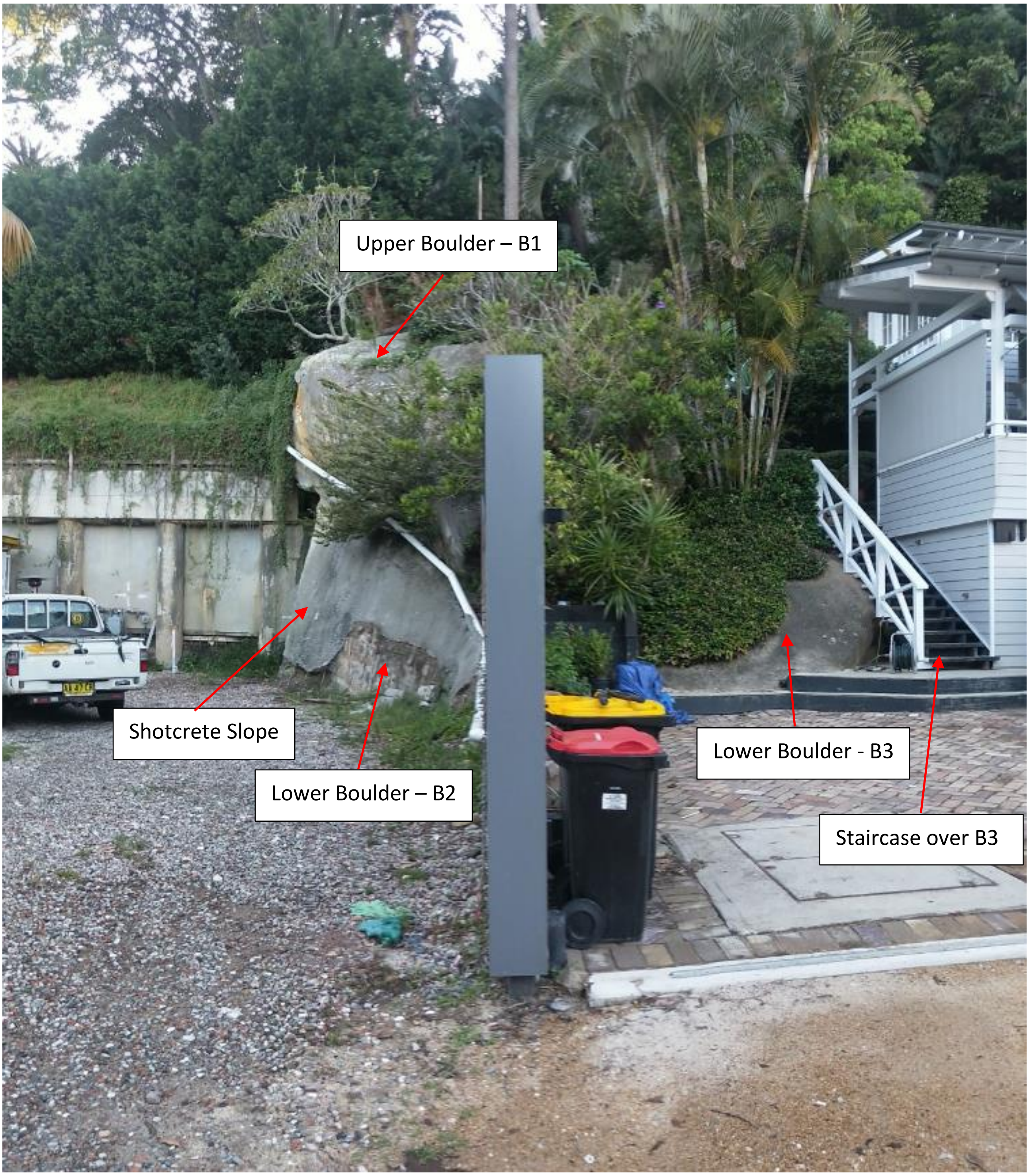
At the heart of the comments prepared by eiaustralia is the expression of concern that the proposed development poses an unacceptable risk to two boulders that span the boundary between 1100 and 1102- 1006 Barrenjoey Road. To understand the risk posed by these boulders it is necessary to understand their location and how they may interact with one another and adjoining structures.

In a report prepared by Warwick Davies of eiaustralia (Ref: P21153.01_DRAFT, dated 16 January 2023), Warwick Davies has provided a review of a memo that he prepared in 2021 when he worked for Davies Geotechnical Consulting Engineers (DG) (Ref: 21-019.C_rev1, dated 327 October 2021). In that memo, an investigation was undertaken to confirm the extent of the boulders present along the boundary. The results of this investigation and the location of these boulders, as extracted from that report are shown below and indicate that three boulders are present; two of which are located on the boundary and span both 1100 and 1102-1106 Barrenjoey Road (B1 and B2) while the third boulder (B3) is located wholly within 1100 Barrenjoey Road and is at roughly the same level as the lower of the two boulders on the boundary. None of these boulders extend below the residence at 1100 Barrenjoey Road nor is this building founded on these boulders, although the timber staircase does span across and run over the top of B3. To provide more context to these sketches we have also included an annotated photo that shows the conditions across the boundary.



Plan Showing Location of the Three Boulders


NORTH
BDY
No. 1100
No. 1102
SURFACE OF B1 SLOPING
UP AT 30°/35° TO EAST
HA1/DCP1
B1 IS NOT
INTERSECTED AT
LINE OF SECTION.
INTERPRETED SHAPE
PROJECTED FROM
UPHILL.
B1
BDY
OVERHANG
VARIES, NIL
TO 1.0m
HA2
DCP2
SOIL
TERRACE
5.54
ELEVATION RL(m)
AHD
10
5
0
SHOTCRETE
SOIL
EXCAVATE FACE
ON B2
B3
BASEMENT
B2
EXCAVATED
LEVEL
REF DCP1
~2.75
SOIL
SOIL ?
BOULDER SHAPES AND INTERLOCKING
OR OTHERWISE ARE SCHEMATIC
DWELLING



In eiaustralia's assertion that the geotechnical report lacks information and any recognition of the critical element of site stability it appears that the issues being raised are:

That there is some uncertainty regarding the design of the shotcrete and mesh and materials present below B1

While eiaustralia maintain that there is uncertainty regarding the materials present below B1 and the thickness of the shotcrete and mesh panel, this is simply not the case. The thickness of the shotcrete panel has been investigated and determined and is reported in *Section 4 Geology and Subsurface Conditions* above.

While our original reports discussed the need to determine the design of the shotcrete and mesh panel this recommendation was made on the basis that all or part of the panel would remain. This is not the case and the basement excavation will result in the removal of the shotcrete and mesh panel in its entirety. Consequently, the design detail of the shotcrete and mesh panel is irrelevant when considering the proposed new structure. In addition, we strongly disagree that uncertainty exists with regards to the materials present below the boulder and the thickness of the shotcrete and mesh panel. As discussed, these are described and documented above in *Section 4 Geology and Subsurface Conditions*.

That no advice has been provided to indicate how the boulders will be supported in both the short and long term

The above statement is manifestly untrue. As detailed in our updated letter presented in Appendix D (Ref: 33618Ylet2rev4, dated 26 June 2023 – the original letter (Ref: 33618Ylet2rev4, dated 31 January 2023) was updated with some minor changes for the purposes of this report), the method that will be used to support the boulders in both the temporary and permanent case is clearly detailed. This letter not only describes the manner in which the boulders will be supported but also the methodology. In essence, the cut along the southern boundary will be supported by a piled retaining wall that will either be anchored or propped, as required. Where the boulders are present it is envisaged that they will be supported by an anchored or propped soldier pile wall. Detailed recommendations for the design of these walls are provided above in *Section 7.1 Conditions Recommended to Establish the Design Parameters*.

That no advice has been provided to indicate will the boulders be safely excavated

The above statement is also manifestly untrue. A detailed methodology including controls has been set out in Appendix D (Ref: 33618Ylet2rev4, dated 26 June 2023 – the original letter (Ref: 33618Ylet2rev4, dated 31 January 2023). This methodology addresses the risks posed by vibration and disturbance to the boulders should inappropriate excavation methods be adopted. While this letter details the methods and controls that should be adopted, it must once again be highlighted that none of the three boulders extend below the house nor is the house supported on them. Consequently, the risk of damage to the house as a result of transmitted vibration or movement of the boulders themselves is very low, almost inconceivable. In addition to this very low risk it must also be appreciated that B3, which is closest to the house, does not extend over the boundary into 1102-1106 Barrenjoey Road. As a result, for the excavation of B1 and B2 to result in movement of B3 there must be direct transmission of movement between the boulders. As these boulders are discrete, as shown in the above diagrams prepared by DG, the likelihood of disturbance to B1 and/or B2 that is of such magnitude that it may result in appreciable movement of B3 is highly unlikely. Even should such movements occur, as the house is neither supported nor connected to the boulders it is almost inconceivable that it would be damaged. This may not be the same for the sewer. Notwithstanding this, the methodology set out in Appendix D has been prepared to control and mitigate the risks associated with vibration and movement.

That no advice has been provided on how to anchor the wall, if it is to be anchored

This statement is also untrue. *Section 7.1 Conditions Recommended to Establish the Design Parameters* above provides detailed advice on design parameters to be adopted for the design of anchors should they be required.

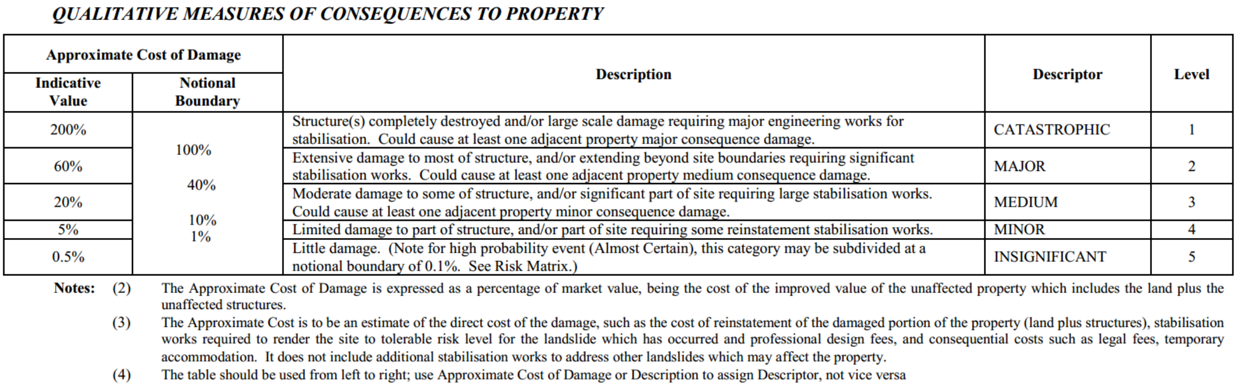
Further Comments on Assertions made in Reports Prepared by Eiaustralia

On review of the excavation methodology presented in Appendix D, eiaustralia have commented on the apparent contradiction of stating that excavation will be completed using "non-percussive excavation techniques" while at the same time also stating that "*hand held jack hammers would be permitted*". This has been taken out of context and there is no contradiction. What was stated in our letter was that *"No percussive excavation techniques other than hand-held jackhammers are to be adopted."* The rationale behind the adoption of non-percussive excavation techniques is to limit the potential magnitude of transmitted vibrations. While it is true that hand-held jackhammers are a form of percussive excavation, their small size and light weight means that transmitted vibrations generated are of very small magnitude, so small that such excavation techniques are used immediately adjacent to structures without adverse impact.

Eiaustralia have commented on the "environmental risk" posed by the proposed excavation and have assessed that this risk is possibly medium or high using the Australian Geomechanics Society's (AGS) Landslide Risk Management methodology for risk to property. The assessment of environmental risk is beyond our area expertise (and we similarly assume that it is also outside the area of expertise of Warwick Davies of eiaustralia who is a geotechnical engineer) and consequently, we make no comment on the medium to high risk that eiaustralia have assessed the boulders to pose. However, as geotechnical engineers the application of the Australian Geomechanics Society's (AGS) Landslide Risk Management methodology falls within our area of expertise. In this regard it must be recognised that the policy has been developed to assess risk to life and risk to property posed by slope instability. The methodology has not been developed to assess environmental risk.

In arriving at the assessed environmental risk eiaustralia have an assessed minor to major consequence should trimming of the boulders occur. While the AGS risk matrix has been attached to demonstrate how a medium or high risk outcome is achieved the qualitative measure for assessing the consequence has not. We have attached this measure of consequence below and from this it can be seen that consequence is related to the percentage damage resulting from the event when compared with the total improved value of the property. As a result, the risk matrix has been developed with respect to the consequences of the occurrence of an event. To simply assume that this matrix can then be directly applied to "environmental risk" shows a misunderstanding of the application of the AGS methodology. In addition, the likelihood of the occurrence has been assessed to be possible, which is considered to a reasonable estimate assuming uncontrolled excavation. This is clearly a flawed assumption as nowhere has it been stated that excavation would be completed in an uncontrolled manner. In fact the opposite is true as evidenced by the recommendations of this report and the excavation methodology presented in Appendix D of this report and developed for the proposed excavation of the boulders.

While eiaustralia have stated that the matrix could be modified they have provided no basis for how the matrix and the assessment of likelihood and consequence have been modified to allow an assessment of “environmental risk” to be made. In this regard it is hard to place much weight on this assessment, particularly when “environmental risk” itself has not been defined.



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
60%	100%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	40%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%	1%	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

9.3 Response to the Comments Raised by Notice of Determination

In the Notice of Determination, Point 4 states that one of the reasons for the refusal was that the proposed development is inconsistent with the provisions of Clause 7.7 Geotechnical Hazards of the Pittwater Environmental Plan 2014.

Clause 7.7 (4) states that:

Development consent must not be granted to development on land to which this clause applies unless

(a) the consent authority is satisfied that the development will appropriately manage waste water, stormwater and drainage across the land so as not to affect the rate, volume and quality of water leaving the land, and
(b) the consent authority is satisfied that
  i. the development is designed, sited and will be managed to avoid any geotechnical risk or significant adverse impact on the development and the land surrounding the development, or
  ii. if that risk or impact cannot be reasonably avoided—the development is designed, sited and will be managed to minimise that risk or impact, or
  iii. if that risk or impact cannot be minimised—the development will be managed to mitigate that risk or impact.

With regards to the above, a detailed risk assessment of the site has been completed and detailed comments and recommendations have been provided to mitigate these risks, where they cannot be avoided. In this regarded an engineered retention system will be constructed around the site. Installation of a properly engineered and constructed retention system will remove all identified risks currently posed by the site, with the exception of Hazard C to the minor extent that it will be located outside of the proposed retention system. In this way the proposed development will reduce the risk currently posed by the site and make it a safer site in terms of risk to both life and property. To this end we feel that the refusal of the development on the basis of Clause 7.7 – Geotechnical Risks should be reconsidered.

8 OVERVIEW

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) between borehole locations or from those inferred in preparing this report. 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 A1

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	**A**	**B**	**C**	**D**	**E**
	Stability of existing large boulder near/at the southern boundary (requires shotcrete slope below to fail, assumed as engineered)	Stability of the soldier pile wall to the east of the buildings (assumed as engineered)	Stability of the hillside upslope (east) of the soldier pile wall (assumed as engineered)	Stability of the localised 0.8m high vertical cut in fill along a 1m length at the southern boundary	Stability of new engineered retaining walls
Assessed Likelihood	Rare	Barely Credible	Rare	Likely	Barely Credible
Assessed Consequence	Minor	Minor	Minor	Insignificant	Medium
Risk	Very Low	Very Low	Very Low	Very Low	Very Low
Comments					

***Property Value Assumed to be $4.5 million**

TABLE B1

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A**	**B**	**C**	**D**	**E**
	Stability of existing large boulder near/at the southern boundary (requires shotcrete slope below to fail, assumed as engineered)	Stability of the soldier pile wall to the east of the buildings (assumed as engineered)	Stability of the hillside upslope (east) of the soldier pile wall (assumed as engineered)	Stability of the localised 0.8m high vertical cut in fill along a 1m length at the southern boundary	Stability of new engineered retaining walls
Assessed Likelihood	Rare	Barely Credible	Rare	Likely	Barely Credible
Indicative Annual Probability	10^{-5}	10^{-6}	10^{-5}	10^{-2}	10^{-6}
Duration of Use of area Affected (Temporal Probability)	Below, 5min/day (3.47×10^{-3})	Above, 1 min/month (2.28×10^{-5}) Below, 10min/day (6.94×10^{-3})	Above or on, 5 min/day (3.47×10^{-3}) Below, 10min/day (6.94×10^{-3})	Below, 5min/day (3.47×10^{-3})	Above, 1hr/day (4.17×10^{-2}) Below, 8hrs/day (3.33×10^{-1})
Probability of not Evacuating Area Affected	0.5	Above, 1.0 Below, 1.0	Above, 1.0 Below, 1.0	0.1	Above, 1.0 Below, 1.0
Spatial Probability	0.43 (7.5m length boulder)/17.5m length driveway)	Above, 0.11 (5m length fails/45m length of wall) Below, 0.11 (5m length fails/45m length of wall)	Above, 0.076 (5m length of slope failure/66m length of upper slope) Below, 0.11 (5m length of slope failure/45m length of site below site)	0.057 (1m length/17.5m length driveway)	Above, 0.035 (5m/142m) Below, 0.035 (5m/142m)
Vulnerability to Life if Failure Occurs Whilst Person Present	1.0	Above, 0.5 Below, 1.0	Above, 0.5 Below, 1.0	0.001	Above, 0.5 Below, 1.0
Risk for Person most at Risk	7.46×10^{-9}	Above, 1.25×10^{-12} Below, 7.63×10^{-10}	Above, 1.32×10^{-9} Below, 7.63×10^{-9}	1.98×10^{-10}	Above, 7.30×10^{-10} Below, 1.17×10^{-8}
Combined total Risk FOR Person most at Risk	3.0×10^{-8}				

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Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A

MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Residential Development
Location: 1102 Barrenjoey Road, Palm Beach, NSW

Ref No: 33618YJ
Report: A
Report Date: 10/11/2020
Page 1 of 1

AS 1289	TEST METHOD	2.1.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
101	3.15 - 3.25	11.0
102	0.20 - 0.40	6.8

Notes:
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 05/11/2020.
- Sampled and supplied by client. Samples tested as received.

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NATA Accredited Laboratory Number:1327

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10/11/2020
Authorised Signature / Date
(T. Finnegan)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Reform Projects Pty Ltd	**Ref No:**	33618YJ
Project:	Proposed Residential Development	**Report:**	B
Location:	1102 Barrenjoey Road, PALM BEACH, NSW	**Report Date:**	11/11/20
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
101	3.95 - 3.98	0.04	1	A
	4.15 - 4.19	0.1	2	A
	5.39 - 5.43	0.07	1	A
	5.80 - 5.84	0.08	2	A
	6.10 - 6.13	0.1	2	A
	6.74 - 6.77	0.01	<1	A
	7.21 - 7.24	0.07	1	A
	8.23 - 8.26	0.06	1	A
	8.87 - 8.89	0.1	2	A
	9.00 - 9.02	0.4	8	A
	9.37 - 9.40	0.4	8	A
102	0.77 - 0.80	0.07	1	A
	1.39 - 1.43	0.1	2	A
	1.91 - 1.95	0.09	2	A
	2.14 - 2.17	0.2	4	A
	2.75 - 2.78	0.3	6	A
	3.17 - 3.21	0.1	2	A
	3.28 - 3.32	0.1	2	A
	3.83 - 3.87	0.1	2	A
	4.13 - 4.16	0.09	2	A
	4.72 - 4.75	0.1	2	A
	5.30 - 5.33	0.7	1	A
	5.17 - 5.21	0.6	12	A
	5.76 - 5.80	0.3	6	A
	6.27 - 6.31	0.06	1	A
	6.69 - 6.72	0.08	2	A

NOTE: SEE PAGE 2

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Reform Projects Pty Ltd	**Ref No:**	33618YJ
Project:	Proposed Residential Development	**Report:**	B
Location:	1102 Barrenjoey Road, PALM BEACH, NSW	**Report Date:**	11/11/20

Page 2 of 2

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
102	7.30 - 7.32	0.1	2	A
	7.83 - 7.86	0.1	2	A
	8.06 - 8.08	0.2	4	A
	8.65 - 8.68	0.2	4	A

NOTES

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

Envirolab Services Pty Ltd
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www.envirolab.com.au

CERTIFICATE OF ANALYSIS 255075

Client Details

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

Sample Details

Your Reference	**33618YJ, Palm Beach**
Number of Samples	3 Soil
Date samples received	05/11/2020
Date completed instructions received	05/11/2020

Analysis Details

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

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

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

Report Details

Date results requested by	12/11/2020
Date of Issue	12/11/2020

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		255075-1	255075-2	255075-3
Your Reference	UNITS	BH101	BH101	BH101
Depth		1-1.2	2.6-2.8	3.0-3.15
Date Sampled		04/11/2020	04/11/2020	04/11/2020
Type of sample		Soil	Soil	Soil
Date prepared	-	06/11/2020	06/11/2020	06/11/2020
Date analysed	-	06/11/2020	06/11/2020	06/11/2020
pH 1:5 soil:water	pH Units	7.2	7.4	7.0
Chloride, Cl 1:5 soil:water	mg/kg	20	32	10
Sulphate, SO4 1:5 soil:water	mg/kg	33	29	<10
Resistivity in soil*	ohm m	150	200	310

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	255075-2
Date prepared	-			06/11/2020	1	06/11/2020	06/11/2020		06/11/2020	06/11/2020
Date analysed	-			06/11/2020	1	06/11/2020	06/11/2020		06/11/2020	06/11/2020
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	7.2	7.2	0	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	20	10	67	98	90
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	33	29	13	102	108
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	150	150	0	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

0178

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BOREHOLE LOG

Borehole No.
101
1 / 2

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33618YJ **Method:** SPIRAL AUGER **R.L. Surface:** ~2.7 m
Date: 4/11/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./J.M.

Groundwater Record:
(1.96m depth) GW ON 5/11/20
(2.4m depth) GW & COLLAPSE ON COMPLETION OF AUGERING
(2.16m depth) GW ON 18/11/20
(2.5m depth) GW ON COMPLETION OF CORING

Groundwater Record	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
	■									FILL: Gravel, fine to coarse grained igneous and sandstone gravel, brick and concrete fragments.	D M			APPEARS WELL COMPACTED
	■				N = 9 3,4,5	2				FILL: Silty sand, fine grained, brown, trace of fine to coarse grained sandstone gravel, and concrete fragments.				GRAVEL EMBEDDED IN SPT CATCHER, POTENTIALLY AFFECTED SPT
	■			■			1			FILL: Clayey sand, fine to medium grained, grey and brown, trace of quartz gravel.				POSSIBLY MARINE (NATURAL)
	■			■	N = 3 1,1,2	1	2		SM	Silty SAND: fine to medium grained, yellow brown.	M W	VL		MARINE
	■			■		0				Silty SAND: fine to coarse grained, red brown, trace of clay and fine to medium grained sub-rounded ironstone gravel and fine grained quartz gravel.				
				■	N > 15 3,15/ 100mm REFUSAL		3		-	Extremely Weathered laminite: silty CLAY, medium plasticity, dark brown, with iron indurated bands.	XW	Hd		NEWPORT FORMATION
											DW - XW	VL - Hd		VERY LOW 'TC' BIT RESISTANCE
						-1 -2 -3 -4	4 5 6			Interbedded SANDSTONE and SILTSTONE: fine grained, grey and dark grey, with extremely weathered bands. REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 9.4m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 9.4m TO 1.9m. CASING 1.9m TO 0m. 2mm SAND FILTER PACK 9.4m TO 1.4m. BENTONITE SEAL 1.4m TO 0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

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CORED BOREHOLE LOG

Borehole No.
101
2 / 2

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33618YJ
Core Size: NMLC
R.L. Surface: ~2.7 m
Date: 4/11/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: B.S./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
				START CORING AT 3.42m					
100% RETURN		-1	4	Extremely Weathered interbedded sandstone and siltstone: silty CLAY, medium plasticity, grey, dark grey and orange brown, sandstone, with iron indurated bands and very low strength bands and sandy clay seams of low plasticity.	XW	Hd		(3.60m) HP: >600 kPa (3.80m) HP: >600 kPa	Newport Formation
				LAMINITE: fine grained, grey and dark grey.	HW	VL	0.040 0.10		
		-2	5	Extremely Weathered laminite: silty CLAY, medium plasticity, dark grey, with iron indurated bands.	XW	Hd		(4.55m) HP: >600 kPa (4.95m) HP: 510 kPa (5.20m) HP: 500 kPa	
		-3	6	SILTSTONE: dark grey, distinctly bedded at 0°.	HW	VL	0.070 0.080 0.10 0.010	(5.44m) Be, 0°, Un, R, Cn (5.57m) Jh, 70° (5.91m) Jh, 70° (6.27m) J, 60°, P, R, Clay FILLED, 2 mm.t (6.55m) XWS, 0°, 140 mm.t	
		-4	7	LAMINITE: fine grained, grey and dark grey, distinctly bedded at 0°.			0.070	(6.83m) XWS, 0°, 10 mm.t (6.95m) XWS, 0°, 30 mm.t (7.26m) Be, 0°, Un, R, Cn (7.51m) XWS, 0°, 20 mm.t	
		-5		Extremely Weathered laminite: silty CLAY, medium plasticity, dark grey and grey, with very low strength bands.	XW	Hd		(7.64m) J, 90°, Un, R, Clay Ct	
			8	NO CORE 0.35m					
		-6	9	LAMINITE: fine grained, grey and dark grey, with iron indurated and sandstone bands, distinctly bedded at 0-10°.	HW	VL - L	0.060 0.10	(8.21m) Be, 0°, Un, R, Clay Ct (8.50m) XWS, 0°, 10 mm.t (8.73m) XWS, 0°, 15 mm.t (8.83m) Be, 5°, Un, R, Fe Sn (8.90m) Be, 0°, Un, R, Cn	Newport Formation
					MW	M	0.40	(9.05m) Be, 0°, P, R, Cn	
		-7		END OF BOREHOLE AT 9.40 m					

SPACING (mm): 600 200 60 20

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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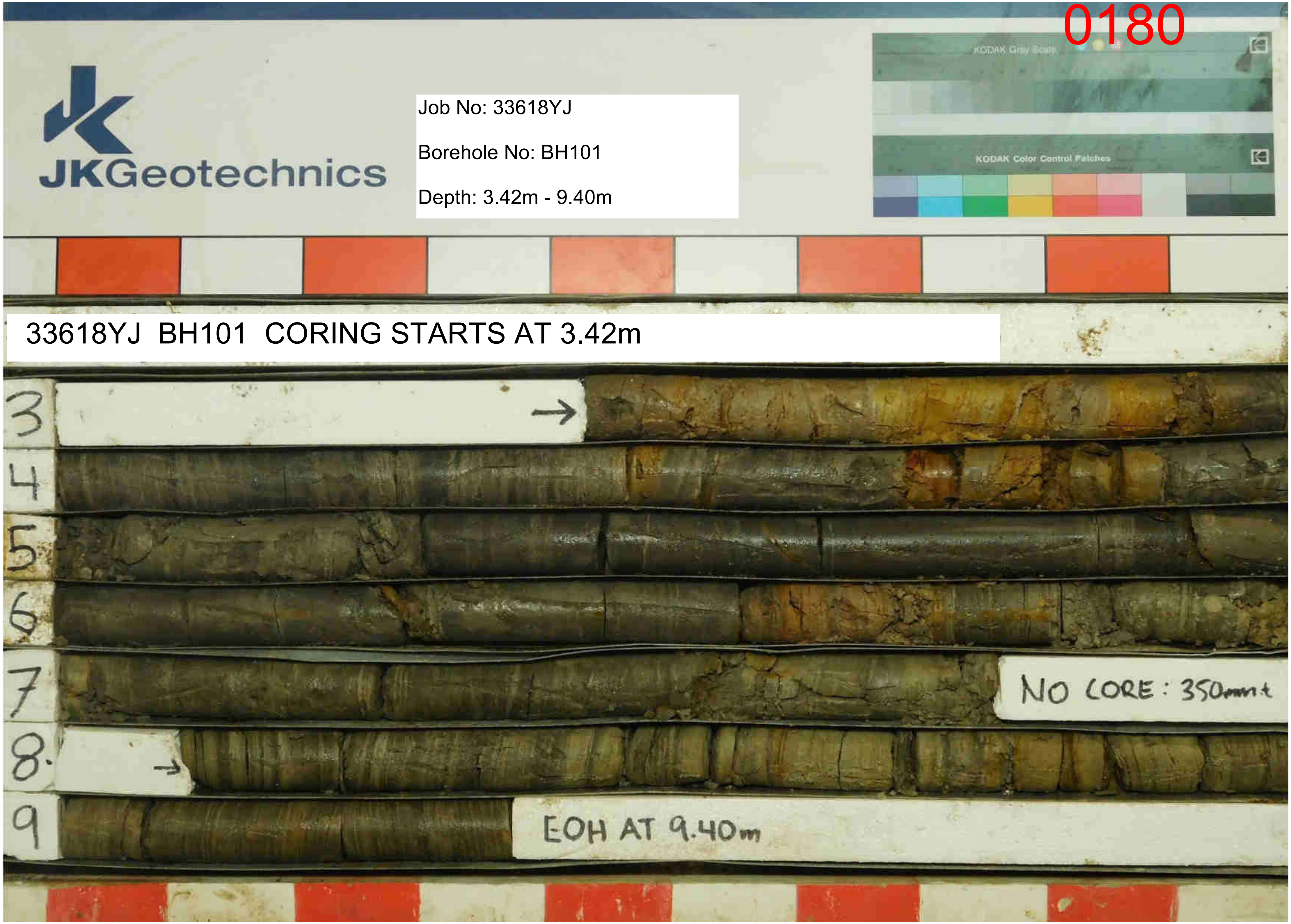
Job No: 33618YJ

Borehole No: BH101

Depth: 3.42m - 9.40m

33618YJ BH101 CORING STARTS AT 3.42m



JKGeotechnics

BOREHOLE LOG

Borehole No.
102
1 / 3

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33618YJ **Method:** SPIRAL AUGER **R.L. Surface:** ~2.7 m
Date: 4/11/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./J.M.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■									FILL: Gravel, fine to coarse grained, igneous and sandstone gravel, bricks and concrete fragments.	D			
				■					-	Interbedded SANDSTONE and SILTSTONE: fine to medium grained, dark grey, grey and brown, with iron indurated bands.	DW	VL - L		NEWPORT FORMATION VERY LOW 'TC' BIT RESISTANCE
										REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.98m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 8.98m TO 1.5m. CASING 1.5m TO 0m. 2mm SAND FILTER PACK 8.98m TO 0.6m. BENTONITE SEAL 0.6m TO 0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						2								
							1							
						1								
							2							
						0								
							3							
						-1								
							4							
						-2								
							5							
						-3								
							6							
						-4								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33618YJ PALMBEACH.GPJ <<DrawingFile>> 24/11/2020 10:06 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

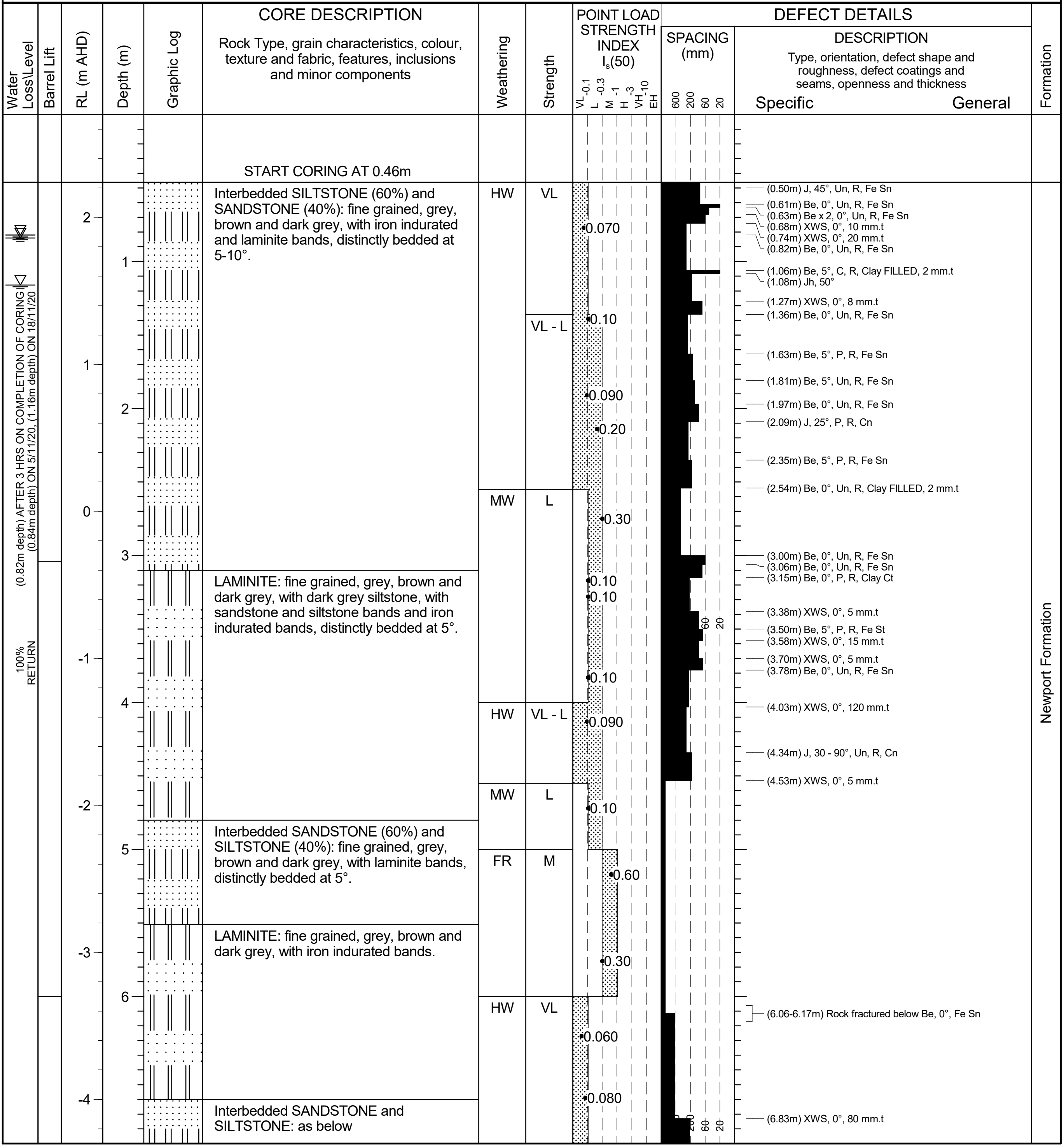
JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
102
2 / 3

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33618YJ	**Core Size:** NMLC	**R.L. Surface:** ~2.7 m
Date: 4/11/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK205	**Bearing:** N/A	**Logged/Checked By:** B.S./J.M.



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

START CORING AT 0.46m

Interbedded SILTSTONE (60%) and SANDSTONE (40%): fine grained, grey, brown and dark grey, with iron indurated and laminite bands, distinctly bedded at 5-10°.

LAMINITE: fine grained, grey, brown and dark grey, with dark grey siltstone, with sandstone and siltstone bands and iron indurated bands, distinctly bedded at 5°.

Interbedded SANDSTONE (60%) and SILTSTONE (40%): fine grained, grey, brown and dark grey, with laminite bands, distinctly bedded at 5°.

LAMINITE: fine grained, grey, brown and dark grey, with iron indurated bands.

Interbedded SANDSTONE and SILTSTONE: as below

Water Loss\Level: (0.82m depth) AFTER 3 HRS ON COMPLETION OF CORING (0.84m depth) ON 5/11/20, (1.16m depth) ON 18/11/20

100% RETURN

Weathering	Strength
HW	VL
	VL - L
MW	L
HW	VL - L
MW	L
FR	M
HW	VL

Point load strength index: 0.070, 0.10, 0.090, 0.20, 0.30, 0.10, 0.10, 0.10, 0.090, 0.10, 0.60, 0.30, 0.060, 0.080

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

(0.50m) J, 45°, Un, R, Fe Sn
(0.61m) Be, 0°, Un, R, Fe Sn
(0.63m) Be x 2, 0°, Un, R, Fe Sn
(0.68m) XWS, 0°, 10 mm.t
(0.74m) XWS, 0°, 20 mm.t
(0.82m) Be, 0°, Un, R, Fe Sn
(1.06m) Be, 5°, C, R, Clay FILLED, 2 mm.t
(1.08m) Jh, 50°
(1.27m) XWS, 0°, 8 mm.t
(1.36m) Be, 0°, Un, R, Fe Sn
(1.63m) Be, 5°, P, R, Fe Sn
(1.81m) Be, 5°, Un, R, Fe Sn
(1.97m) Be, 0°, Un, R, Fe Sn
(2.09m) J, 25°, P, R, Cn
(2.35m) Be, 5°, P, R, Fe Sn
(2.54m) Be, 0°, Un, R, Clay FILLED, 2 mm.t
(3.00m) Be, 0°, Un, R, Fe Sn
(3.06m) Be, 0°, Un, R, Fe Sn
(3.15m) Be, 0°, P, R, Clay Ct
(3.38m) XWS, 0°, 5 mm.t
(3.50m) Be, 5°, P, R, Fe St
(3.58m) XWS, 0°, 15 mm.t
(3.70m) XWS, 0°, 5 mm.t
(3.78m) Be, 0°, Un, R, Fe Sn
(4.03m) XWS, 0°, 120 mm.t
(4.34m) J, 30 - 90°, Un, R, Cn
(4.53m) XWS, 0°, 5 mm.t
(6.06-6.17m) Rock fractured below Be, 0°, Fe Sn
(6.83m) XWS, 0°, 80 mm.t

Newport Formation

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

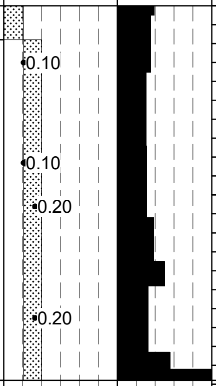
CORED BOREHOLE LOG

Borehole No.
102
3 / 3

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 1102-1106 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33618YJ
Core Size: NMLC
R.L. Surface: ~2.7 m
Date: 4/11/20
Inclination: VERTICAL
Datum: AHD
Plant Type: JK205
Bearing: N/A
Logged/Checked By: B.S./J.M.

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 DETAILS – SPACING (mm)	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
100% RETURN		-5 -6	8		Interbedded SANDSTONE and SILTSTONE: fine grained, grey and brown sandstone, inter bedded with dark grey siltstone, with laminite and iron indurated bands and highly weathered bands.	HW MW	VL L	0.10 0.10 0.20 0.20		(7.05m) XWS, 0°, 130 mm.t (7.20-7.50m) J, 50 - 90°, Ir, S, Cn (7.74m) J x 2, 40 - 80°, Ir, S, Cn (8.12m) J, 30°, C, R, Clay Ct (8.35m) XWS, 0°, 70 mm.t (8.48m) J, 80°, Un, R, Cn (8.83m) J, 90°, Ir, R, Cn (8.92m) XWS, 0°, 10 mm.t	
		-7 -8 -9 -10 -11	9 10 11 12 13		END OF BOREHOLE AT 8.98 m						



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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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Job No: 33618YJ

Borehole No: BH102

Depth: 0.46m - 8.98m

33618YJ BH102 CORING STARTS AT 0.46m

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

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title: **SITE LOCATION PLAN**

Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

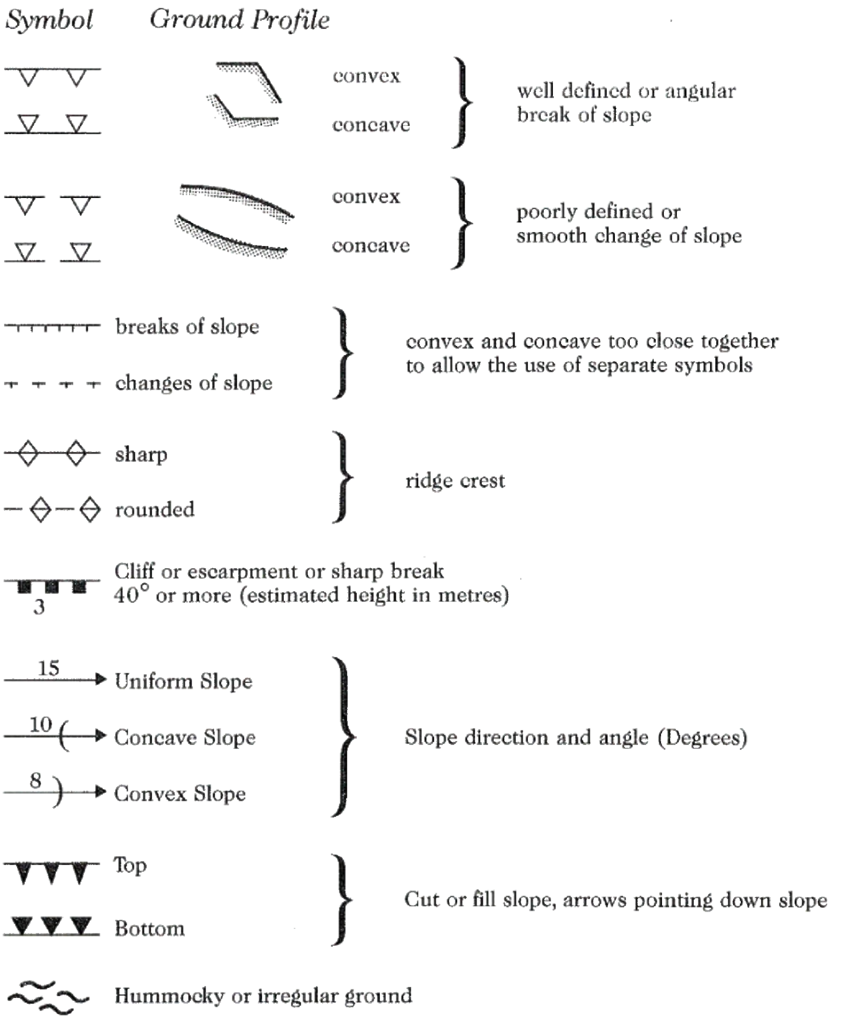
Report No: 33618YJrptrev4

Figure No: 1

JKGeotechnics

PLOT DATE: 29/06/2023 1:51:43 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG

TOPOGRAPHY



Symbol *Ground Profile*

convex
concave
well defined or angular break of slope

convex
concave
poorly defined or smooth change of slope

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

sharp
rounded
ridge crest

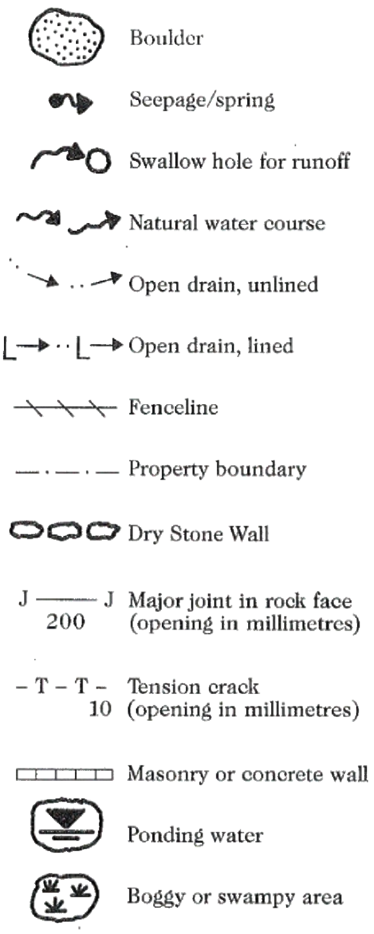
Cliff or escarpment or sharp break 40° or more (estimated height in metres)

Uniform Slope
Concave Slope
Convex Slope
Slope direction and angle (Degrees)

Top
Bottom
Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

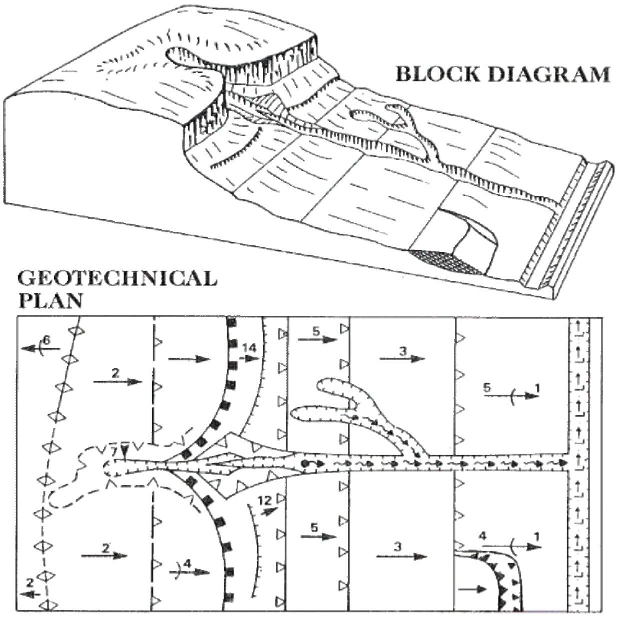
OTHER FEATURES



Boulder
Seepage/spring
Swallow hole for runoff
Natural water course
Open drain, unlined
Open drain, lined
Fenceline
Property boundary
Dry Stone Wall
Major joint in rock face (opening in millimetres)
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).



Title:
GEOTECHNICAL MAPPING SYMBOLS

Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

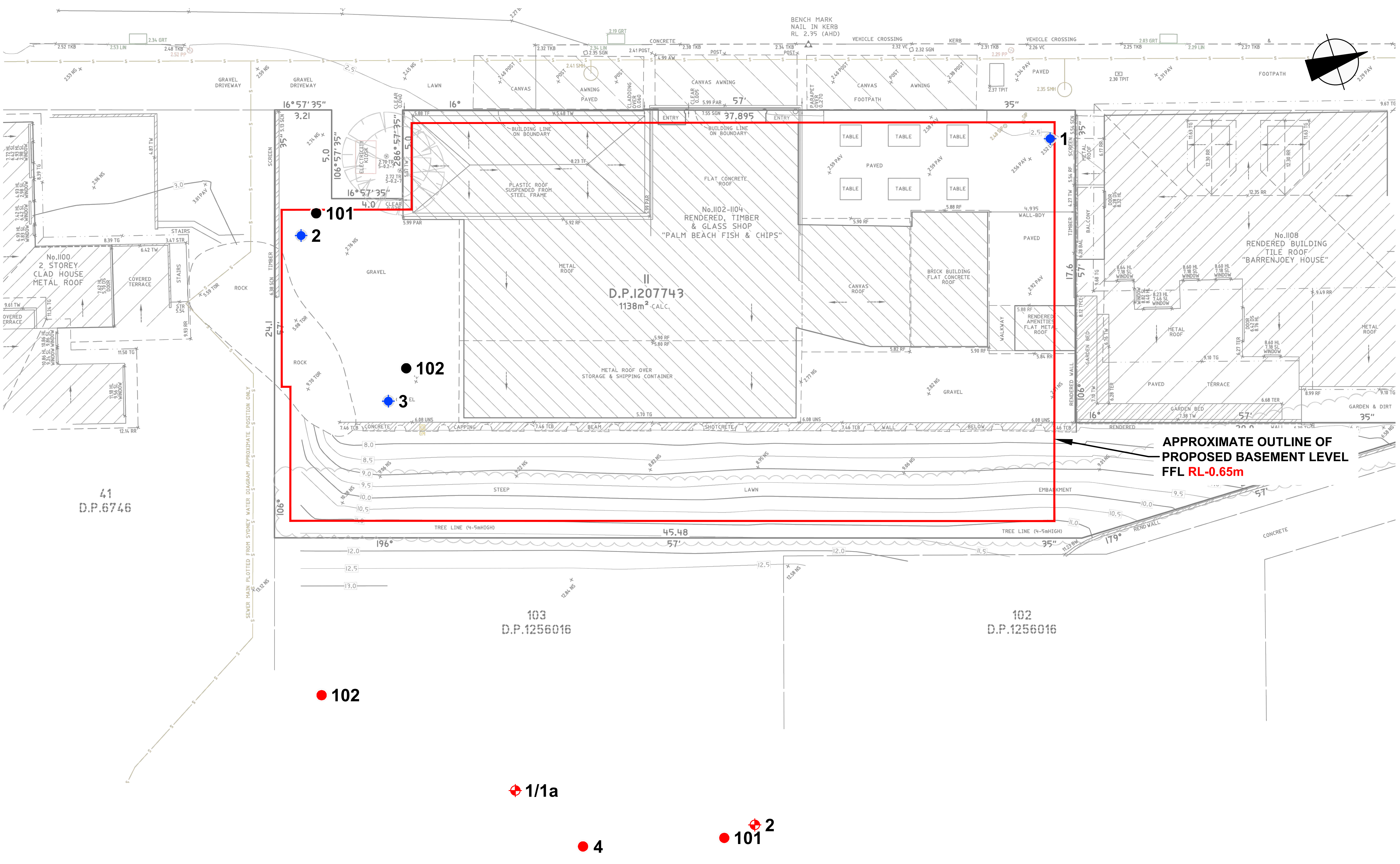
Report No: 33618YJ

Figure No: 6

JKGeotechnics

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

PLOT DATE: 24/11/2020 11:53:11 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJ.DWG



LEGEND

- ● BOREHOLE AND WELL (JK GEOTECHNICS, 2020)
- ● BOREHOLE AND WELL (WITT CONSULTING, 2019)
- ● BOREHOLE BY CROZIER GEOTECHNICAL CONSULTANTS
- ⊕ DCP TEST CROZIER GEOTECHNICAL CONSULTANTS

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

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

Title:
BOREHOLE LOCATION PLAN

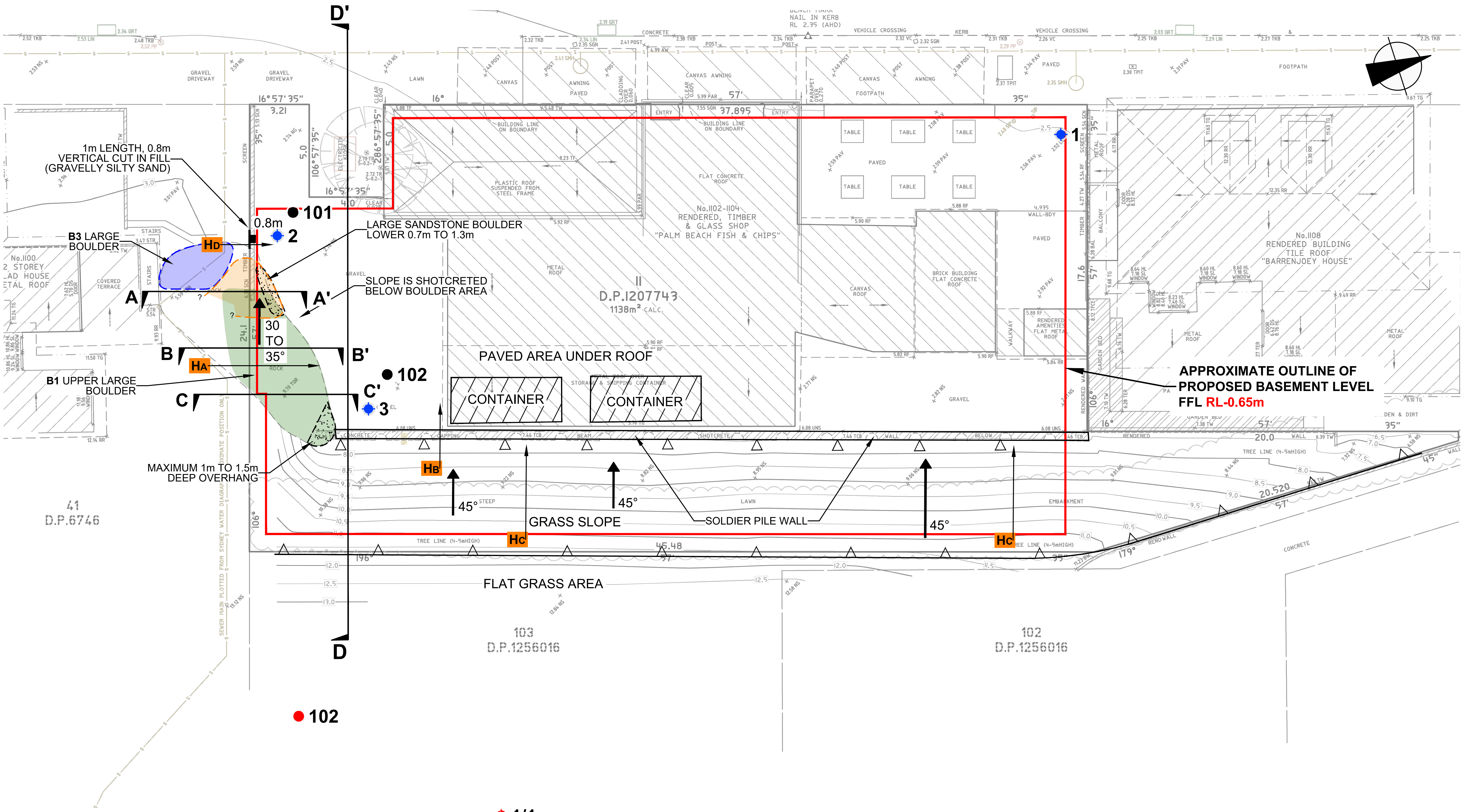
Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJrptrev4

Figure No: 2

JKGeotechnics

PLOT DATE: 25/03/2024 1:42:06 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG



LEGEND

●	BOREHOLE AND WELL (JK GEOTECHNICS, 2020)
●	BOREHOLE AND WELL (WITT CONSULTING, 2019)
●	BOREHOLE BY CROZIER GEOTECHNICAL CONSULTANTS
⊕	DCP TEST CROZIER GEOTECHNICAL CONSULTANTS
HD →	GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS
A⌊ ⌋A'	CROSS SECTION

NOTES:
1. REFER TO FIGURE 4 FOR CROSS SECTIONS A-A', B-B' AND C-C'.
2. REFER TO FIGURE 5 FOR CROSS SECTION D-D'.

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

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

Title: **PLAN OF NOTABLE GEOTECHNICAL FEATURES + GEOTECHNICAL HAZARDS**

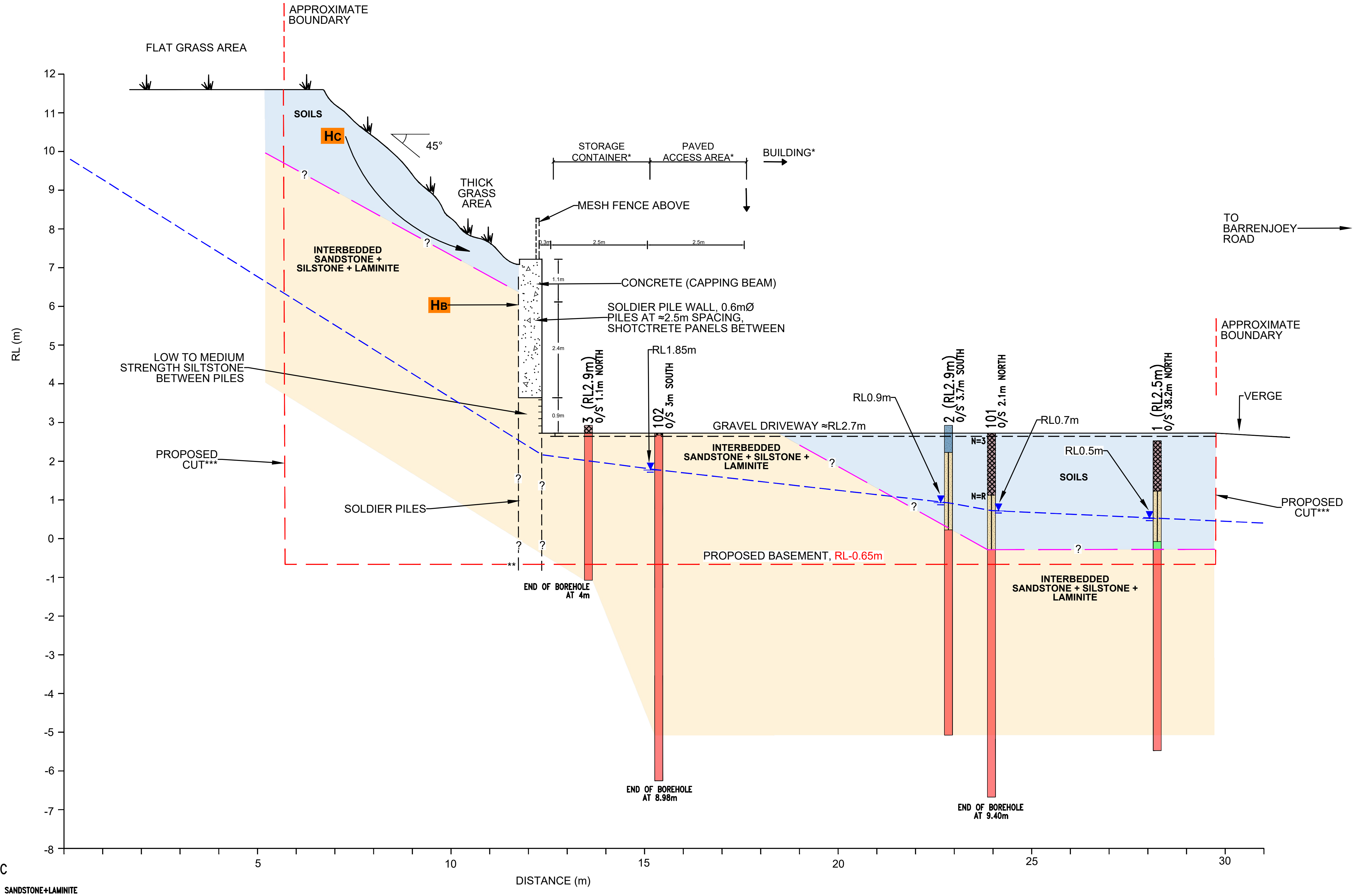
Location: 1102 - 1106 BARRENJOEY ROAD, PALM BEACH, NSW

Report No: 33618YJrptrev4

Figure No: 3

JKGeotechnics

PLOT DATE: 25/03/2024 1:42:41 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG



MATERIAL GRAPHIC

🟥 INTERBEDDED SILTSTONE, SANDSTONE+LAMINITE

▥ SILTY SAND ▦ SAND

▨ FILL 🟩 CLAY

LEGEND

— ? — ? INFERRED GEOTECHNICAL UNIT

— — ▼ — — INFERRED GROUNDWATER

Hc → GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS

NOTES:
- * 6m LONG STORAGE , ACCESS AREA + BUILDING SET IN 11m FROM SOUTHERN BOUNDARY + 6m FROM BOULDER.
- ** SOLDIER PILE WALL TOW SHOWN AT RL-0.25m IN ARCHITECTURAL CROSS SECTION.
- *** EXTENTS SHOWN ARE FOR PROPOSED WIDEST PORTION OF EASTERN + WESTERN BOUNDARY. HIGHEST GROUNDWATER SHOWN

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

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

Title: **SECTION D-D'**

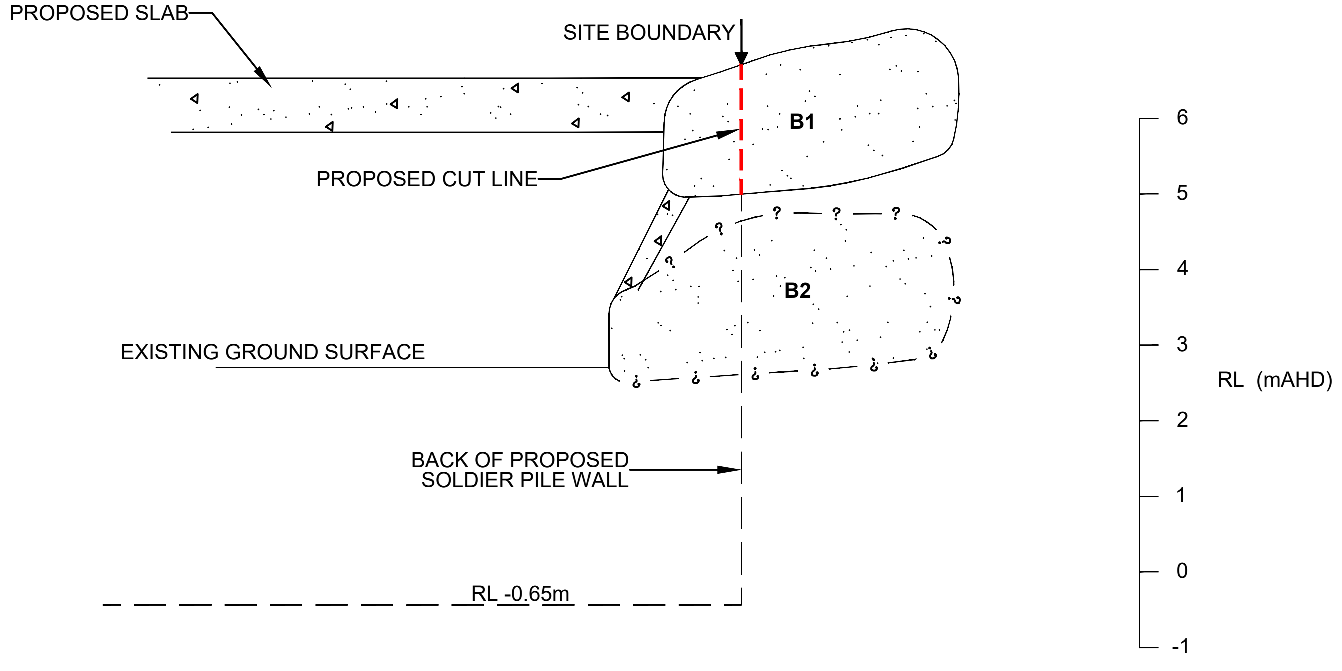
Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJrptrev4

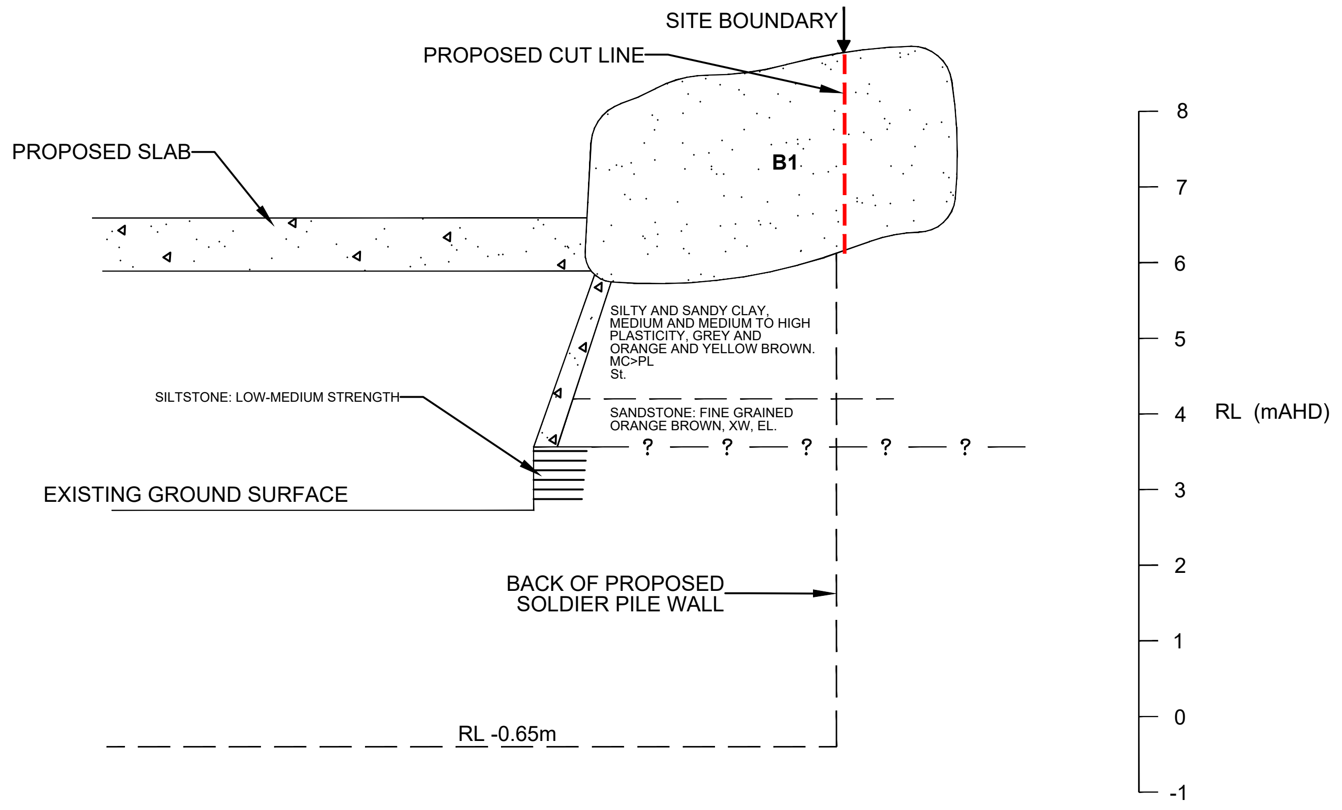
Figure No: 5

JKGeotechnics

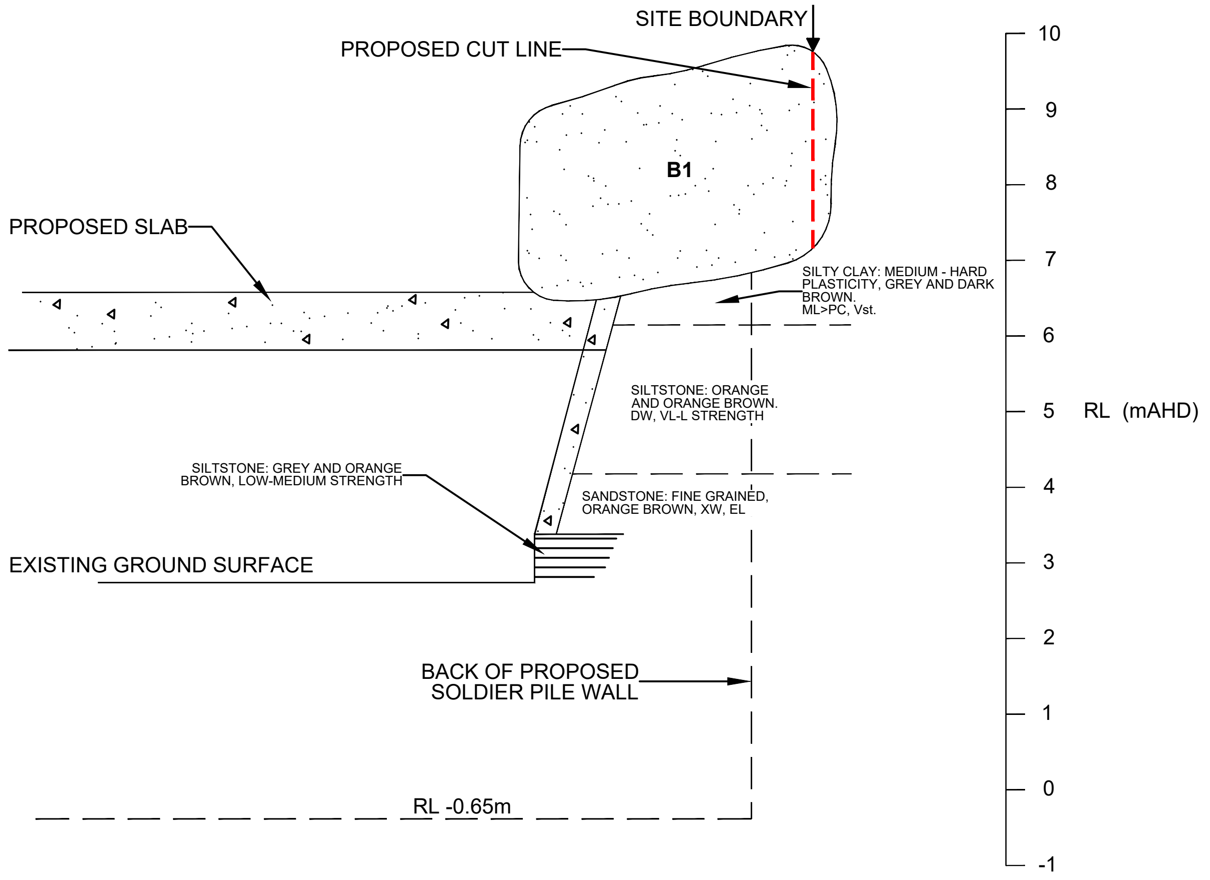
PLOT DATE: 25/03/2024 1:43:40 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG



SECTION A-A



SECTION B-B



SECTION C-C

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

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

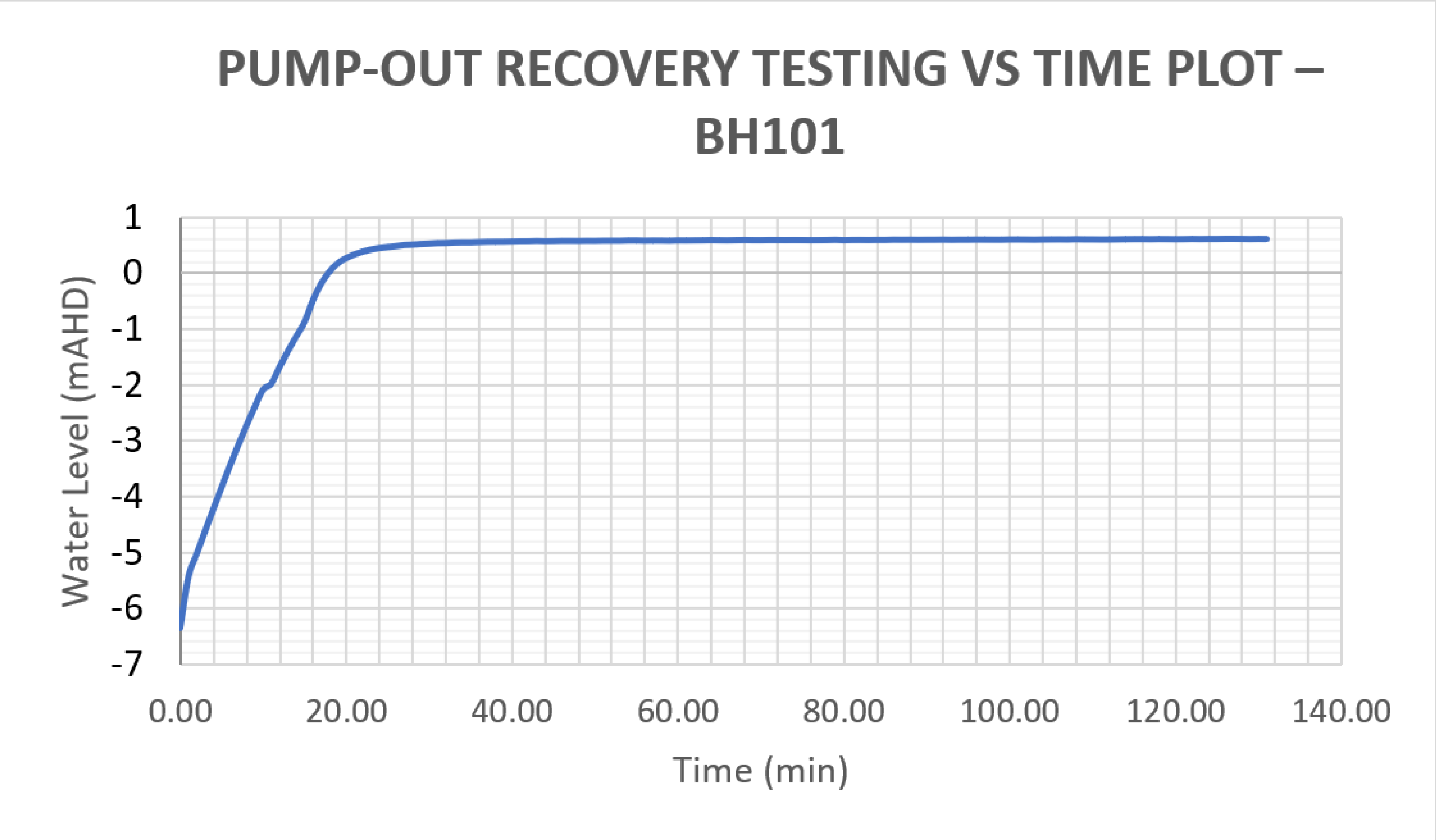
Title:
CROSS SECTIONS A-A', B-B' AND C-C'

Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJrptrev4
Figure No: 4

JKGeotechnics

PLOT DATE: 25/03/2024 1:43:07 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJRPTREV4.DWG

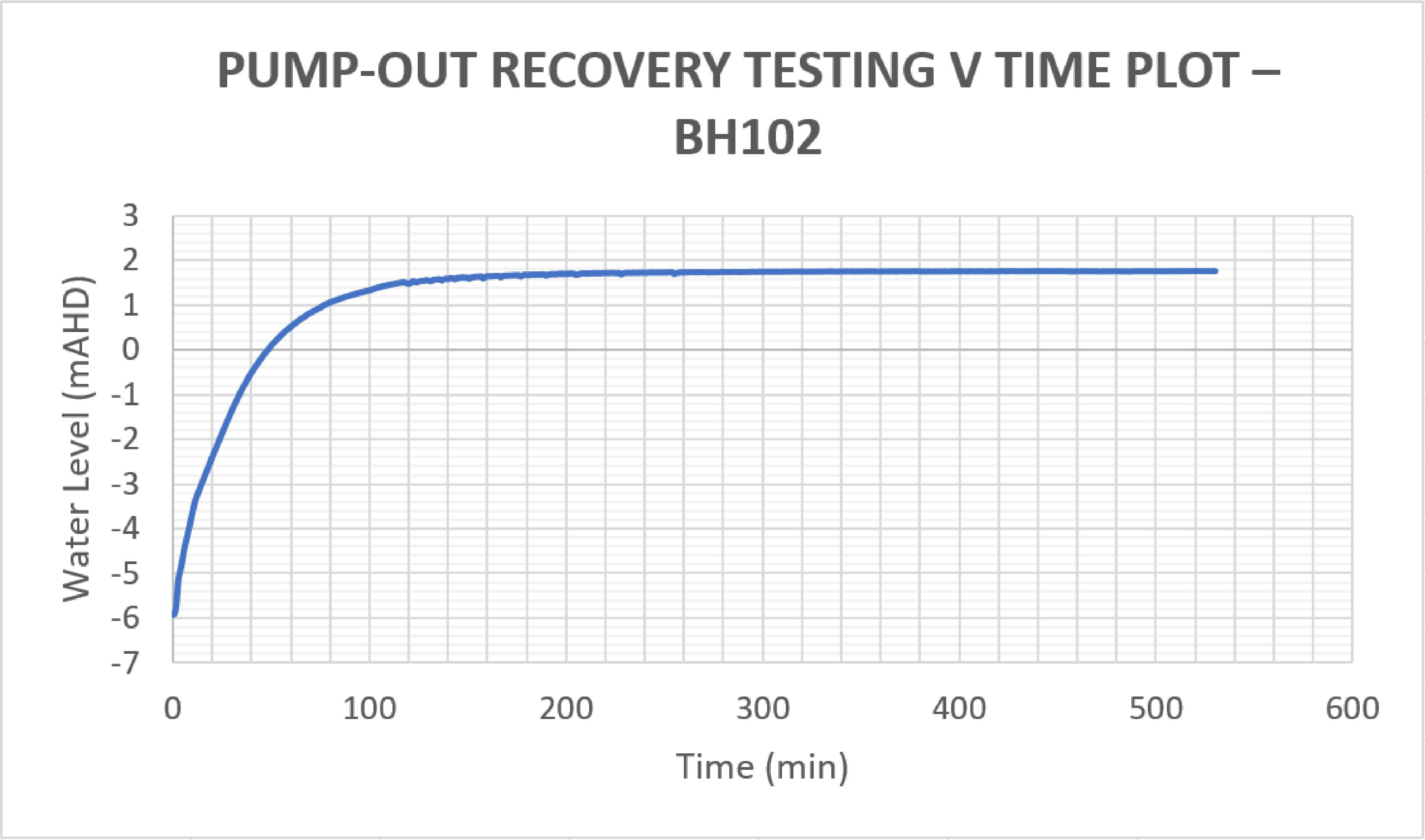


PUMP-OUT RECOVERY TESTING VS TIME PLOT – BH101

JKGeotechnics

Report No. 33618YJ

Figure No. 7

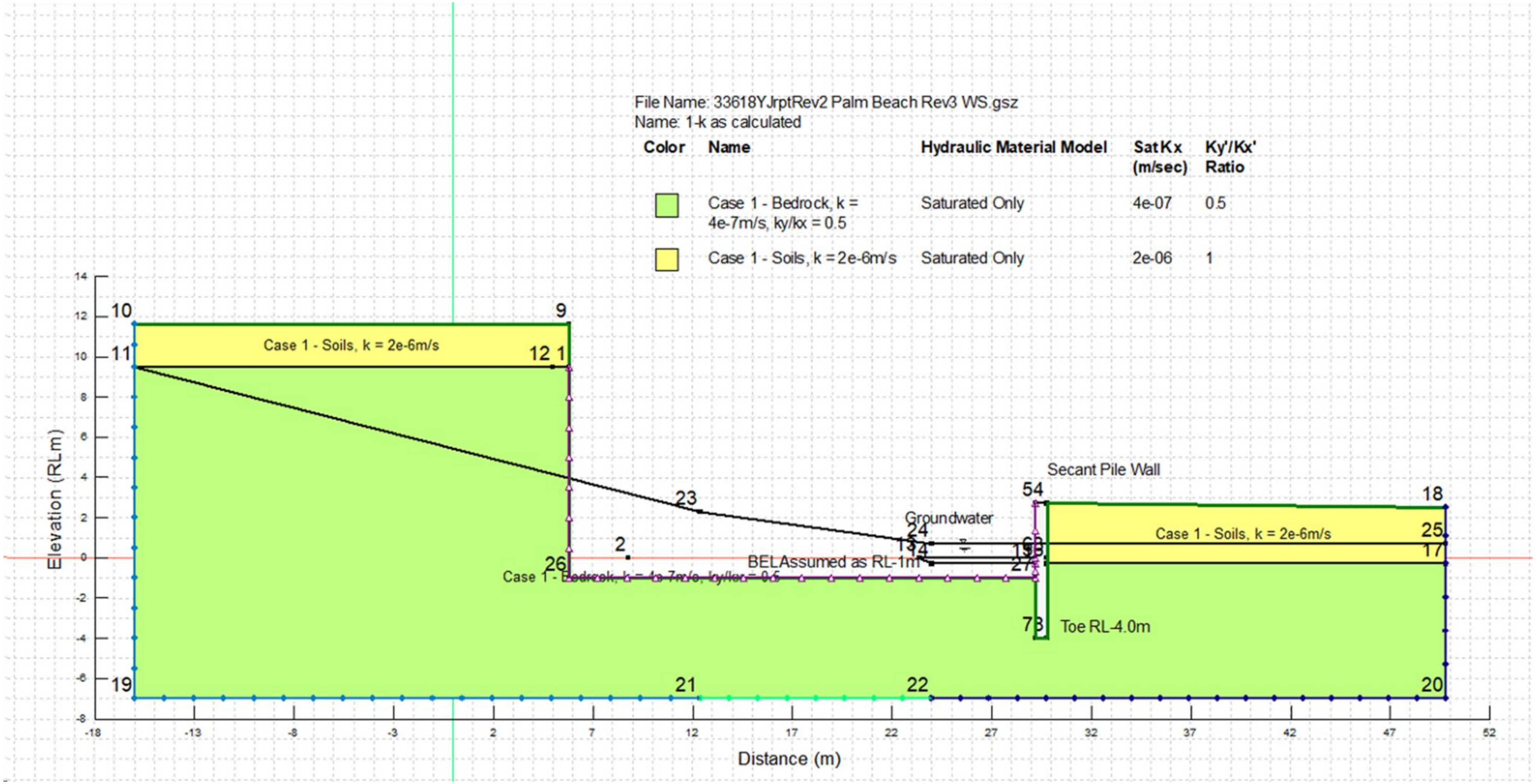


PUMP-OUT RECOVERY TESTING VS TIME PLOT – BH102

JKGeotechnics

Report No. 33618YJ

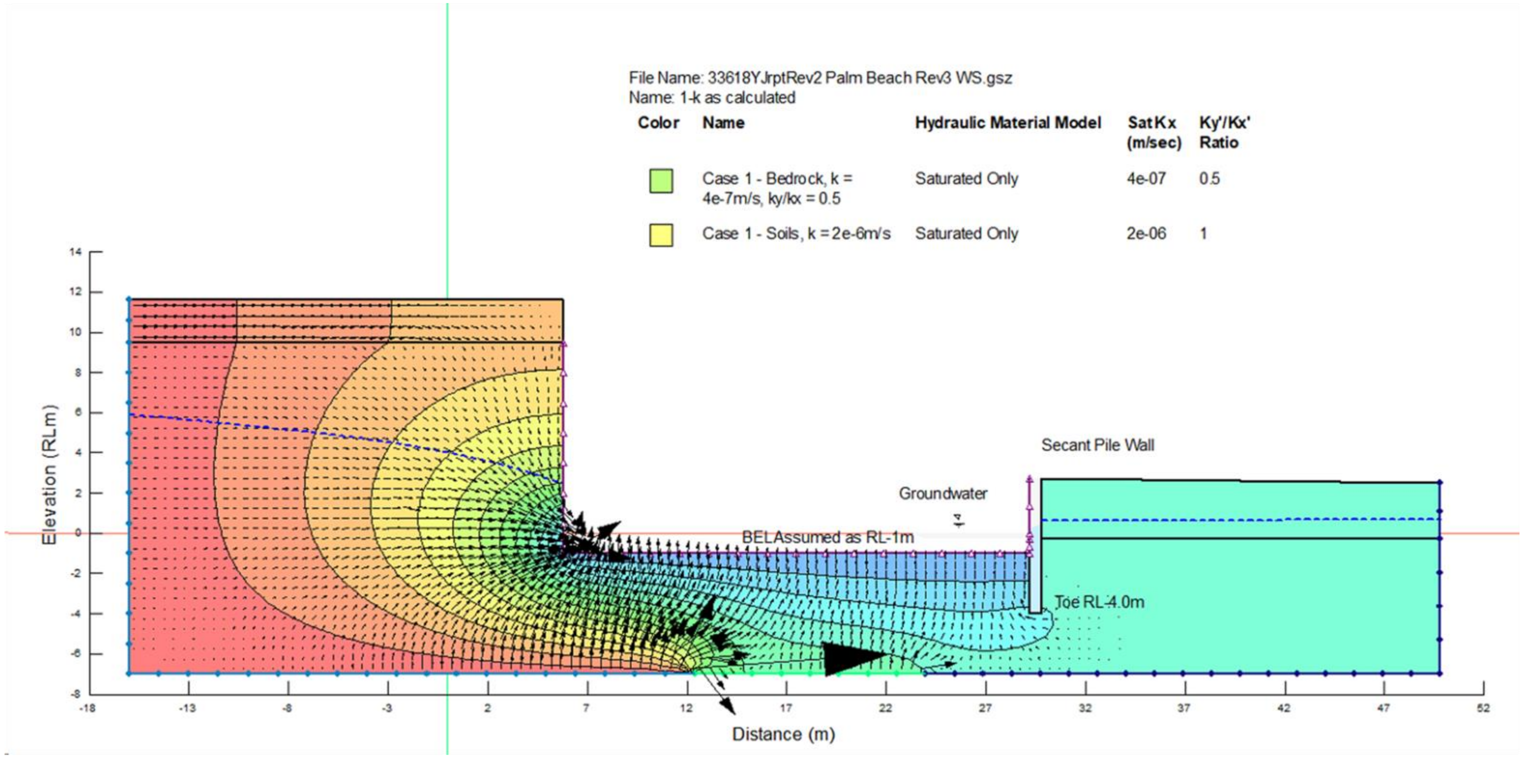
Figure No. 8



SEEP/W – GEOTECHNICAL MODEL

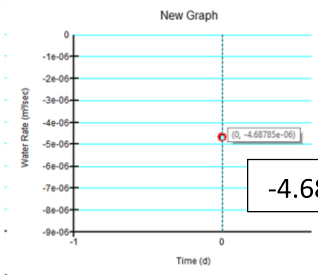
JKGeotechnics

Report No. 33618YJrev4C Figure No. 9



File Name: 33618YJrptRev2 Palm Beach Rev3 WS.gsz
Name: 1-k as calculated

Color	Name	Hydraulic Material Model	Sat Kx (m/sec)	Ky'/Kx' Ratio
🟩	Case 1 - Bedrock, k = 4e-7m/s, ky/kx = 0.5	Saturated Only	4e-07	0.5
🟨	Case 1 - Soils, k = 2e-6m/s	Saturated Only	2e-06	1



-4.68E-6

SEEP/W – OUTPUT EXTRACTION VOLUMES

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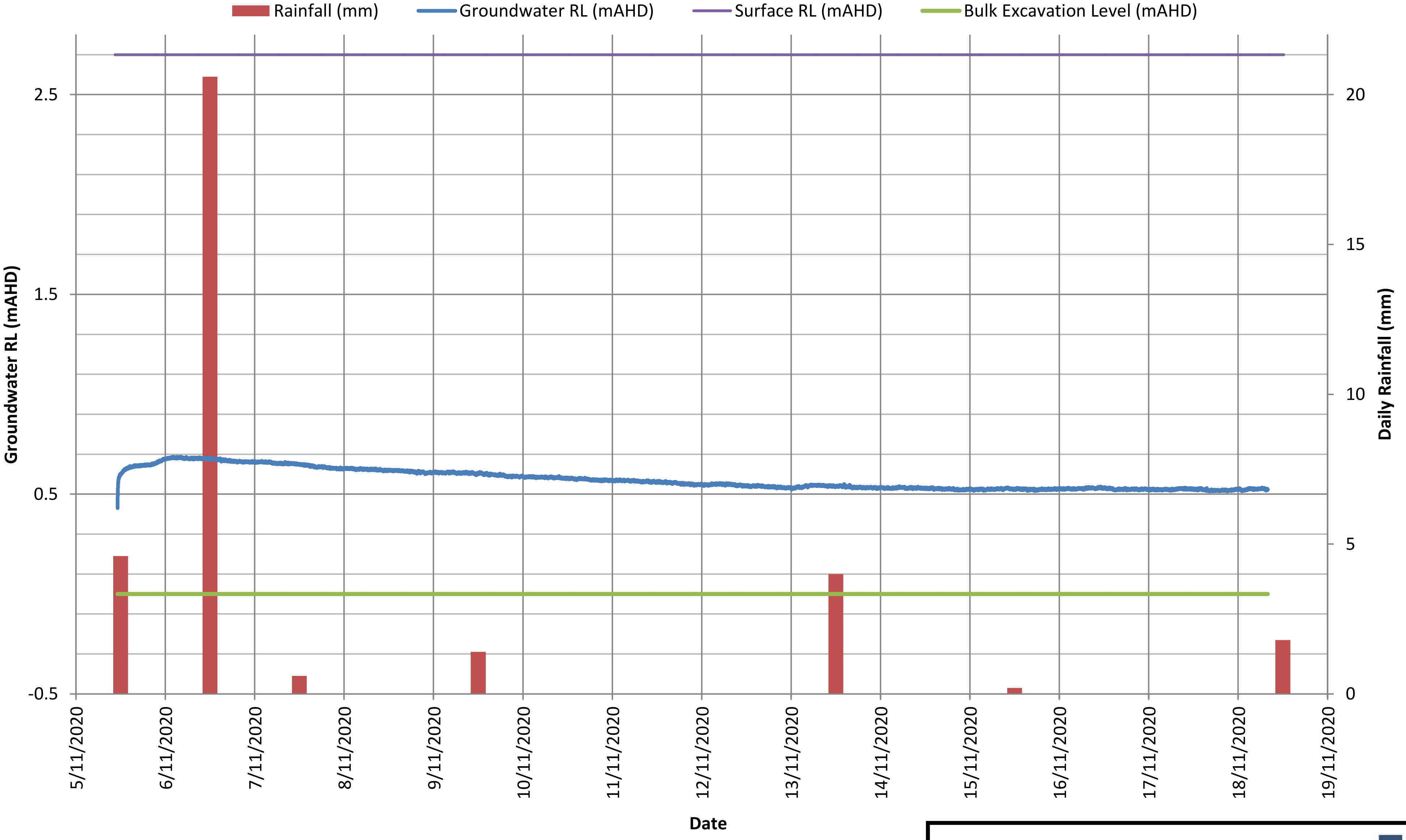
Report No. 33618YJrev4C Figure No. 10

File Name: 33618YJ Palm Beach BH101
Date Printed: 27/11/2020

Groundwater Level and Daily Rainfall -vs- Time Plot
BH101



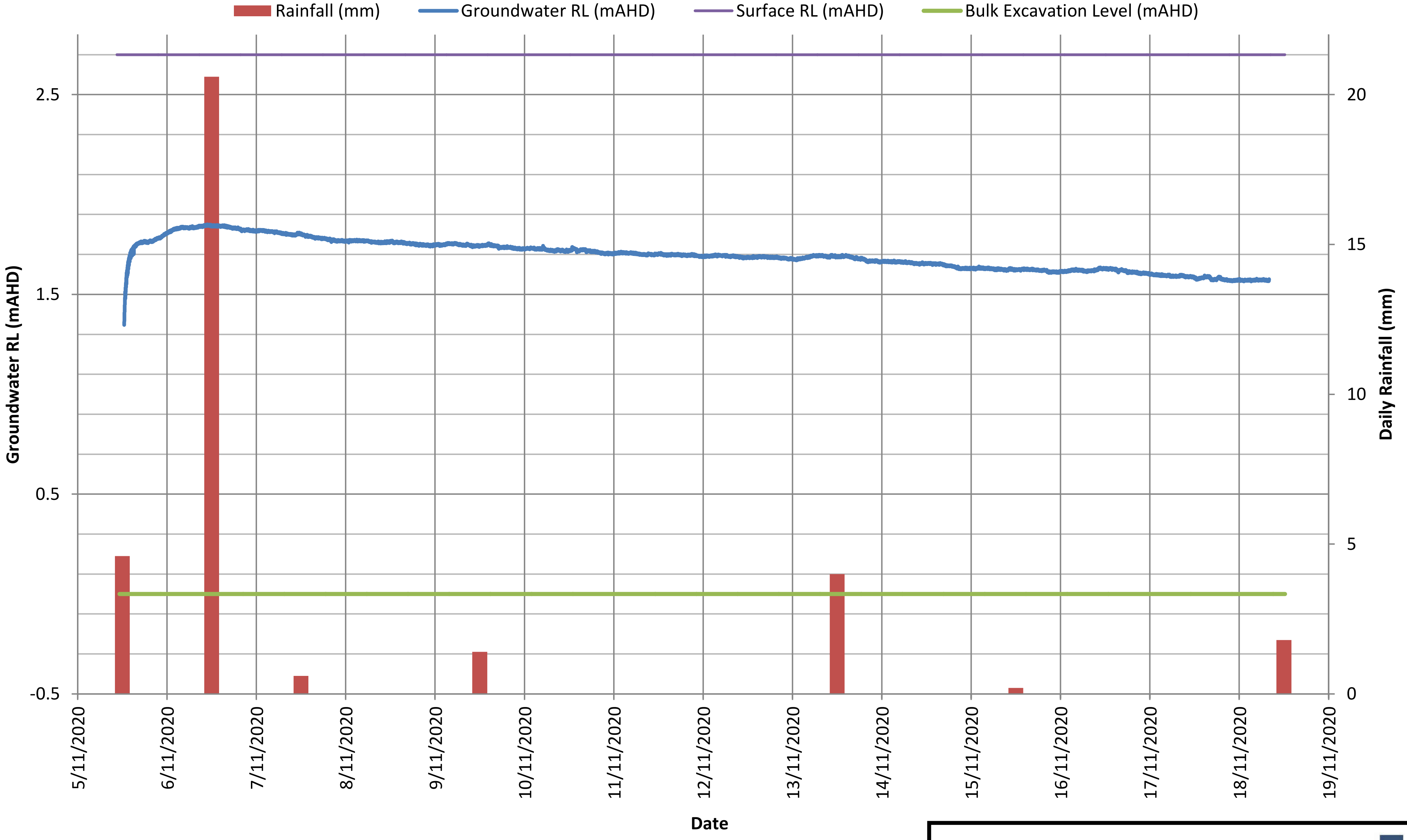
Rainfall data from Mona Vale Golf Club, Station number : 066141

JKGeotechnics

Report No. 33618YJ **Figure No. 11**

File Name: 33618YJ Palm Beach BH102
Date Printed: 27/11/2020

Groundwater Level and Daily Rainfall -vs- Time Plot
BH102



Rainfall data from Mona Vale Golf Club, Station number : 066141

JKGeotechnics

Report No. 33618YJ **Figure No. 12**

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

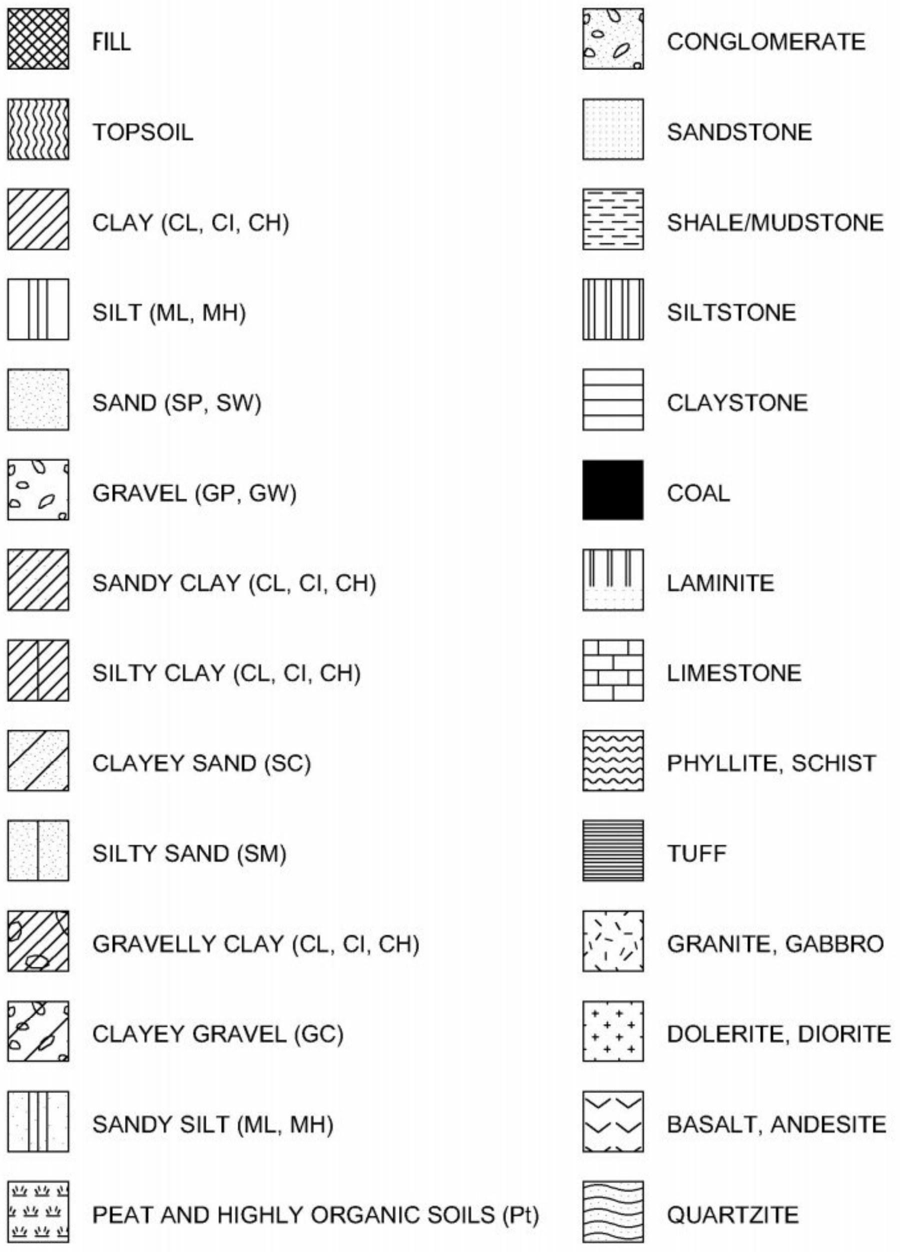
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS



SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory **Classification** Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

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

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:

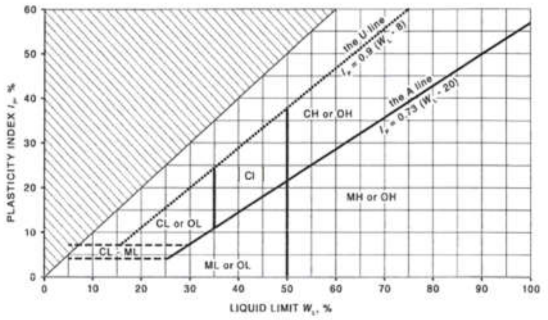
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $I_{s(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>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.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>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.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>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.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

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

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.

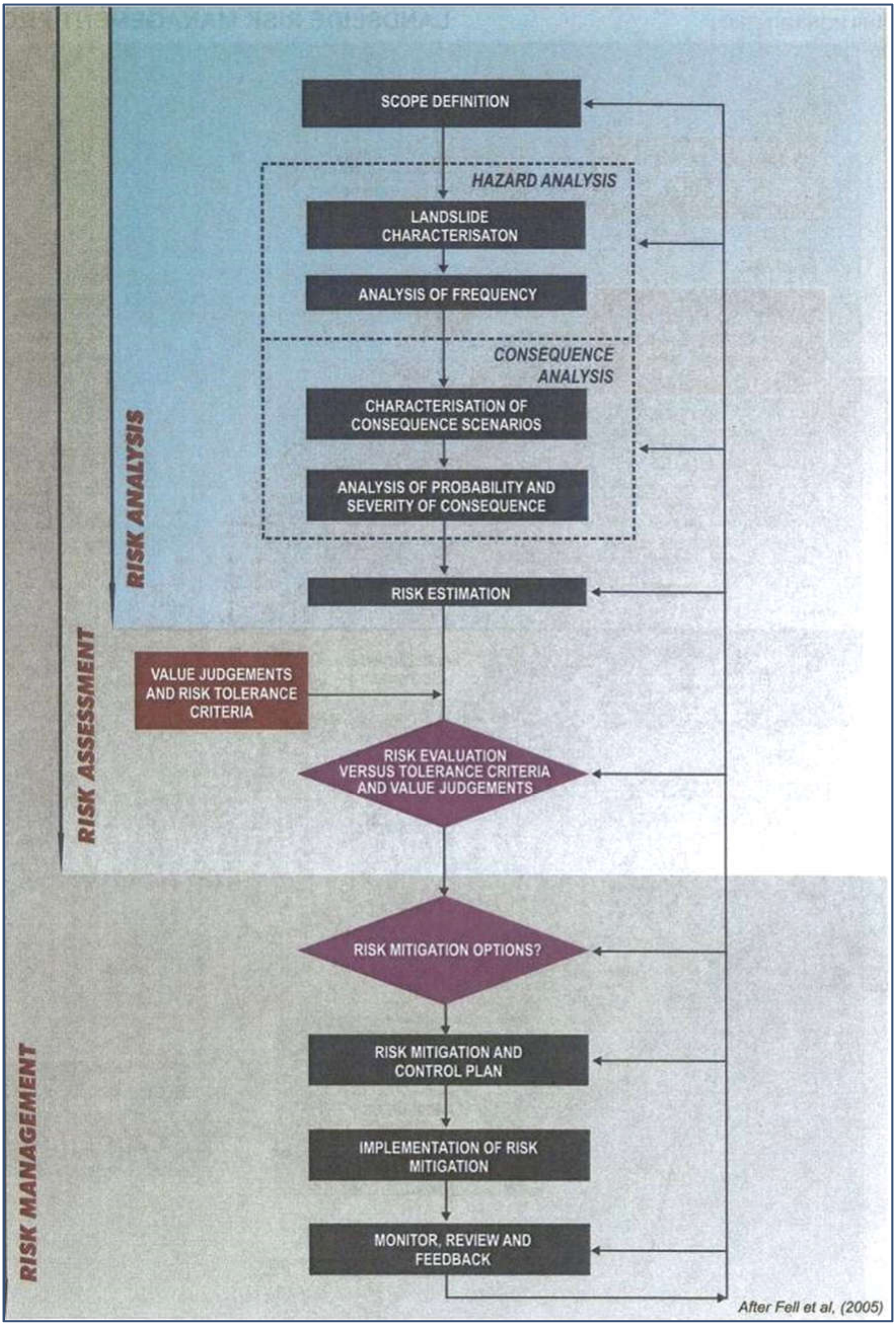


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

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

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

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

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

What Causes a Landslide?

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

Does a Landslide Affect You?

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

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

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

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

TABLE 1 – Slope Descriptions

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

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

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

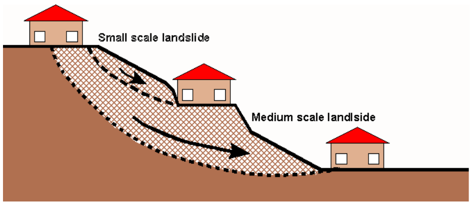


Figure 1

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

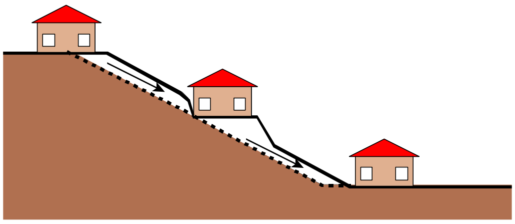


Figure 2

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

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

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

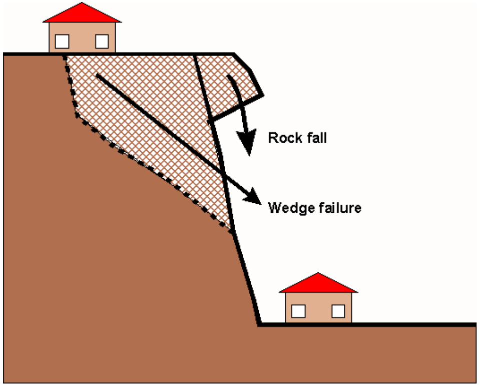


Figure 3

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

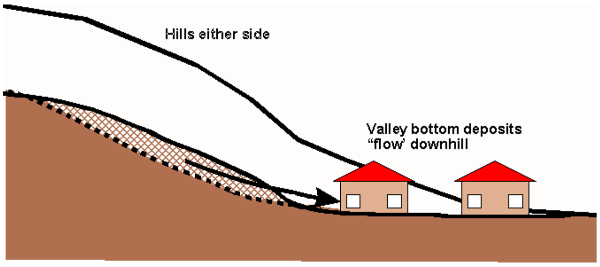


Figure 4

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

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

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AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

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

Landslide Risk Assessment

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

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

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

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

Risk to Property

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

TABLE 2 – LIKELIHOOD

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

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

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

TABLE 1 – RISK TO PROPERTY

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

Risk to Life

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

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

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

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

TABLE 3 – RISK TO LIFE

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

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

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APPENDIX B

SOME GUIDELINES

FOR

HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

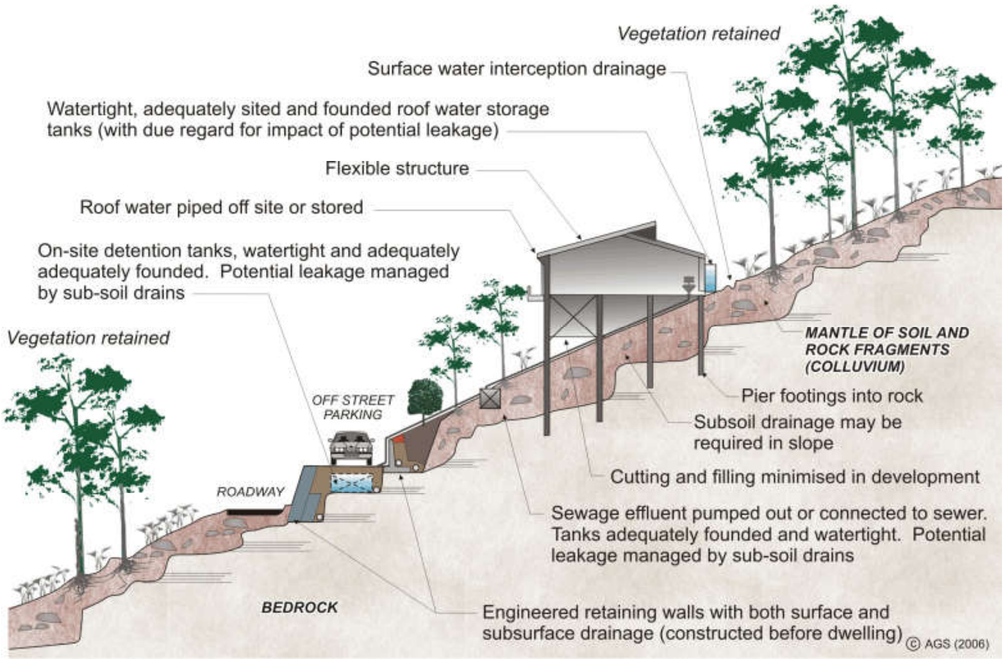
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminant bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements.
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance (including onto properties below). Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc. in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on bedrock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within bedrock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide generous falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge of roof run-off into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use of absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by a geotechnical consultant.	
SITE VISITS	Site visits by consultant may be appropriate during construction.	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is extracted 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 LR8 (CONSTRUCTION PRACTICE)

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES FOR GOOD HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that due to level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfill the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

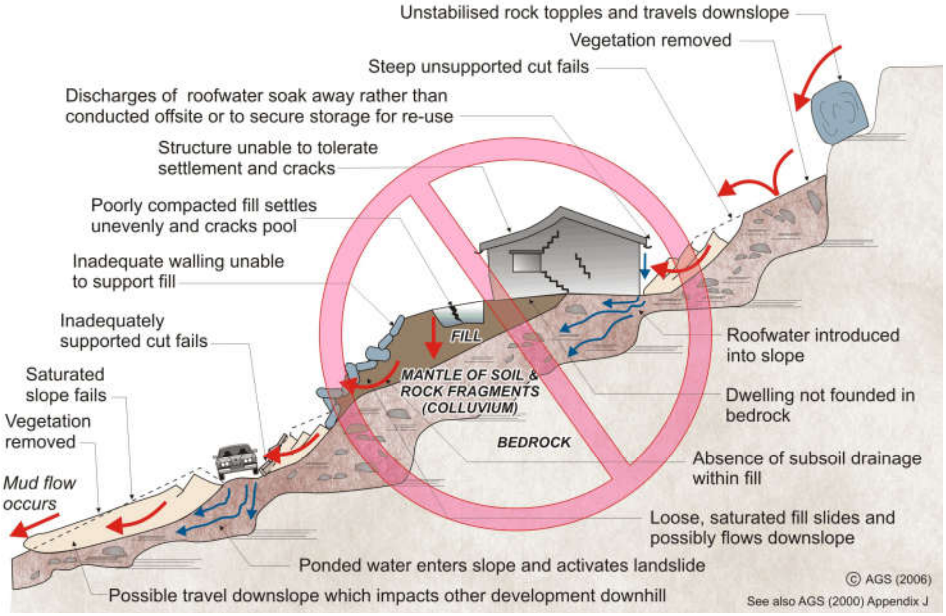
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soaks into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herringbone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

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
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APPENDIX C

BOREHOLE LOGS BH1 TO BH3

FROM WITT CONSULTING PTY LTD

GEOTECHNICAL INVESTIGATION

Witt Consulting Pty Ltd
ABN 76 102 953 515
witt.com.au

Excavation No: BH1

Sheet: 1 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface: 2.52 m
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is$_{(50)}$ MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							PAVING				
							Gravelly SAND, yellow				
					0.50 1.00		FILL, silty sand, sandy clay, sandstone fragments, grey to black			Potentially Fill	
	▽				1.50 2.00 2.50		SAND, yellow, loose, moist saturated at 1.7 m standing water level observed at 2 m b.g.l				
							Silty CLAY, red and brown, firm to stiff, moist				
					3.00 3.50		Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	

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WITT Consulting

Witt Consulting Pty Ltd
ABN 76 102 953 515
witt.com.au

Excavation No: BH1

Sheet: 2 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
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Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is(50) MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							continued........Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	
					4.50						
					5.00						
					5.50						
					6.00						
					6.50						
					7.00						
					7.50						

Witt Consulting Pty Ltd
ABN 76 102 953 515
witt.com.au

Excavation No: BH1

Sheet: 1 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface: 2.52 m
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is(50) MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							PAVING				
							Gravelly SAND, yellow				
					0.50 1.00		FILL, silty sand, sandy clay, sandstone fragments, grey to black			Potentially Fill	
	▽				1.50 2.00 2.50		SAND, yellow, loose, moist saturated at 1.7 m standing water level observed at 2 m b.g.l				
							Silty CLAY, red and brown, firm to stiff, moist				
					3.00 3.50		Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	

0227

WITT Consulting

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ABN 76 102 953 515
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Excavation No: BH2

Sheet: 1 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface: ~2.9 m
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is$_{(50)}$ MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							GRAVEL				
					0.50		SAND, coarse yellow			Potentially Fill	
							Silty SAND, black, trace gravel				
					1.00 1.50		Silty SAND, yellow, loose, moist				
	▽				2.00 2.50		standing water level observed at 2 m b.g.l				
					3.00 3.50		Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	

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ABN 76 102 953 515
witt.com.au

Excavation No: BH2

Sheet: 2 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface:
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is(50) MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							continued........Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	
					4.50						
					5.00						
					5.50						
					6.00						
					6.50						
					7.00						
					7.50						

0229

WITT Consulting

Witt Consulting Pty Ltd
ABN 76 102 953 515
witt.com.au

Excavation No: BH2

Sheet: 1 of 3

Borehole Log

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface: ~2.9 m
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength Is(50) MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER							GRAVEL				
							SAND, coarse yellow			Potentially Fill	
					0.50						
							Silty SAND, black, trace gravel				
					1.00		Silty SAND, yellow, loose, moist				
					1.50						
	▽				2.00		standing water level observed at 2 m b.g.l				
					2.50						
							Interbedded Siltstone & Sandstone, residual soil to extremely weathered, grey and yellow brown, very low strength			Rnn	
					3.00						
					3.50						

WITT Consulting

Witt Consulting Pty Ltd
ABN 76 102 953 515
witt.com.au

Borehole Log

Excavation No: BH3

Sheet: 1 of 1

Job No: WittC-MATTOX-R-A

Client: Tony Mattox
Principal:
Project: Barrenjoey Road Palm Beach
Site location: 1102 Barrenjoey Road Palm Beach NSW 2108

Date commenced: 23/10/2019
Date completed: 23/10/2019
Logged by: NK

Equipment type: Hanjon D&B 8D
Excavation dimensions: 100 mm Diameter

R.L. surface: 2.9 m
Easting:
Northing:

Vertical datum:
Horizontal datum:

Method	Water	TCR	RQD (SCR)	R.L. (m)	Depth (m)	Graphic Log	Material SOIL TYPE; plasticity or particle characteristics, colour, secondary and minor components ROCK TYPE; weathering, colour, secondary and minor components	Weathering	Inferred Strength $Is_{(50)}$ MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH		Average Defect Spacing (mm) 20 60 200 600 1000
SOLID FLIGHT AUGER	0.6 m Above Ground Level						GRAVEL				
							Interbedded Siltstone & Sandstone, residual soil to slightly weathered, grey, very low strength			Rnn	
					0.50						
					1.00						
					1.50						
					2.00						
					2.50						
					3.00						
					3.50						
							End borehole 4 m B.G.L				

APPENDIX D

Proposed Construction Methodology for Excavation of the Sandstone Boulders on the Southern Site Boundary

Date: 26 June 2023
Ref: 33618Ylet2rev4

Reform Projects Pty Ltd
15/108 Dunning Ave
ROSEBERY NSW 2018

Attention: Alex Swiney
Email: aswiney@reformprojects.com.au

GEOTECHNICAL ADVICE
PROPOSED CONSTRUCTION METHODOLOGY FOR EXCAVATION OF THE SANDSTONE BOULDERS ON THE SOUTHERN SITE BOUNDARY
1102 BARRENJOEY ROAD, PALM BEACH, NSW

1 INTRODUCTION

Construction of the proposed development will result in a finished floor level of RL-0.615m along the southern boundary of 1102 Barrenjoey Road, Palm Beach. Construction of the basement will typically result in excavation to a maximum depth of about 3.2m, although immediately adjacent to the southern boundary the existing cut is up to about 7m higher than current excavated levels and excavation on this existing cut will also be required. On Thursday 19 January 2023 JK Geotechnics met with Warwick Davies of EI Australia to discuss and understand the geotechnical comments relating to the development.

Investigations completed by JK Geotechnics, Witt Consulting, DF Dickson & Associates and Davies Geotechnical Consulting Engineers (DG) have revealed that the site is underlain by two different subsurface profiles. Over roughly the western half of the proposed basement excavation sandy soils are present that overlie an interbedded siltstone and sandstone bedrock which lies at approximately the proposed bulk excavation level. Groundwater was present within these sandy soils and was measured at levels as high as RL0.9m. Over the eastern portion of the site interbedded siltstone and sandstone bedrock was encountered at relatively shallow depth.

Excavation of the western portion of the site has previously been completed and current levels are at approximately RL2.7 to RL2.8m. A soldier pile retaining wall approximately 4.6m in height is present approximately 7.5m in from the eastern boundary. Along the southern boundary excavation has been completed to a maximum depth of about 6.5m. Along this boundary, just at the western extent of the shallow bedrock profile, is a large boulder (B1) that straddles the boundary between 1100 and 1102 Barrenjoey Road. This boulder is approximately 2m thick and is present at ground level and has toppled from Hawkesbury Sandstone beds present higher up in the topography. Below the boulder shotcrete is present and provides support to these materials from just above the current excavation level to the underside of the boulder. A second apparently smaller boulder (B2) is also exposed below the western end of this upper boulder at the current bulk excavation level. Low to medium strength siltstone is exposed at the base of the excavation below the shotcrete and it is assumed that siltstone is predominantly present behind the shotcrete, although it is likely that soils are present directly below the boulders. The Dynamic Cone Penetration tests completed by DG just to the south of the site in 1100 Barrenjoey Road suggests that bedrock is present at about RL3.4m, which is the depth refusal occurred. Investigation completed by DG suggests that B2 also straddles the boundary and that a third boulder (B3) is present immediately to the south of this lower boulder and is located wholly within 1100 Barrenjoey Road. Based on the investigation completed and the extent of the three boulders shown on the produced plan, DG confirmed that none of the three boulders extended below the house located on 1100 Barrenjoey Road. The attached Sections A-A, B-B and C-C indicate the approximate dimensions of the boulders, the materials present and the proposed extent of the development. The attached plan indicates the location of the sections.

Prior to the commencement of excavation, some form of shoring/retention will need to be installed. Over the western portion of the site where water charged sands are present, a cantilevered secant pile wall is proposed. This wall will extend along the northern and southern site boundaries to the point where shallow bedrock is encountered beyond which point a soldier pile wall is proposed. Over heights of about 3m to 4m this soldier pile wall will either be anchored or internally propped. Over the length of the southern boundary where B1 is present the following practically achievable retention scheme is proposed. This is:

- Remove that portion B1 that extends into the alignment of the proposed excavation and construct an anchored or propped soldier pile retaining wall.

Below we have considered in more detail the potential construction methodology that may be adopted for this short section of the southern boundary and a possible inspection and testing regime that may also be adopted. In this regard the regime only relates to this short section of the southern boundary.

2 ANCHORED OR PROPPED SOLDIER PILE RETAINING WALL

2.1 ANTICIPATED CONSTRUCTION METHODOLOGY

The anticipated construction methodology comprises the following:

- Construct a working platform to the underside of B1.
- Cut back B1 to the alignment of the rear of the proposed basement shoring system to expose the materials below.
- With the exception of hand operated jack hammers, cutting back of B1 would be completed using non-percussive excavation techniques, such as rock saws, rotary grinders, rock splitting etc.
- Install the soldier piles from the platform starting from underside of B1 and extending to the required toe level. This would include the installation of piles through B2 and the underlying soils and sandstone and siltstone bedrock. Where possible, piles would be installed such that they span B2.
- Progressively remove the working platform and deepen the excavation to bulk excavation level to allow the construction of shotcrete and mesh panels and the installation of anchors or internal props.

This will result in the excavation of clayey soils, siltstone and sandstone bedrock and that part of B2 that extends beyond the proposed cutline. It is anticipated that excavation would extend to depths of no greater than about 2m before the shotcrete and mesh panels are progressively installed. Anchors or props would be installed at the depths nominated on the structural drawings with excavation extending to depths no greater than those shown on the structural drawings. This would typically be expected to be no greater than about 0.5m to 1m below the proposed anchor/prop locations.

- Once excavation is completed and the retaining walls are supported by the built structure the anchors/props may be destressed.

2.2 POSSIBLE INSPECTION AND TESTING REGIME

The geotechnical monitoring program aims to monitor the following:

- Placement of the working platform.
- Cutting back of upper boulder.
- Installation of the soldier pile wall.
- That excavation is appropriately completed in a staged manner to allow ground anchors/props to be installed.
- Confirm the ability of the temporary ground anchors to carry their design loads (if used).
- That excavation is completed to bulk excavation level and excessive over-excavation does not occur.

2.2.1 DILPIDATION SURVEY OF ADJACENT PROPERTY AND INSTALLATION OF VIBRATION MONITORS

Prior to the commencement of works a dilapidation survey of the property at 1100 Barrenjoey Road must be completed. In addition, continuous vibration monitors must also be installed. These should be solidly fixed to the property at 1100 Barrenjoey Road. The vibration monitors must measure transverse, vertical and longitudinal ground vibrations and their vector sum. The monitoring equipment must measure the vibration in terms of peak particle velocity as specified in AS2187. The equipment must be equipped with computer loggers which provide a graphical presentation of vibration velocity versus vibration frequency.

An alarm must be raised instantaneously when any of the three vibration components or their vector sum reaches the preset trigger level. The alarm can take any form provided the plant operator is made aware, in real time, when the vibration limit is being approached.

The completion of a dilapidation survey and installation of vibration monitors comprises a hold point.

2.2.2 PLACEMENT OF WORKING PLATFORM

The working platform must be placed as engineered fill. In this regard Level 1 earthworks testing must be completed in accordance with AS3798-2007 over the full height of the platform. Earthworks testing comprises a hold point.

Contingency
Where the fill is not placed under Level 1 earthworks control it must be removed and placed under Level 1 earthworks control. Where testing indicates that the density or moisture of the material does not comply with the specification the layer placed in which the test failure occurred must be re-worked until the layer complies with the earthworks specification.

2.2.3 CUTTING BACK OF UPPER AND LOWER BOULDERS (B1 AND B2)

During the cutting back of B1 and B2 vibration monitoring must be operated continuously. No percussive excavation techniques other than hand-held jackhammers are to be adopted. Prior to the commencement of works the excavation contractors work methodology must be submitted to the geotechnical engineer for review and approval. This will constitute a hold point. This work methodology must include an initial start-up meeting immediately prior to the commencement of excavation between the operator, builder and geotechnical engineer, which will allow all parties to confirm that they understand the risks present, the excavation technique that will be adopted and the mitigation measures that will be applied. This will constitute a hold point.

Contingency
Should transmitted vibrations exceed preset limits all work shall cease immediately. Details of the activities occurring at the time of the exceedance must be logged and the manager informed immediately. The work methods causing the exceedance must be identified and an alternative work strategy must be devised in conjunction with the plant operator and manager or geotechnical engineer.

2.2.4 INSTALLATION OF SOLDIER PILE WALL

The geotechnical consultant must be on site during the drilling of at least 30% of all piles along the southern shoring elevation to provide a degree of confidence that the piles have been satisfactorily constructed. The inspected piles must be evenly spaced over the full length of the wall with inspections commencing with the drilling of the first pile. It will be the contractor’s responsibility to coordinate site visits by the geotechnical consultant to satisfy these requirements. The geotechnical consultant will observe drilling performance and record the final pile depths. The geotechnical consultant will prepare a written site report for each daily inspection. The piles are not deemed approved until the written site report has been issued by the geotechnical consultant.

During piling the contractor must, as a minimum, keep a record of the pile location, diameter and toe RL. These records must be progressively supplied to the geotechnical engineer at not greater than weekly intervals starting from the commencement of piling. These records will then be compared to those recorded by the geotechnical engineer whilst they were on site during piling.

On completion of installation, the contractor must certify that the soldier pile walls have been constructed in accordance with the approved drawings.

Contingency
Where piles have been poured but are not considered to be satisfactory, excavation shall not commence until approval is provided by both the structural and geotechnical engineer. Where further analysis indicates that the pile(s) will not perform satisfactorily, additional piles, anchors and/or rock bolts may need to be incorporated in the design. The design of remedial measures will be provided by the geotechnical and structural engineers on an 'as needed' basis.

2.2.5 EXCAVATION

Excavation must be completed in a staged manner to allow the installation and stressing of each row of anchors/props. In this regard, excavation must be completed to no lower than the level detailed on the structural drawings and extend no deeper until all anchors/props have been installed, successfully stressed/jacked (in accordance with the requirements of Section 2.2.6 below) and locked off. This will comprise a hold point. At this stage, the geotechnical consultant must confirm that the excavation depth has not been exceeded and that geotechnical conditions are similar to those described in the geotechnical report. Further excavation cannot commence until the geotechnical consultant has confirmed, in writing, that excavation can continue.

This process must be repeated for all rows of anchors/props.

Contingency
If it appears that excavation extends below the approved level, then the geotechnical consultant may require that the excavation be backfilled to restore stability.

2.2.6 ANCHORS/PROPS

Prior to the installation of any anchors/props, the contractor's proposed anchor/prop specification must be reviewed and approved by the geotechnical consultant and structural engineer in accordance with the structural drawings.

Anchors/props will be installed in accordance with the structural drawings and the first excavation hold point reached. For each anchor, the contractor must record, as a minimum, the anchor number, the drilled diameter of the hole, the anchor inclination, the total drilled depth, the number of strands in each anchor, the diameter and yield stress of each strand, the working load of each anchor, the total anchor length, the free length and the bond length (and in accordance with any requirements shown on the structural drawings). These records must be progressively supplied to the geotechnical consultant and structural engineer at not greater than weekly intervals, starting from the commencement of anchoring.

Once each row of anchors is installed, a hold point is reached. This hold point ends once the grout reaches sufficient strength (and in accordance with any requirements shown on the structural drawings) to allow the stressing of the anchors and the contractor certifies that the anchors/props have been installed in accordance with the approved drawings. One third of all anchors evenly spaced along the length and height of the retaining wall must be stressed in the presence of the geotechnical consultant or structural engineer (and in accordance with any requirements shown on the structural drawings). Records of stressing for all anchors and jacking of all props must be supplied to the geotechnical engineer no later than 72 hours after stressing and lock off. Where multi-strand anchors are used, all strands must be proof-loaded simultaneously. All anchors must be proof-loaded to 1.3 times the design working load in a minimum of four equal increments with the load and extension of the anchors recorded (and in accordance with any requirements shown on the structural drawings). Once the proof-load is successfully held for a period of time specified by the structural drawings, the anchors must be locked off. Following the locking off of each row of anchors, the contractor and structural engineer (or geotechnical consultant) must certify that the anchors have been successfully proof-tested. At least 10% should be subjected to lift-off testing 72 hours after initial stressing. If any anchor fails the lift-off test, all anchors should be re-tested.

Contingency

Where anchors are not successfully stressed and do not hold their proof-load, the anchor will be replaced as directed by the geotechnical consultant or structural engineer (and in accordance with any requirements shown on the structural drawings). These replacement anchors must be proof-loaded as detailed above to confirm that they have the capacity to carry the design loads. Where props have not been installed at the correct location or cannot hold the jock load they will be reinstalled at the correct location and a the jack replaced or repaired such that it is capable of carrying the design load.

De-Stressing

The anchors and props must be de-stressed once the permanent structure has been completed and provides adequate lateral support to the soldier pile wall and in accordance with the requirements of the structural drawings. The structural engineer or geotechnical consultant must inspect the de-stressing of at least 10% of all anchors, evenly spaced over the length and height of the wall. The contractor must certify that all anchors have been de-stressed.

3 PROPOSED CONSENT CONDITION

In the development of the Construction Certificate plans required to be submitted to the Certifying Authority, the structural engineer is to consider the content of JK Geotechnics letter and earlier geotechnical report (Ref: 33618Ylet3, dated 31 January 2023 and Ref: 33618YJrptrev3, dated 16 September 2022). A detailed construction methodology for the retention of the southern boundary is to be included in the structural drawings.

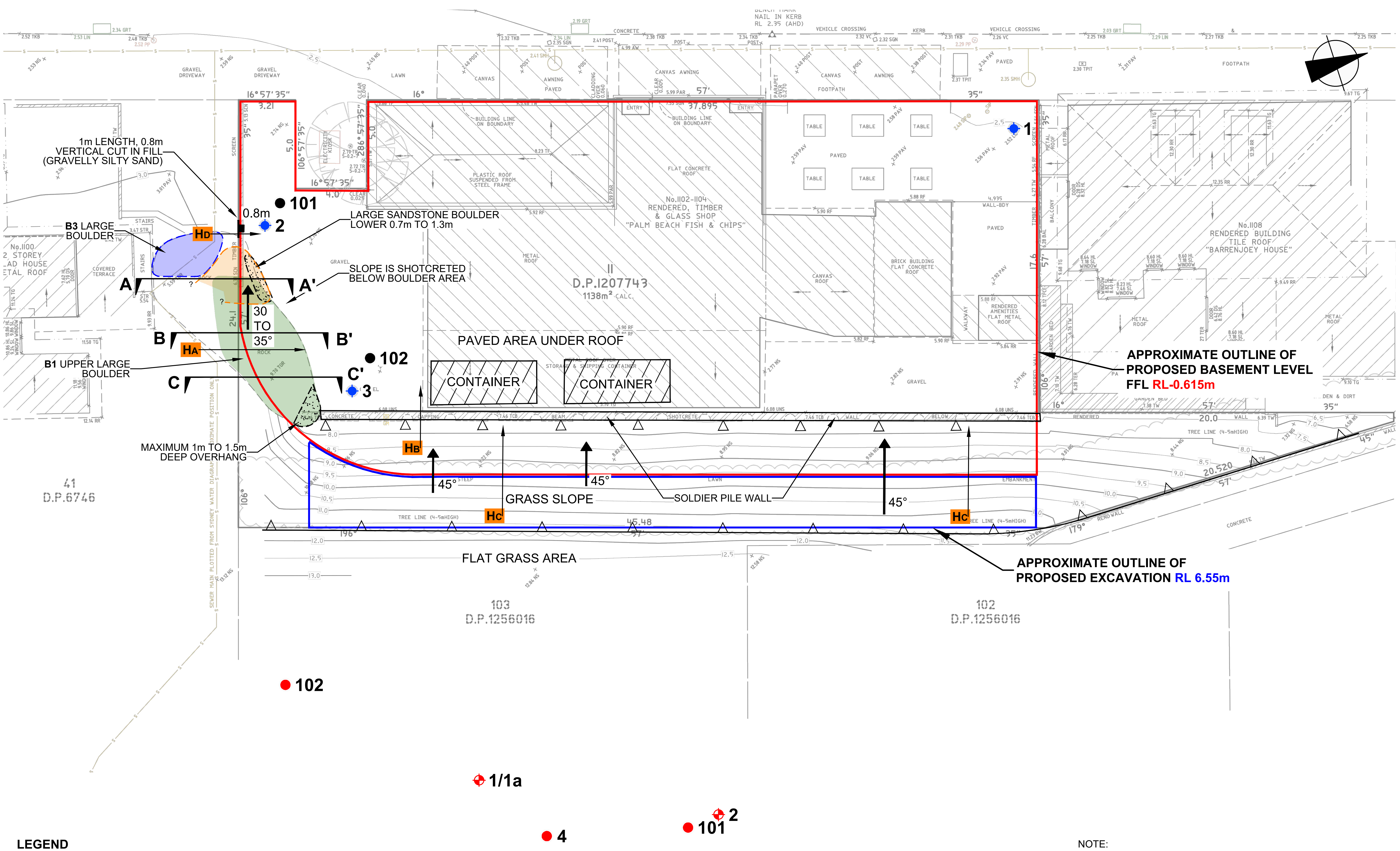
4 GENERAL COMMENTS

The proposed construction methodology and inspection and test regime presented above is of a generalised nature. There will inevitably be some amendment of both the construction methodology and inspection and test regimes during the development of the structural drawings.

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

Woodie Theunissen



NOTE:
REFER TO FIGURE 2 FOR CROSS SECTIONS A-A', B-B' AND C-C'.

LEGEND

●	BOREHOLE AND WELL (JK GEOTECHNICS, 2020)
⊕	BOREHOLE AND WELL (WITT CONSULTING, 2019)
●	BOREHOLE BY CROZIER GEOTECHNICAL CONSULTANTS
⊕	DCP TEST CROZIER GEOTECHNICAL CONSULTANTS
HD →	GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS
A A'	CROSS SECTION

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

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

Title: **PLAN SHOWING LOCATIONS OF SECTIONS**

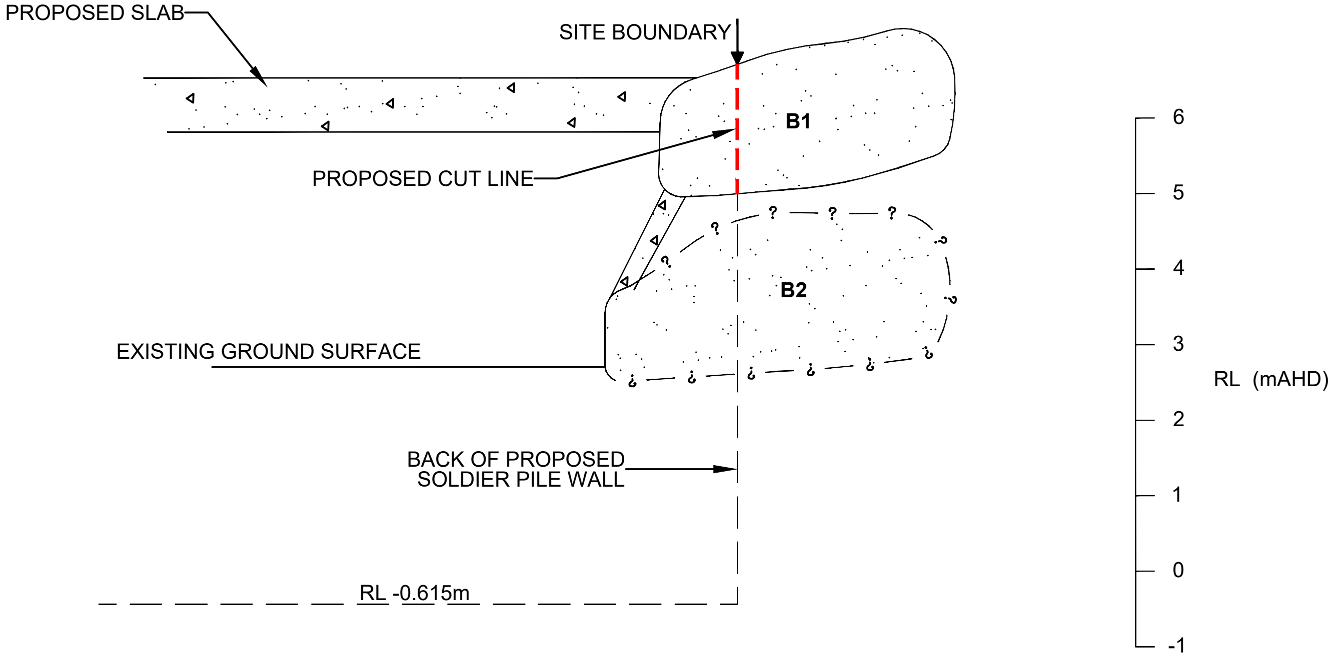
Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJlet2rev4

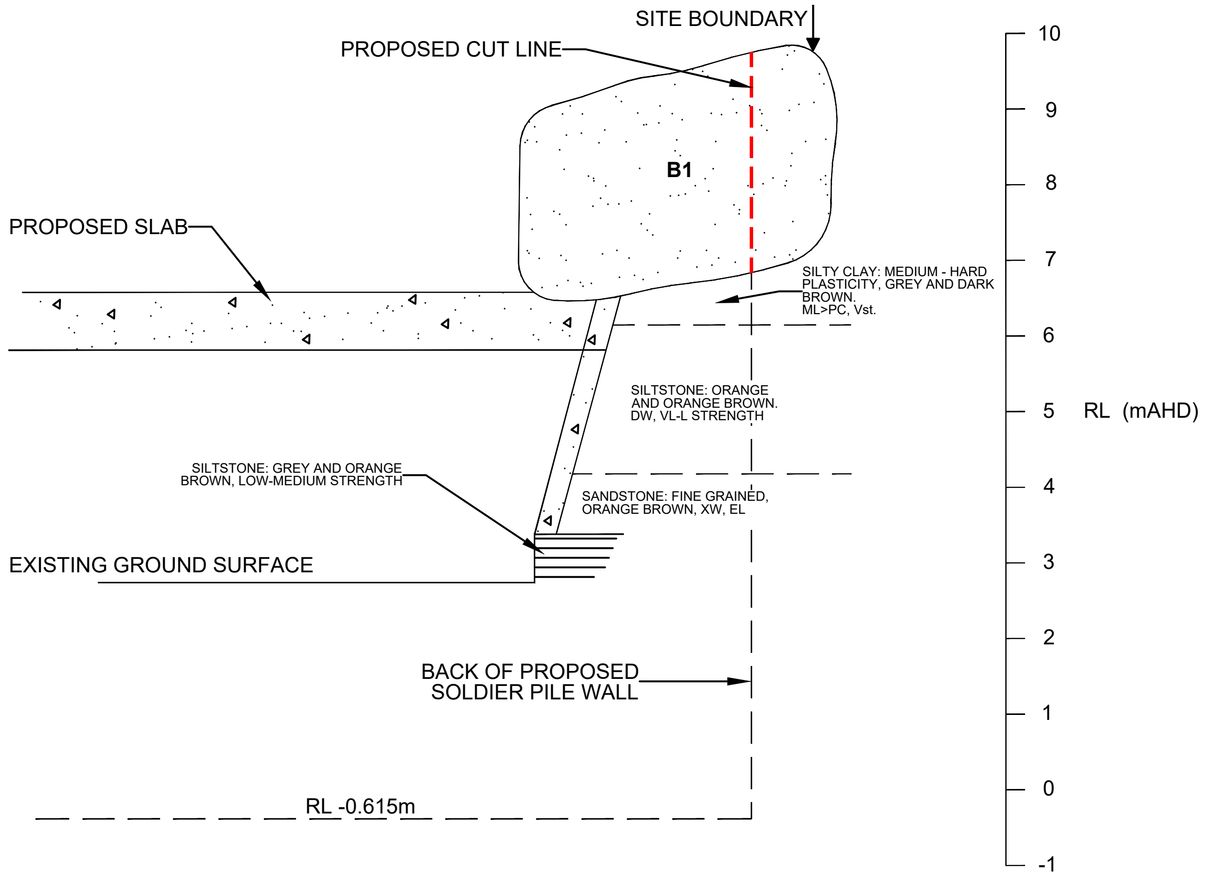
Figure No: 1

JKGeotechnics

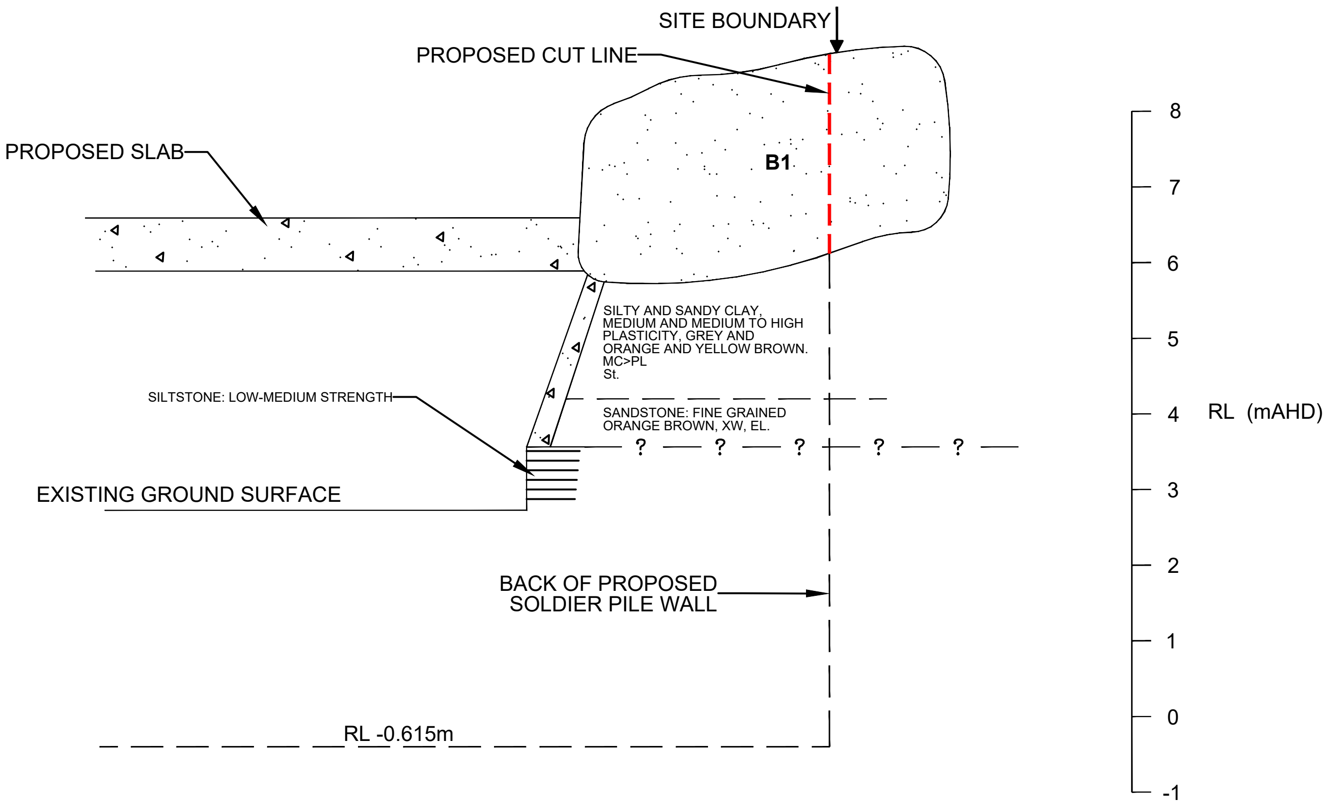
PLOT DATE: 29/06/2023 2:40:58 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJLET2REV4.DWG



SECTION A-A



SECTION C-C



SECTION B-B

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

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

Title:
CROSS SECTIONS A-A', B-B' AND C-C'

Location:
1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

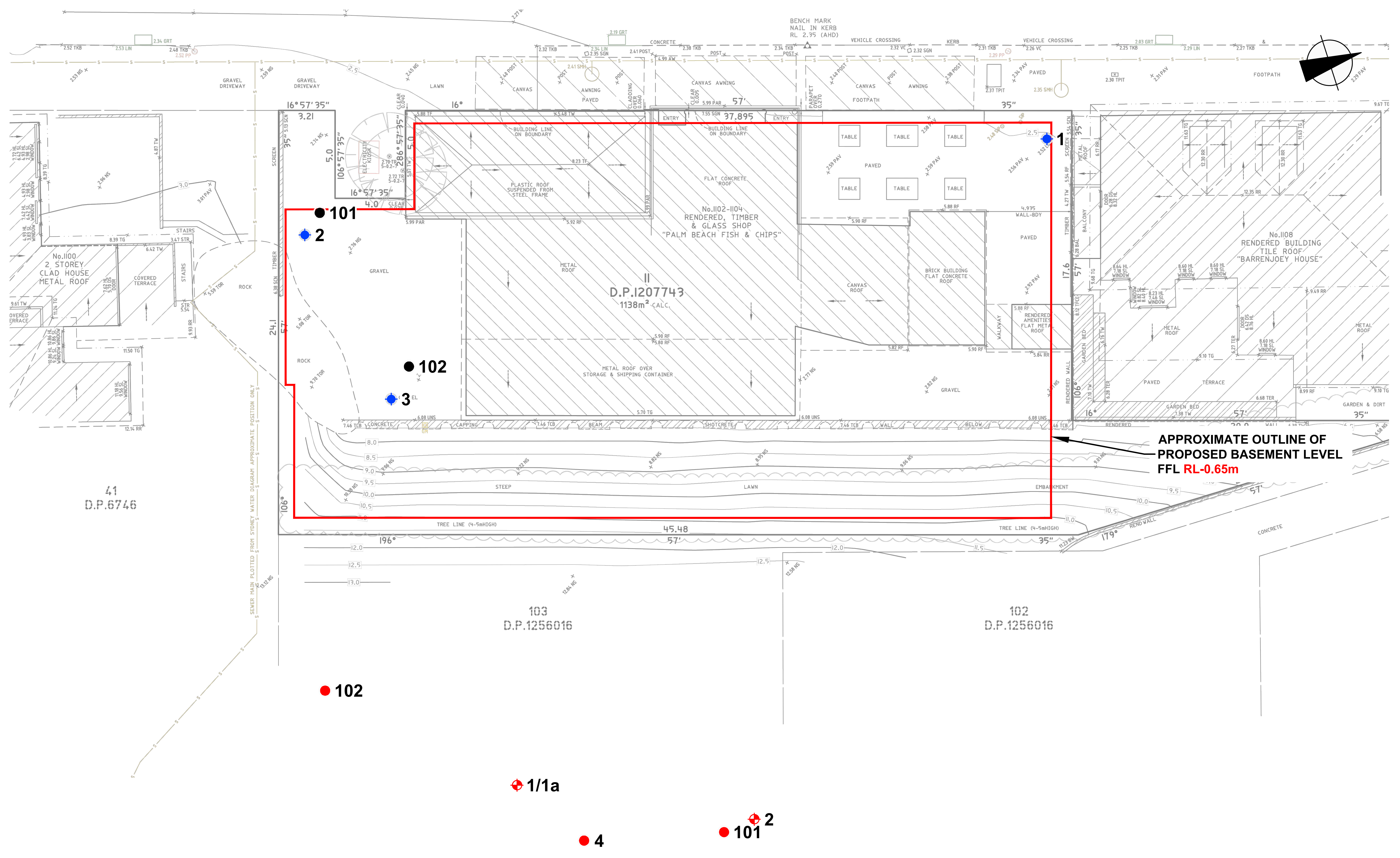
Report No: 33618YJlet2rev4
Figure No: 2

JKGeotechnics

PLOT DATE: 29/06/2023 2:41:06 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618YJLET2REV4.DWG

APPENDIX B1

0242



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LEGEND

- ● BOREHOLE AND WELL (JK GEOTECHNICS, 2020)
- ⊕ BOREHOLE AND WELL (WITT CONSULTING, 2019)
- ● BOREHOLE BY CROZIER GEOTECHNICAL CONSULTANTS
- ⊕ DCP TEST CROZIER GEOTECHNICAL CONSULTANTS

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

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

Title: **BOREHOLE LOCATION PLAN**

Location: 1102 - 1106 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618YJrptrev4

Figure No: 1A

JKGeotechnics

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

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

Title:

BOREHOLE LOCATION PLAN

Location: 1102 BARRENJOEY ROAD,
PALM BEACH, NSW

Report No: 33618Y

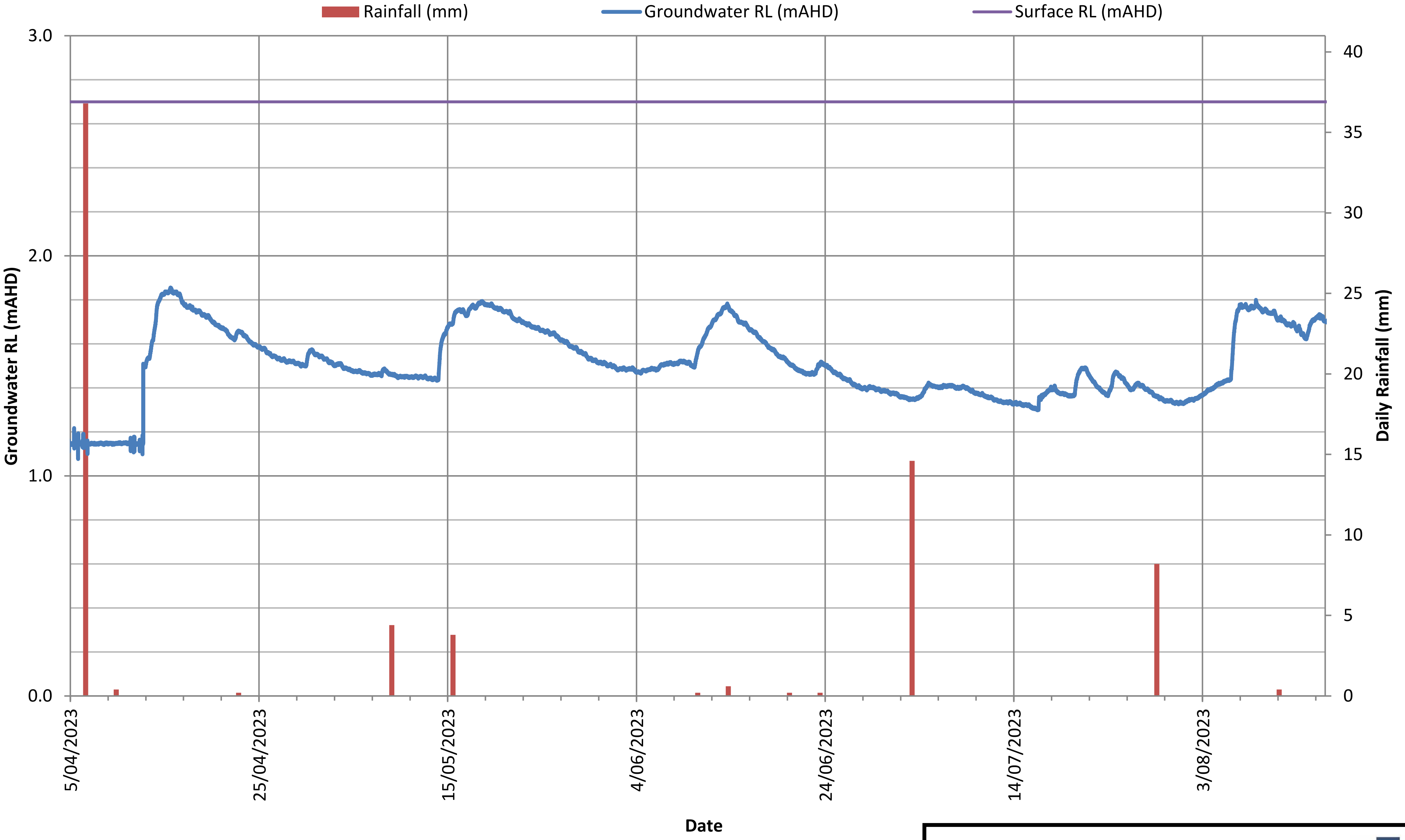
Figure No: 1B

JKGeotechnics

PLOT DATE: 21/06/2024 3:17:14 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\33000'S\33618YJ PALM BEACH\CAD\33618Y.DWG

File Name: 33618Y Palm Beach Groundwater Monitoring Plot 16.08.23
Date Printed: 21/06/2024

Groundwater Level and Daily Rainfall -v- Time Plot
BH102



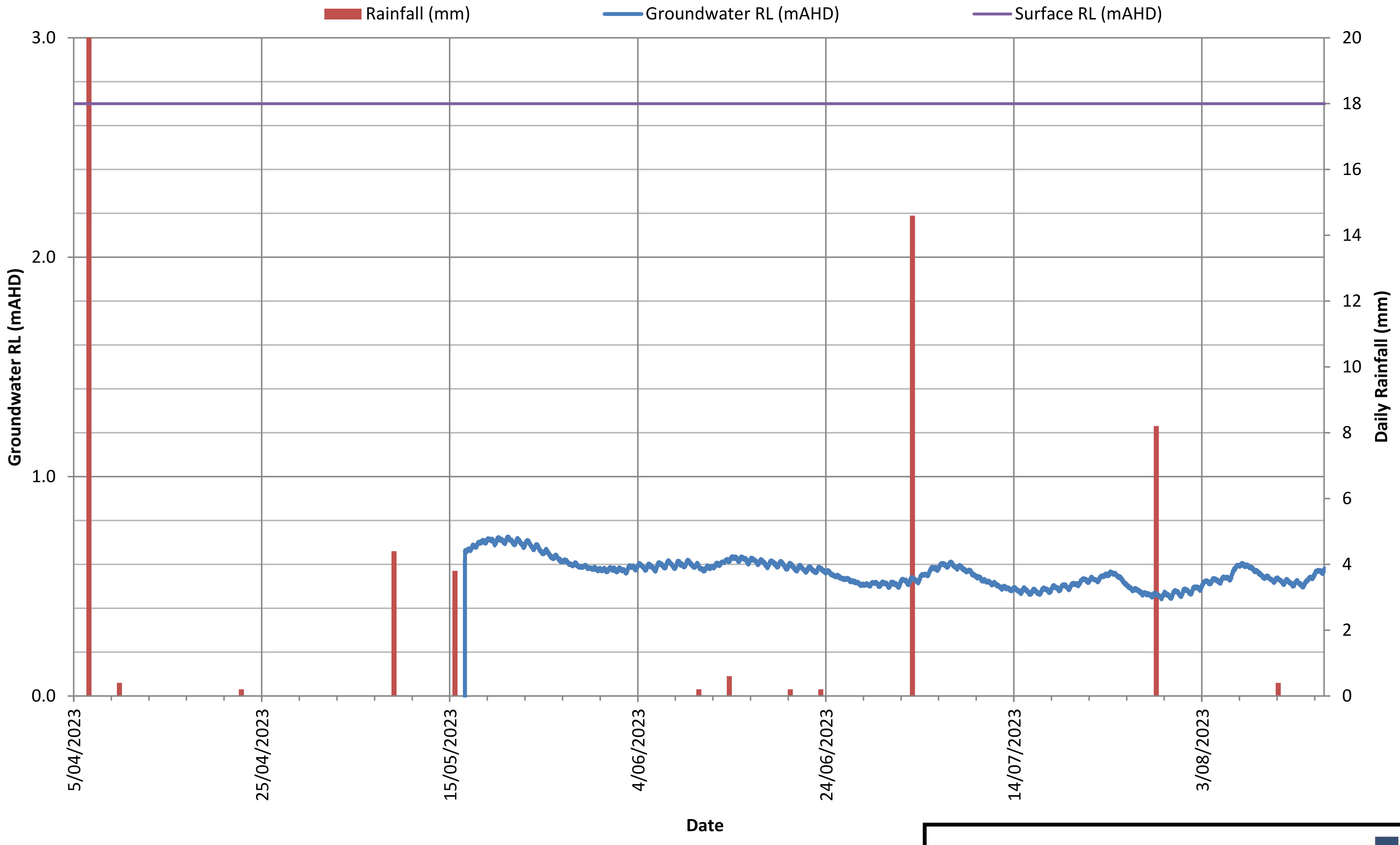
Rainfall data from Holsworthy Control Range, Station number : 6711

JKGeotechnics

Report No. 33618YJ **Figure No. 2**

File Name: 33618Y Palm Beach Groundwater Monitoring Plot 16.08.23
Date Printed: 21/06/2024

Groundwater Level and Daily Rainfall -v- Time Plot
BH201



Rainfall data from Holsworthy Control Range, Station number : 6711

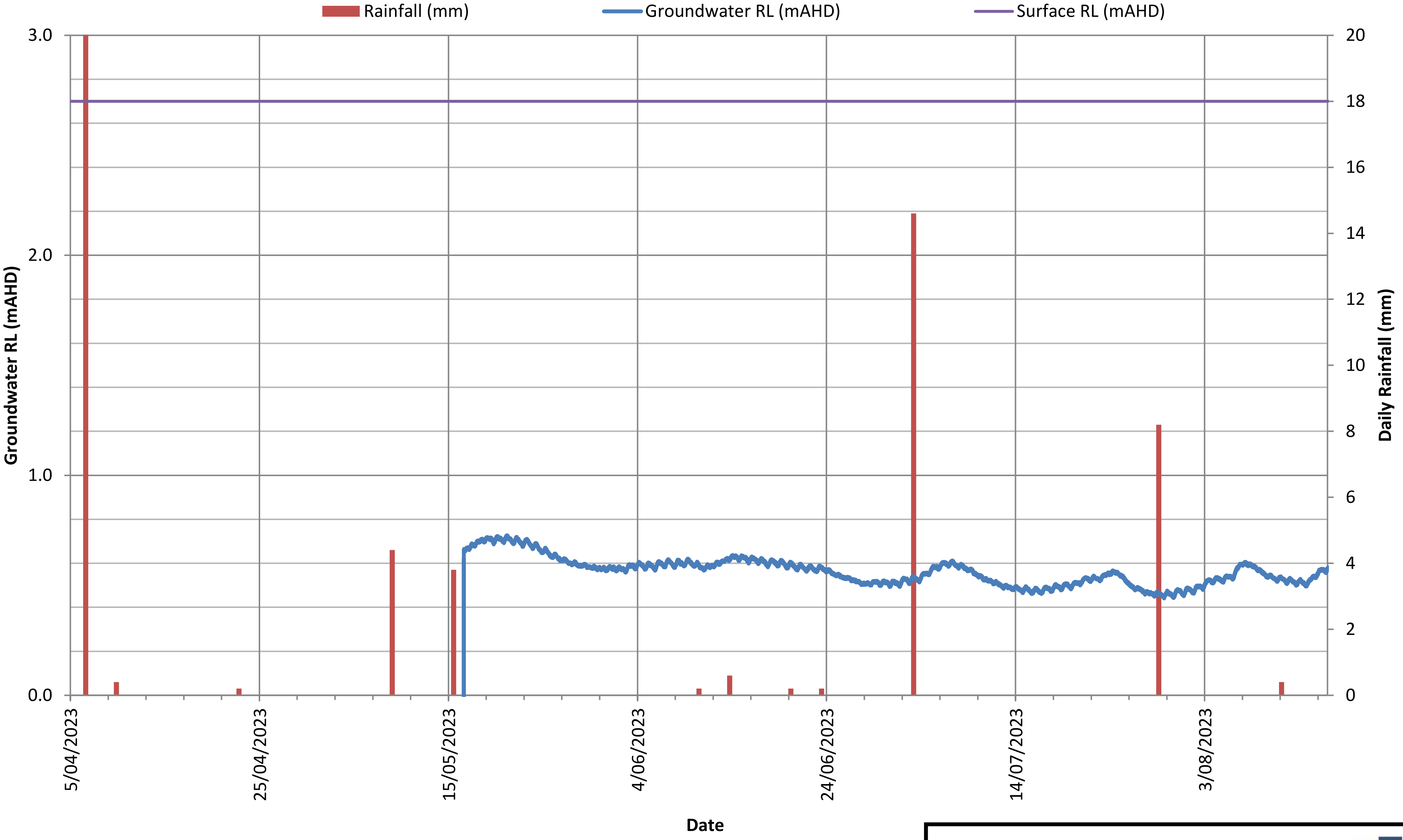
JKGeotechnics

Report No. 33618YJ **Figure No. 3**

File Name: 33618Y Palm Beach Groundwater Monitoring Plot 16.08.23
Date Printed: 21/06/2024

Groundwater Level and Daily Rainfall -v- Time Plot

BH201

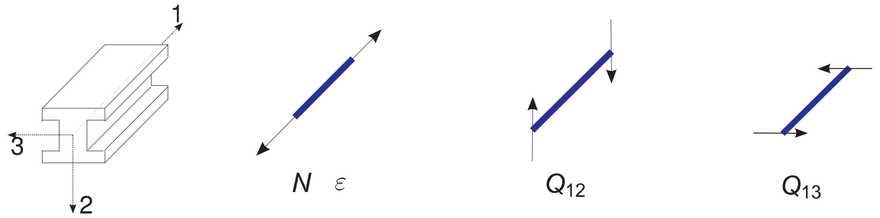


Rainfall data from Holsworthy Control Range, Station number : 6711

JKGeotechnics

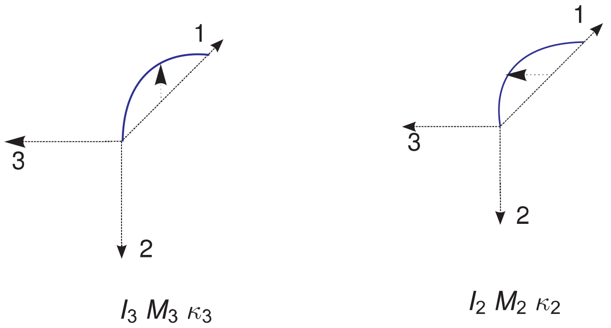
Report No. 33618YJ **Figure No. 3**

APPENDIX C1



a. Local Axes b. Axial force N c. Shear force Q_{12} d. Shear force Q_{13}

Figure 9.6 Positive axial force and shear forces in beams



a. Bending moment M_3 b. Bending moment M_2

Figure 9.7 Positive bending moments in beams

The *Bending moment* M_3 is the bending moment due to bending around the third axis (Figure 9.7a), whereas the *Bending moment* M_2 is the bending moment due to bending around the second axis (Figure 9.7b). The *Bending moment* M_1 is the *Torsion* (bending around the first axis).

9.4.4 RESULTING FORCES IN PLATES

When a plate is displayed, axial forces N_1 and N_2, shear forces Q_{12}, Q_{23} and Q_{13} and moments M_{11}, M_{22} and M_{12} are available from the *Forces* menu. These forces represent the actual forces at the end of the calculation step.

Note that axial forces are positive when they generate tensile stresses, as indicated in Figure 9.5. The sign of bending moments and shear forces depend on the plate's local system of axes.

The *Structure axes* option from the *View* menu may be used to display the plate's local system of axes (1,2,3). The first and second direction lie in the plane of the plate whereas the third direction is perpendicular to the plate.

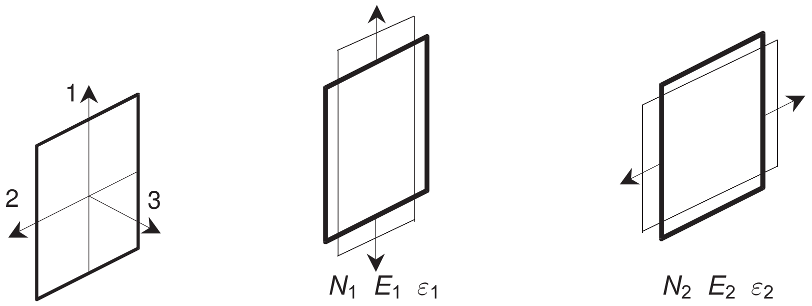
The *Axial force* N_1 is the axial force in the first direction (Figure 9.8b). The *Axial force N2* is the axial force in the second direction (Figure 9.8c).

The *Shear force* Q_{12} is the in-plane shear force (Figure 9.9a). The *Shear force* Q_{13} is the shear force perpendicular to the plate over the first direction (Figure 9.9b), whereas the *Shear force* Q_{23} is the shear force perpendicular to the plate over the second direction

(Figure 9.9c).

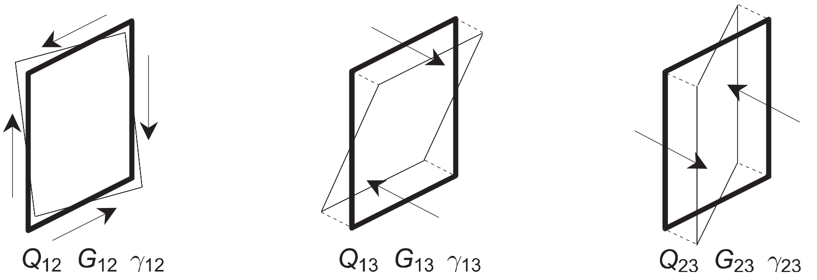
The *Bending moment* M_{11} is the bending moment due to bending over the second axis (around the second axis) (Figure 9.10b). The *Bending moment* M_{22} is the bending moment due to bending over the first axis (around the first axis) (Figure 9.10c).

The *Torsion moment* M_{12} is the moment according to transverse shear forces (Figure 9.10a).



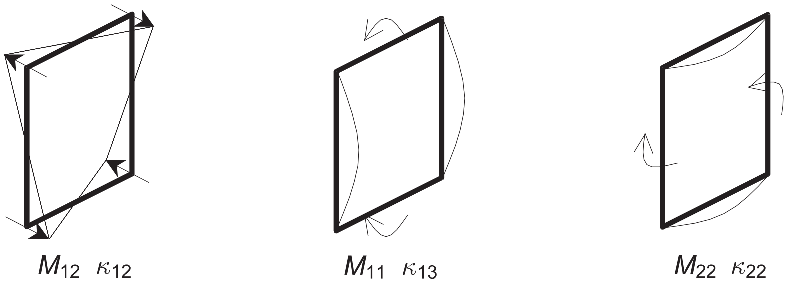
a. Local plate directions b. Axial force N_1 c. Axial force N_2

Figure 9.8 Positive axial forces in plates and geogrids



a. Shear force Q_{12} b. Shear force Q_{13} c. Shear force Q_{23}

Figure 9.9 Positive shear forces in plates



a. Torsion moment M_{12} b. Bending moment M_{11} c. Bending moment M_{22}

Figure 9.10 Positive bending moments in plates