Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants , a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED NEW DEVELOPMENT

at

154 – 158 PACIFIC PARADE, DEE WHY

Prepared For

Harrington Dee Why Pty Ltd

Project No.: 2024-193
March 2024

Document Revision Record

Issue No	Date	Details of Revisions
0	20 March 2025	Original issue

TABLE OF CONTENTS

APPENDICES

1	Notes Relating to this Report
2	Borehole Logs
3	Point Load Test Results
4	Risk Assessment Tables
5	AGS Terms and Descriptions

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants a division of PJC Geo - Engineering Pty Ltd

GEOTECHNICAL INVESTIGATION REPORT FOR PROPOSED FOUR STOREY DEVELOPMENT WITH BASEMENT 154 -158 PACIFIC PARADE, DEE WHY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a four-story mixed use development with two levels of basement parking at No.154 to No.158 Pacific Parade, Dee Why, NSW. The investigation was undertaken at the written request of Platform Architects on behalf of the client Harrington Dee Why Ptd Ltd.

Crozier Geotechnical Consultant (CGC) has previously undertaken a preliminary geotechnical assessment report to support the Development Application (DA) submission (Project No.: 2024-193, Dated 24 October 2024) however a sub-surface investigation is required to provide information on ground conditions to assist in the structural design of the development.

The investigation and reporting were undertaken as per the Fee Proposal P24-417.4, Dated: 15 November 2024 with the exception that drilling was required to significantly greater depths than initially proposed from 3 bores to between 12m-15m depth (a combined meterage of between 36.0m to 45.0m) to 3 bores to depths of 30.3m, 9.45m and 22.0m (a combined meterage of 61.75m).

Northern Beaches Council's - Warringah 2011 LEP and DCP states that all building development applications must be accompanied by a geotechnical landslip assessment. That developments within Class 'A', 'B' and 'D' landslip risk zone may require a preliminary assessment only where excavation/fill is <2.0m depth, however Class 'C' and 'E' sites and where excavation/fill >2.0m depth is proposed in other sites then a full geotechnical report is required.

This site is located within landslip risk Class 'A' within the Landslip Risk Map – Northern Beaches Mapping portal. A review of the preliminary checklist and the proposed works identified that the Development Application (DA) involves works which exceed the preliminary assessment guidelines.

The investigation comprised:

a) A DBYD Plan review and on-site clearance of test locations by an accredited service clearance contractor.
b) Geotechnical inspection and mapping of the site and adjacent properties by a Senior Engineering Geologist.
c) A photographic record of existing site conditions.
d) Geotechnical boreholes at 3 locations using a combination of auger, wash-bore and core drilling techniques to a maximum depth of 30.3m.
e) Installation and development of one groundwater monitoring well.
f) Monitoring of groundwater levels within existing wells (installed by others) and the recently installed groundwater well.
g) Rock strength testing using Is50 Point Load Testing methods.

The following plans were supplied and relied upon for the work:

- Architectural Drawings – Platform Architects, Drawing No.: DA 1000 – 1006, 3000, Sections A – D, undated.
- Survey Drawing – David Stutchbury Registered Surveyor, Reference No.: 11770/23, Dated: 26/06/2023.
- Structural Sketch – MPN, Secant pile wall.

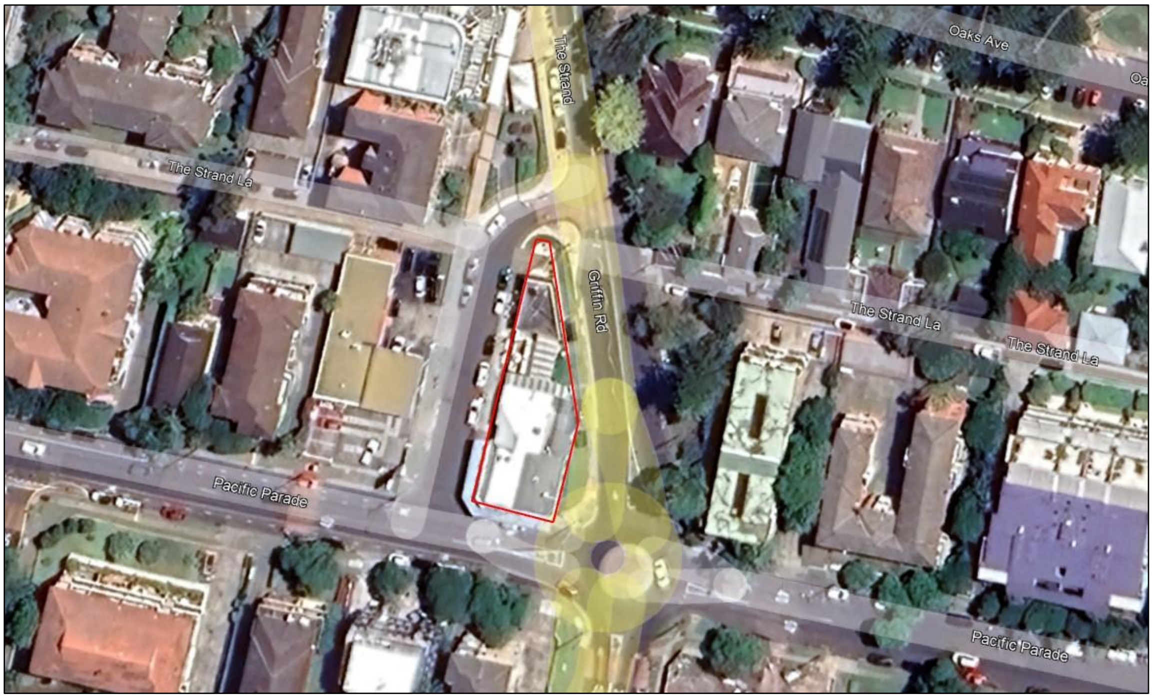
1.1 Proposed Development

It is understood that the development is to comprise a four-storey mixed use structure with two levels of basement parking under with the deepest excavation depth of approximately 8.0m underlying the south end of the structure to achieve a basement slab level of RL5.87m. An excavation depth of approximately 6.0m will be required underlying the north end of the proposed structure due to the fall in ground surface elevation. The basement excavation will extend up to all the site boundaries. It is understood that the current preliminary structural design includes a secant piled wall to support the basement excavation followed by the construction of a tanked basement.

2.0 SITE FEATURES:

2.1. Description:

The site is irregular in shape and covers an area of approximately 550m^2 in plan as referenced from publicly available on-line information. It is located between The Strand (to the north and west), Griffin Road (to the east) and Pacific Parade to the south. It is located within gently north-east dipping topography and the ground surface elevation varies between a high of RL13.5m within the southeast corner and a low of RL15.5m near the north of the site. It has north, east (combined), south and west boundaries of 2.7m, 48.6m, 14.9m and 48.6m respectively as determined from the survey plan provided. An aerial photograph of the site and its surrounds is provided below (Photograph 1), as sourced from Google Earth.



Photograph 1: Aerial view of site (outlined red) and surrounds

The north end of the site contains a small area of grass and concrete outdoor areas.

The majority of the remainder of the site is occupied by two single-storey structures which contain a restaurant and bar premises with an open sheltered garden section located between the two structures and concrete floor slabs.

To the north and west, The Strand comprises an asphalt pavement with concrete kerb and pedestrian pavement. A strip of grass is present to the north of the site boundary within the Strand easement.

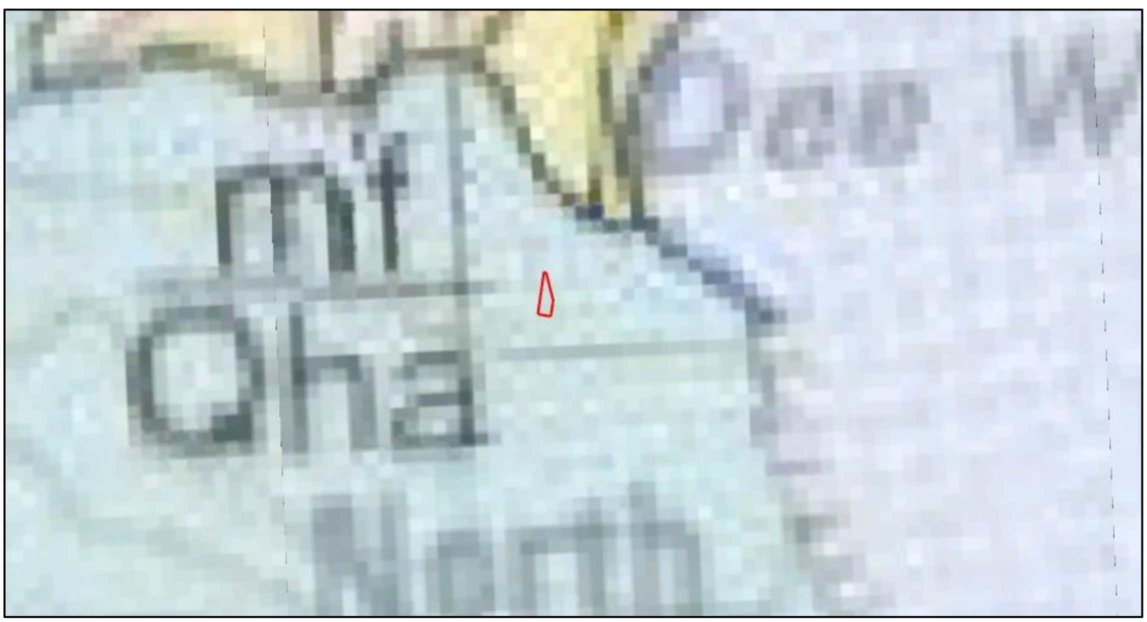
Griffin Road, to the east contains an asphalt carriageway with a concrete kerb and asphalt pedestrian pavement. A grass section is shown to the east of the site within the easement which contains an electricity substation.

To the south of the site, Pacific Parade contains an asphalt roadway with concrete pedestrian pavement and kerb.

2.2. Geological Setting

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is in an area underlain by Triassic deposits of the Hawkesbury Sandstone (Rh). The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminate.

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas is often rectilinear in plan, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defects orientations being south-east and north-east. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders. An extract of the relevant geological sheet is provided as Extract 1.



Extract 1: Extract from the relevant Geological Sheet series with the site outlined red.

3.0 FIELD WORK:

3.1. Methods:

The investigation involved the drilling of three boreholes (BH1 to BH3) within or directly adjacent to the site on the 11, 12 and 13 February 2025. Prior to commencement of drilling, the borehole locations were checked by an accredited service locator for underground services.

The boreholes were drilled using a Commachio Geo 205 geotechnical drilling rig to depths of 33.3m (BH1), 9.45m (BH2) and 21.5m (BH3). BH1 and BH3 were undertaken initially by utilising solid stem, spiral flight auger drilling techniques through the near surface soils/weathered bedrock prior to installing steel drilling casing. These boreholes were then extended utilising a combination of washbore or triple-tube core drilling techniques to acquire core samples/advance the borehole for logging purposes by a Senior Engineering Geologist. BH2 was completed using spiral flight auger techniques only.

Standard Penetration Tests (SPT's) were undertaken within the boreholes in general accordance with AS1289.6.3.1 – 1997, "Determination of the penetration resistance of a soil Standard Penetration Test (SPT)".

Geotechnical logging of the strata encountered was undertaken in accordance with AS1726:2017 'Geotechnical Site Investigations' and was based on inspection of cuttings recovered on the augers, supplemented with inspection of the SPT samples and the rock core recovered.

Following completion of drilling, a groundwater monitoring standpipe was installed in BH3. BH1 and BH2 were backfilled with arisings on completion.

Explanatory notes are included in Appendix: 1. Borehole log sheets are provided in Appendix: 2. A borehole location plan is provided in Figure 1.

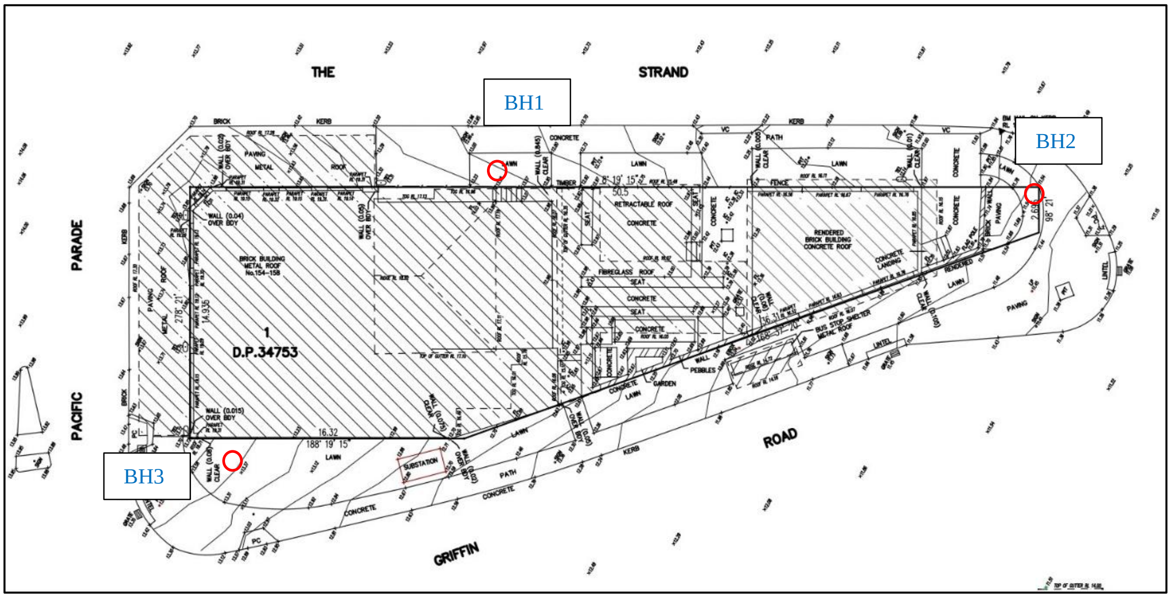


Figure 1: Borehole Location Plan

3.2. Field Observations:

The topography of the site and surrounding area dips very gently towards the north and east and outcrops of bedrock or soil cuttings were not observed.

The adjacent properties beyond the surrounding roads and easements adjacent to the site comprised residential unit blocks of brick construction formed entirely above ground surface levels, none of which appeared to display indications of cracking or deformation within the side walls and were all at least 15m from the site boundaries.

A service station lies to the west of the site and borders Griffen Road.

The adjacent carriageways and easements comprised asphalt surfaces with concrete kerbs including pedestrian pathways. Some minor cracking was observed within the pavement surfaces however it is not considered to represent a significant geotechnical issue.

The existing structures within the site were of rendered masonry construction and appeared in good condition with no significant cracking observed.

The neighbouring buildings and properties were only inspected from within the site or from the road reserve however the visible aspects did not show any signs of large-scale slope instability or other major geotechnical concerns which would impact the site.

3.3. Groundwater Monitoring

To allow the groundwater level to be monitored after completion of the fieldwork, a PVC standpipe piezometer was installed in BH3. Summary details of the construction of the piezometer are given in Table 1.

Table 1: Summary Piezometer Construction Details

Borehole	Piezometer Construction Details		
	Total Length (m)	Casing (m bgl)	Screen Section (m bgl)
BH3	8.3	0.0 – 3.0	3.0-8.3

bgl=Below ground level

4.0 INVESTIGATION RESULTS:

4.1 Ground Conditions

For a description of the subsurface conditions encountered at the borehole locations, the Borehole Log sheets should be consulted. However, a broad summary of the subsurface ground conditions encountered is given below:

Table 2: Summary of Subsurface Conditions at the borehole locations

Strata	m to base	Description
Fill	1.0m	Sandy clay fill was encountered to a maximum depth of 1.0m (BH3) and appears to generally comprise reworked natural soils.
Silty/Clayey Sand	3.50m	Underlying the fill at all borehole locations, medium dense silty clayey sand was encountered to a maximum depth of 3.5m which contained zones of sandy clay.
Sandy Clay/Clayey Sand	28.0m	Underlying the uppermost sand soil zone, a unit of firm to very stiff sandy clay and medium dense to very dense clayey sand was encountered to a maximum depth of 28.0m. Within BH1 and BH3 interpreted sandstone cobbles/boulders were encountered below depths 21.2m and 15.5m respectively. The cobbles/boulders required the use of core drilling techniques to advance the boreholes. However, this did result in core loss in both BH1 and BH3 within strata that has been interpreted as clayey soils between adjacent boulders.
Sandstone	–	Underlying the soils, bedrock was encountered within BH1 only at a depth of 28.0m. The unit initially comprised a horizon of extremely weathered very low strength strata which rapidly became uniform, medium strength sandstone below 28.2m depth and was relatively defect free.

Groundwater was encountered during auger drilling within all boreholes at depths of between 3.0m (BH3 and 3.9m (BH1). The use of water as a drilling flush medium precluded groundwater observation following commencement of washbore/core drilling.

The results of groundwater monitoring/development undertaken during one subsequent monitoring visit are shown in Table 3. The water level measured in the monitoring wells installed by others previously have also been provided in Table 3. Additional monitoring results will be provided in future letter reports for completeness.

Table 3: Groundwater Observations.

Location	**Groundwater Observations (m bgl) in standpipe**	
	11/02/2025	**05/03/25**
BH3	-	2.83
N06	3.08	2.86
N07	2.91	2.8
N09	2.53	2.83

bgl=Below ground level

4.2 Rock Core Photographs

Photograph 2 shows the core recovered during the field investigation which should be viewed in conjunction with the relevant borehole log sheet.

Borehole 1: 21.2m to 30.30m depth

Borehole 3: 17.0m to 21.0m depth

4.3 Laboratory Testing

Selected rock core samples recovered from BH1 were tested for measurement of rock strength, using Point Load Test [Is(50)] methods, in accordance with AS4133 4.

Point Load Test results are provided in Appendix: 3. Three rock core samples were tested both diametrically and axially and the results indicated that the bedrock tested ranged between very low to medium strength although the bedrock recovered below 28.2m was uniformly medium strength.

5.0 COMMENTS:

5.1. Geotechnical Assessment:

The proposed works involve construction of a multistorey mixed-use development with a two-level basement proposed below, requiring excavation to a depth of between approximately 6.0m and 8.0m.

Based on the investigation results it is anticipated that variable strength sand and clay soil will be encountered for the entire excavation depth and that shallow groundwater inflows (around 2.5m to 3.0m depth) will also be encountered. Due to the location/depth of the excavation and anticipated ground conditions, it is considered that continual support will be required for all excavation faces to protect adjacent properties/structure. It is understood that a secant pile wall is proposed to assist in the construction of a tanked basement and may be the preferred option by Council to reduce the impact of groundwater drawdown and future long-term dewatering of the basement. Driven support is not recommended based on the anticipated ground conditions.

To allow 'dry' construction of the basement slab, it is envisaged that temporary groundwater control will still be necessary unless a 'cut off' wall is adopted to effectively seal any inflows through the basement floor prior to the completion of a fully tanked basement.

Where a tanked basement is constructed, it is envisaged that the impact on adjacent properties will be minimal as the requirement for long-term dewatering is eliminated. It is expected temporary dewatering requirements would be low due to the proposed secant wall provided it is seated within a suitable horizon of low hydraulic conductivity. Where elevated levels of temporary dewatering is necessary, it may invoke the integrated development for NSW Act (Water Management Act NSW, 2000). Further analysis to the requirements of the NSW Government '*Minimum Requirements for Building Site and Groundwater Investigations'* October 2022 may also be required.

Due to the depth of excavation, it is envisaged that anchoring (or internal propping) will be necessary to provide temporary support to the basement until the construction of the permanent internal floor slab supports. The scale of the site and ground conditions interpreted indicate propping is the most viable and lower risk option.

Any temporary support will need to be designed based on the ground conditions encountered underlying the site. Where anchors are proposed, permission from adjacent properties will be required and anchors will need to be temporary in nature.

Unless a very 'robust' wall design is proposed it is envisaged that some minor deflection of the side walls may occur, and monitoring of the excavation crest (to an accuracy of not less than +/- 2mm horizontally and vertically) will be necessary to ensure any movements are detected at an early stage to allow refinement of support design as required. For anchored walls inclinometers are recommended within several of the support piles and measured on at least a weekly basis to detect, as early as possible, any potential movements above those anticipated to allow refinement in support to be undertaken if required.

It is envisaged that structure loads may need to found within bedrock of at least low strength (potentially stronger subject to structural design). This will require relatively deep footings (at least 28.0m depth) to found within bedrock. It will also be necessary to utilize a piling rig capable of drilling through boulders above the bedrock depth. It is envisaged either CFA drilling (or similar) will be required.

The proposed excavation is at least 15.0m from adjacent buildings and bulk excavation of bedrock is not anticipated therefore the impact on neighbouring nearby residential structures and services through vibration is not anticipated.

Based on the obtained DBYD Sydney Water Asset plans it is noted a 300mm diameter sewer lies within the adjacent roadway to the west (The Strand). Consultation is recommended with Sydney Water at an early stage to determine whether an SEA will be required as part of the development and will be subject to the exact location in relation to the zone of influence, which will have to be determined by Sydney Water.

5.2. Site Specific Risk Assessment:

Based on our assessment of available information we have identified the following geotechnical hazard which needs to be considered in relation to the proposed works. The hazard is:

A. Landslip (earth slide <$10m^3$) of soils from the proposed excavation.

The hazard has been assessed in accordance with the methods of the Australian Geomechanics Society (Landslide Risk Management, AGS Subcommittee, May 2002 and March 2007), see Tables: A and B, Appendix: 3 The Australian Geomechanics Society Qualitative Risk Analysis Matrix is enclosed in Appendix: 4 along with relevant AGS notes and figures. The frequency of failure was interpreted from existing site conditions and previous experience in these geological units.

We have undertaken two risk assessments, one assuming no or poorly constructed retaining walls are constructed in relation to the proposed development and a second assuming an engineer designed properly constructed basement retention system is constructed in accordance with this and previous reports as well as future geotechnical directives.

Scenario A: No or Poorly Designed/Constructed Retention System

The **Risk to Life** from **Hazard A** was estimated to be **6.14×10^{-2}** for persons within the roadway adjacent to the excavation, while the **Risk to Property** was considered to be **'Very High'**. The hazard was therefore considered to be '**Unacceptable'** when assessed against the criteria of the AGS 2007.

Scenario B: Engineer Designed and Properly Constructed Retention System

Where an engineer designed, basement and retention system are properly constructed the **Risk to Life** from **Hazard A** was estimated to be **2.40×10^{-7}** for persons within the roadway adjacent to the excavation, while the **Risk to Property** was considered to be **'Low'**. The hazard was therefore considered to be 'A**cceptable'** when assessed against the criteria of the AGS 2007.

As such the project is considered suitable for the site provided the recommendations of this report and any future geotechnical instruction are implemented.

6.0 Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

6.1.1. New Footings:	
Sub-grade material and Maximum Allowable Bearing Pressures for shallow footings	Very low strength bedrock: 750kPa Low strength bedrock: 1000kPa Medium Strength bedrock: 2500kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e – Shallow Soil site
Remarks: All new footings must be inspected and tested by an experienced geotechnical professional before concrete or steel are placed to verify the bearing capacity and the in-situ nature of the founding strata due to its easily disturbed state. This is mandatory to allow them to be 'certified' at the end of the project. All new footings should be founded within material of similar strength to reduce the potential differential settlement.	

6.1.2. Excavation:	
Depth of Excavation	Up to approximately 8.0m depth
Distance of Excavation to Neighbouring Properties/structures	>15.0m for building, directly adjacent to the Council easement/pavements.
Type of Material to be Excavated	A combination of firm to hard sandy/silty clay and medium dense to dense clayey/silty sand.
Remarks: Due to the proposed method of support (secant wall) it is envisaged only temporary batters will potentially be formed within the site prior to completion of the basement floor slab. To maintain safety of ground staff during the excavation of basement, temporary batter slopes should not exceed 1H: 1V. Geotechnical inspection of batters will be required at regular intervals. Sub-vertical batter slopes in clayey soils can stand unsupported over very short time frames however CGC cannot certify or recommend this approach. In addition, seepages within/through batter slopes will significantly reduce stability which must be considered where ground staff are working within the proposed excavation.	
Equipment for Excavation	Excavator with bucket
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Not required due to the depth of bedrock underlying the site and limited potential for vibration generation.
Vibration Calibration Tests Required	Not required
Full time vibration Monitoring Required	Not required

Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • During Installation of proposed secant wall for at least 75% of the retaining structures. • Inspection of footings, at least 75%. • Where unexpected ground conditions are identified, or any other concerns are held.
Dilapidation Surveys Requirement	Recommended for Council property external to the site to enable assessment of current conditions and protect against claims of damage due to potential defection of retention system.
Remarks: Drainage measures will need to be in place during excavation works to divert any surface flow away from any excavation crest and batter slopes prior to basement waterproofing, whilst any groundwater seepage must be controlled within the excavation to allow casting of a 'dry' basement and prevent softening/loosening of soils directly below the future floor slab.	

6.1.3 Retaining Structures:	
Required	It is understood a secant wall is proposed to support the excavation, will be need to be designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures is considered an suitable methodology.
Parameters for calculating pressures acting on retaining walls:	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill	18	$\phi' = 29°$	0.35	0.52	N/A
Sandy/Silty Clay	20	$\phi' = 30°$	0.33	0.47	3.25
Clayey/Silty Sand	20	$\phi' = 30°$	0.32	0.45	3.25

Remarks:
Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas.

6.1.4 Material Properties

Soil and rock properties for use in detailed wall analysis/design should be assessed by the designer, based on the results of this investigation and revised where required following the results of any future investigations/inspections at the site. However, the properties given in the tables below could be used in preliminary analysis. The properties should be used with caution and only by senior engineers familiar with soil structure analysis for deep basement excavations.

It is unknown whether a conventional trapezoidal pressure distribution approach is proposed for any sections of the retaining wall however the 'active/at rest' values provided in Section 6.1.3 could be used for rapid, preliminary analysis with the expectation that some economies in design and construction could be achieved following numerical analysis.

Material Strength Properties

Material	Strength	Undrained Analysis		Drained Analysis*	
		Cohesion (c_u) (kPa)	**Friction (φ_u) Degrees**	**Cohesion (c') (kPa)**	**Friction (φ') Degrees**
Sandy/Silty Clay	stiff	50	0	5	26
	very stiff	100			
	hard	200			
Silty/Clayey Sand	medium dense	-	28-30	-	28-30
	dense	-	35-38	-	35-38
Sandstone/Shale	very low	350 – 450	0	50	28
	low	600		100	35
	medium	1,000		200	40

Material Stiffness Properties

Material	Strength	Young's Modulus E – Mpa[1]
Sandy/Silty Clay	stiff	8-20
	very stiff	15-40
	hard	30-50
Silty/Clayey Sand	medium dense	20-50
	dense	40-80
Sandstone	very low	30-60
	low	50-100
	medium/high	150-500

6.1.5. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Groundwater as a near static water table was encountered at around 2.5m to 3.0m depth during drilling and recorded at similar depths within subsequent monitoring visits.
Excavation likely to intersect	Water Table	Yes
	Seepage	Yes
Site Location and Topography		Gently north dipping
Impact of development on local hydrogeology		Negligible due to tanked basement proposed, some dewatering may be required through further analysis by a groundwater specialist to allow dry construction of the basement, which will be subject to secant wall design depth.
Onsite Stormwater Disposal		Unsuitable due to the extent of development
Remarks: Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to a stormwater system off site.		

7. CONCLUSION:

Based on the results of the investigation the ground conditions within the excavation will comprise a combination of clayey/silty sand and sandy/silty clay to the full depth of the proposed basement and will require full support in order to maintain boundary stability (which is proposed to be supported via the construction of a secant wall). Underlying the site at a depth of at least approximately 28m, bedrock is anticipated which will rapidly grade from very low to low strength to medium strength. Groundwater inflows are anticipated from around 2.5m depth.

Groundwater control will also be necessary, and it is understood that a secant wall is proposed to allow the construction of a tanked basement which is considered an appropriate method to control long term groundwater control within the site and impacts external to the site via any dewatering methods which may be necessary.

Providing a properly constructed tanked basement is constructed, impact on adjacent structures through groundwater drawn down will be negligible.

Economies in design may be feasible where numerical analysis of the wall and temporary anchors/props is undertaken.

The potential for the generation of damaging vibrations is considered low due to the depth of bedrock encountered, however construction monitoring of any wall deflection should be undertaken to detect any movements outside those anticipated at an early stage.

The landslip risk was assessed as '**Unacceptable**' when assessed against the criteria of the AGS 2007. Where an engineer designed, basement is appropriately constructed the likelihood of any instability reduces and the risk becomes '**Acceptable**'.

Kieron Nicholson

Prepared by:	Reviewed by:
Kieron Nicholson	Troy Crozier
Senior Engineering Geologist	Principal Engineering Geologist
	MIEAust.CPEng(NER)
	MAIG. RPGeo; 10197

8. REFERENCES:

1. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
2. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin.

Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

NOTES RELATING TO THIS REPORT

Introduction

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

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination.
The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or 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, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is 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 determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg 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
  as 4, 6, 7 then N = 13
- In the 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 then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- 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 in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

Q_c (MPa) = (0.4 to 0.6) N blows (blows per 300mm)

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

Q_c = (12 to 18) C_u

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Borehole Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm “ “ “ “ “	C	Core		

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, 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. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

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.

Every care is taken with the report as it relates to interpretation of subsurface condition, 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 depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

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

Site Anomalies

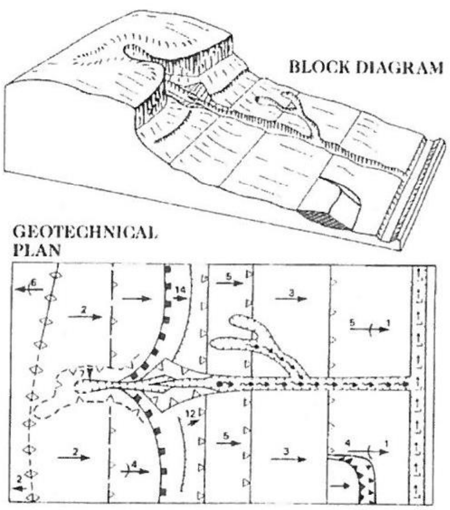
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

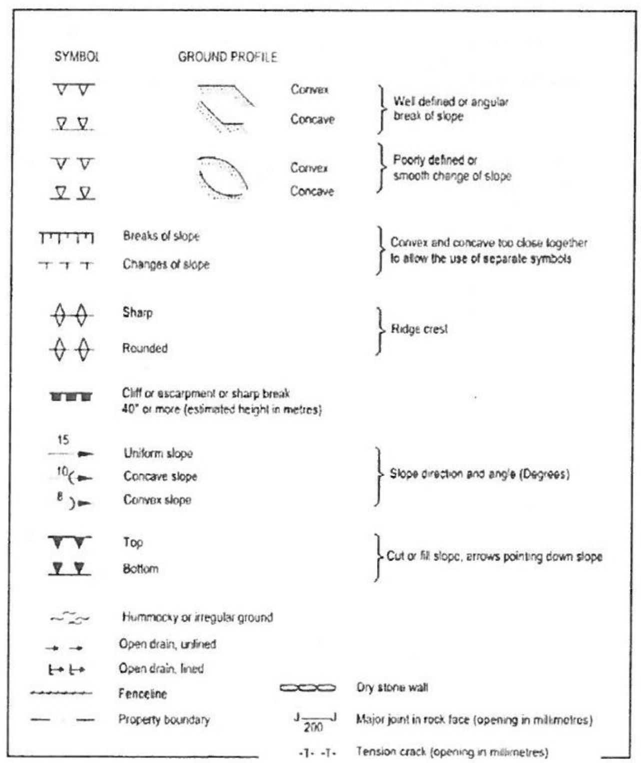
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. 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 special ally 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.

Site Inspection

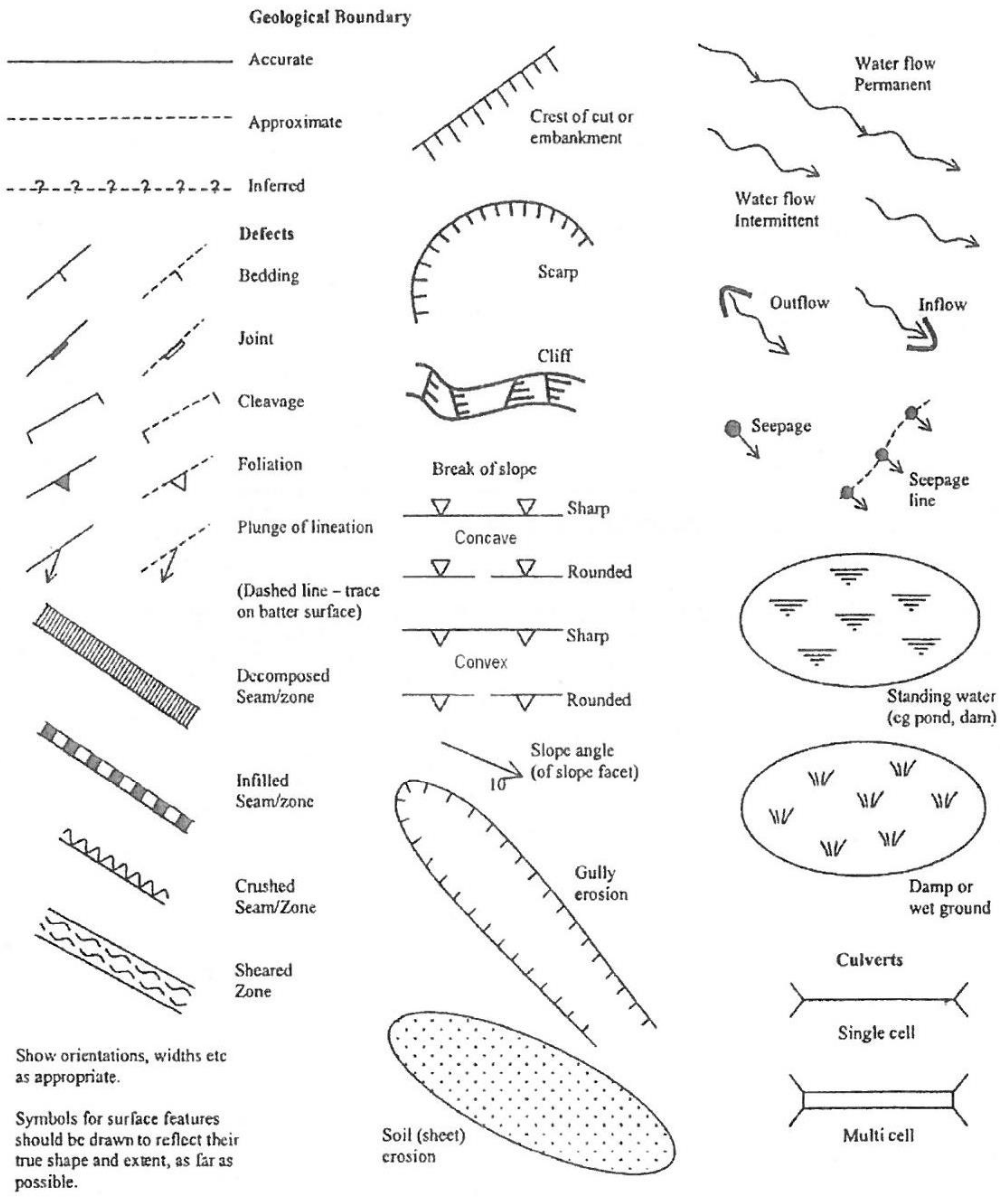
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2

Test Bore - Diamond Core Bore Logs

Client: Harrington Property | **Date:** 11/02/2025 | **Borehole:** 1

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL12.9m | **Sheet:** 1 of 3

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of (XW, HW, MW, SW, FR)			Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
0.00–0.30	FILL: Brown, sandy clay with fine-medium grained sandstone gravel									
0.30	SANDY CLAY/CLAYEY SAND: Medium dense, grey, medium grained, moist									
0.70	…coarse grained sand									
0.90										
1.00	CLAYEY/SILTY SAND: Medium dense, pale grey, medium grained, moist						S	1.0m – 1.45m		2. 6. 10 N=16
2.00										
2.60	…zone of 0.3m of sandy clay, 2.6m-2.9m depth						D	2.6m – 2.9m		
3.00							S	3.0m – 3.4m		9. 7. 3 N = 10
3.50	SANDY CLAY: Very stiff, grey, fine-medium grained sand, moist, zones of clayey sand									
4.00										
4.20	…pale grey						S	4.5m – 4.95m		18. 15. 8 N = 23
5.00										
6.00							S	6.0m – 6.45m		4. 6. 11 N = 17
7.00							S	7.5m – 7.95m		6. 8. 10 N = 18
8.00										
8.50	CLAYEY SAND: Dense, pale grey, locally brown, medium grained									
9.00							S	9.0m – 9.45m		9. 19.20 N = 39
9.90	…orange brown and grey									
10.00										

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger, washbore then NMLC coring | **Logged By:** KN

Water Observations: Groundwater at 3.9m depth | **Casing:** 21.2m

Comments: Borehole collapse to 1.5m, at 8.0-9.9m

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar
PT - parting	SN - stained	RO - rough	UN - undulating
SM - seam	VN - veneer	SO - smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped
SS - sheared surface			IR - Irregular
CS - crushed seam			
DB - drill break			
HB - handling break			

Test Bore - Diamond Core Bore Logs

CROZIER GEOTECHNICAL CONSULTANTS

Client: Harrington Property | **Date:** 11/02/2025 | **Borehole:** 1

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL12.9m | **Sheet:** 2 of 3

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of (XW, HW, MW, SW, FR)			Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
								10.5m		
							S			11, 15, 13 N = 28
11.00								10.95m		
11.50	SANDY CLAY: Stiff, orange brown and grey mottled, fine-medium grained sand, moist									
12.00								12.0m		
							S			3, 5,7 N = 12
								12.45m		
13.00										
								13.5m		
							S			3, 4, 7 N = 11
14.00								13.95m		
15.00										
16.00	CLAYEY SAND: Medium dense, red brown, medium grained, moist									
								16.5m		
							S			10, 10, 7 N = 17
17.00								16.95m		
18.00										
19.00										
								19.57m		
							S			11, 9, 9 N = 18
20.00								20.0m		

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger, washbore then NMLC coring | **Logged By:** KN

Water Observations: Groundwater at 3.9m depth | **Casing:** 21.2m

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar
PT - parting	SN - stained	RO - rough	UN - undulating
SM - seam	VN - veneer	SO - smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped
SS - sheared surface			IR - Irregular
CS - crushed seam			
DB - drill break			
HB - handling break			

Test Bore - Diamond Core Bore Logs

CROZIER GEOTECHNICAL CONSULTANTS

Client: Harrington Property | **Date:** 11/02/2025 | **Borehole:** 1

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL12.9m |

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of (XW HW MW SW FR)		Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
21.00									
21.20			START CORING from 21.2m depth						21.2
	Possible sandstone cobble at 21.2m-21.35m depth								
	SANDY CLAY: Firm, red brown, medium grained sand, moist								
22.00						C	77%	-	
22.20	…brown								22.55m pp= 100kPa (uncorrected)
22.70	…interpreted sandstone cobble		Washboring from 22.70m depth						22.7
23.00									
24.00									
24.10	…interpreted cobble								
25.00									
25.20			START CORING from 25.2m depth						
	Sandstone cobble 25.20m-25.38m								
	No Recovery								
26.00						C	65%	-	
26.10	SANDY CLAY: Stiff to very stiff, grey mottled orange red, sandstone cobble/gravel								
26.70	No Recovery								26.7m
27.00									
						C	33%	-	26.5m pp = 300kPa (uncorrected)
27.80	SANDY CLAY: Hard, grey, and brown, fine to medium grained gravel, laminated sandstone (extremely weathered sandstone)								
28.00	SANDSTONE: Low strength, fine to medium grained, brown								28.2m
28.20	…medium strength grey, laminated								
29.00						C	100%	100	
29.80	…zone of extremely weathered clayey sandstone		29.8m, PT, CN, CL, PL						
30.00	END OF BOREHOLE at 30.30m depth								30.30m

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger, washbore then NMLC coring | **Logged By:** KN

Water Observations: Groundwater at 3.9m depth | **Casing:** 21.2m

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar
PT - parting	SN - stained	RO - rough	UN - undulating
SM - seam	VN - veneer	SO - smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped
SS - sheared surface			IR - Irregular
CS - crushed seam			
DB - drill break			
HB - handling break			

CROZIER
GEOTECHNICAL CONSULTANTS

Test Bore - Diamond Core Bore Logs

Client: Harrington Property | **Date:** 12/02/2025 | **Borehole:** 2

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL11.6m | **Sheet:** 1 of 1

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of XW HW MW SW FR			Rock Strength Ex. Low, Very Low, Low, Medium, High, Very High	Fracture Spacing < 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m	Sample Type	Core Rec. %	RQD %	Test Results and Comments
	FILL: Brown, gravely clay									
0.50	SILTY SAND: Medium dense, dark grey, trace clay, moist									
1.00								1.5m		
1.65	SANDY CLAY: Firm, dark, grey, fine-medium grained sand, moist						S	1.95m		3, 4, 3 N = 7
2.00										
3.00	…zone of pale grey clayey sand, 3.1-3.35m depth							3.0m		
	…stiff below 3.40m depth						S	3.45m		9, 10, 7 N = 17
4.00										
	…very stiff below 4.5m depth							4.5m		
							S	4.95m		6, 9, 17 N = 26
5.00										
6.00	…zones of clayey sand							6.0m		
							S	6.45m		13, 11, 11 N = 22
7.00								7.5m		
							S	7.95m		4, 7, 12 N=19
8.00										
9.00								9.0m		
							S	9.45m		6, 12, 15 N = 27
	END OF BOREHOLE at 9.45m depth									
10.00										

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger to 9.45m | **Logged By:** KN

Water Observations: Groundwater at 3.9m depth | **Casing:**

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar RO
PT - parting	SN - stained	- rough	UN - undulating SO -
SM - seam	VN - veneer	smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped ,
SS - sheared surface		IR - Irregular	
CS - crushed seam			
DB - drill break			
HB - handling break			

CROZIER
GEOTECHNICAL CONSULTANTS

Test Bore - Diamond Core Bore Logs

Client: Harrington Property | **Date:** 13/02/2025 | **Borehole:** 3

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL13.4m | **Sheet:** 1 of 3

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of (XW, HW, MW, SW, FR)		Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
	FILL: Brown, sandy clay								
1.00	SILTY SAND: Medium dense, dark grey, fine to medium grained, moist								
1.50	…very dense, pale grey						1.5m		
						S			19, 17, 14 N = 31
							1.95m		
2.00									
2.20	SANDY CLAY: Soft to firm, dark grey, moist, fine to medium grained, zones of clayey sand								
3.00							3.0m		
						S			1, 1, 3 N = 4
							3.45m		
4.00									
4.30	…firm, pale grey								
							4.5m		
4.80	…zone of silty clay					S			1, 3, 4 N=7
							4.95m		
5.00									
6.00	…very stiff						6.0m		
						S			7, 11, 15 N = 26
							6.45m		
7.00	CLAYEY SAND: Very Dense, pale grey, fine-medium grained, wet								
							7.5m		
						S			7, 8, 23 N = 31
							7.95m		
8.00									
9.00	…dense						9.0m		
						S			11, 9, 10 N = 19
							9.45m		
10.00									

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger/washbore to 17.0m then NMLC (| **Logged By:** KN

Water Observations: Groundwater at 3.0m depth | **Casing:** 17.0m

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar RO
PT - parting	SN - stained	- rough	UN - undulating SO -
SM - seam	VN - veneer	smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped ,
SS - sheared surface		IR - Irregular	
CS - crushed seam			
DB - drill break			
HB - handling break			

Test Bore - Diamond Core Bore Logs

CROZIER GEOTECHNICAL CONSULTANTS

Client: Harrington Property | **Date:** 13/02/2025 | **Borehole:** 3

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL13.4m | **Sheet:** 2 of 3

Depth (m)	Description of Core Rock name, grainsize, texture/fabric, colour	Degree of (XW, HW, MW, SW, FR)			Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
10.50	…red brown									
11.00										
12.00										
13.00										
14.00										
15.00								15.0m		
15.50	cobbles from 15.5m-16.5m depth						S	15.45m		13, 17, 24 N = 41
16.00										
17.00	…sandstone boulder 17.0m-17.4m depth No Recovery-Inerpreted clay zone			START CORING FROM 17.0 m depth						17
18.00							C	30%	-	
18.50										18.5
18.70	Sandstone Boulder at 18.7-19.0m depth									
19.00	No Recovery, 19.0-20.0m depth						C	-	-	
20.00										

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger/washbore to 17.0m then NMLC (| **Logged By:** KN

Water Observations: Groundwater at 3.1m depth | **Casing:** 17.0m

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar RO
PT - parting	SN - stained	- rough	UN - undulating SO -
SM - seam	VN - veneer	smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped ,
SS - sheared surface		IR - Irregular	
CS - crushed seam			
DB - drill break			
HB - handling break			

Test Bore - Diamond Core Bore Logs

Client: Harrington Property | **Date:** 13/02/2025 | **Borehole:** 3

Project: Four Storey Mixed Use Development | **Project No.:** 2024-193 | **Dip:** | **Azimuth:** 90°

Location: 154-158 Pacific Parade, Dee Why | **Surface Level:** RL13.4m | **Sheet:** 3 of 3

Depth (m)	Description of Core: Rock name, grainsize, texture/fabric, colour	Degree of (XW, HW, MW, SW, FR)			Rock Strength (Ex. Low, Very Low, Low, Medium, High, Very High)	Fracture Spacing (< 0.05m, 0.05 to 0.20m, 0.20 to 0.50m, 0.50 to 1.00m, > 1.00m)	Sample Type	Core Rec. %	RQD %	Test Results and Comments
	Sandstone boulder, 20.0m-20.7m depth									
20.70	No Recovery						C	-	-	
21.00										
21.50	END OF BOREHOLE at 21.5m depth									
22.00										
23.00										
24.00										
25.00										
26.00										
27.00										
28.00										
29.00										
30.00										

Rig: Commachio Geo 205 | **Driller:** TT

Type of Boring: Auger/washbore to 17.0m then NMLC (| **Logged By:** KN

Water Observations: Groundwater at 3.1m depth | **Casing:** 17.0m

Comments:

Defect Type	Coating	Roughness	Planarity
JT - joint	CN - clean	VR - very rough	PL - planar
PT - parting	SN - stained	RO - rough	UN - undulating
SM - seam	VN - veneer	SO - smooth	CU - curved
SZ - sheared zone		SL - slickensided	ST - stepped ,
SS - sheared surface		IR - Irregular	
CS - crushed seam			
DB - drill break			
HB - handling break			

Appendix 3

POINT LOAD STRENGTH INDEX TEST RESULTS

CROZIER GEOTECHNICAL CONSULTANTS

Client: Harrington Dee Why Ptd Ltd	Date: 3 March 2025	Date Tested: 3 March 2025
Project: Mixed Use Development	Project No.: 2024-193	
Location: 154-158 Pacific Parade, Dee Why	Date Sampled:	

Borehole No.	Depth (m)	Sample Description (geology)	Test Type	Width (mm)	Platen Seperation (mm)	Failure Load (kN)	Is (MPa)	$Is_{(50)}$ (MPa)	Failure Mode*	Strength (AS1726-2017)
1	28.0-28.14	Sandstone	Diametral	-	52	0.19	0.07	0.07	1	VL
			Axial	52	40	0.50	0.19	0.19	1	L
1	28.7-28.89	Sandstone	Diametral	-	52	1.91	0.71	0.72	1	M
			Axial	52	57	2.87	0.76	0.83	1	M
1	29.5-29.69	Sandstone	Diametral	-	52	2.58	0.95	0.97	1	M
			Axial	52	45	2.75	0.92	0.96	1	M

AS4133.4.1 - Rock Strength Tests - Determination of a point load strength index

*Failure Modes

1	Fracture through fabric of specimen oblique to bedding, not influenced by weak planes
2	Fracture along bedding
3	Fractrure influenced by pre-existing plane, microfracture, vein or chemical alteration
4	Chip or partial fracture

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 4

TABLE : A

Landslide risk assessment for Risk to life-Poor Retention Measures

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <10m³) from excavation through potentially weak soils directly adjacent to all shared boundaries		a) and b) Landslide due to excavation through around 6.0m of weak, water charged soils would occur in a short period of time (almost immediately)	a) May engulf 100 % of pedestrian pavements adjacent to shared boundaries b) Likely to impact significant section of the road adjacent to shared boundaries		a) Person on pavement 1.0hrs/day b) Person in car	a) and b) Likely to not evacuate	a) May undermine pavement, engulfment possible b) May undermine road, engulfment car	
			Almost Certain	**Prob. of Impact**	**Impacted**				
		a) All pedestrian pavements surrounding the site.	0.1	1.00	1.00	0.04	0.75	1.00	**3.13E-03**
		b) Vehicles in all surrounding roadways adjacent to the site (Griffen Road, The Strand and Pacific Parade)	0.1	0.90	0.91	1.00	0.75	1.00	**6.14E-02**

* hazards considered in current condition without suitable retention measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

<u>**TABLE : B**</u>

Landslide risk assessment for Risk to Property-Poor Retention Measures

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide $<10m^3$) from excavation through potentially weak soils directly adjacent to all shared boundaries	a) All pedestrian pavements surrounding the site.	Almost Certain	Event is expected to occur over design life.	Catastrophic	Site structures completely destroyed, significant stabilising or MAJOR damage to neighbouring property.	Very High
		b) Vehicles in all surrounding roadways adjacent to the site (Griffen Road, The Strand and Pacific Parade)	Almost Certain	Event is expected to occur over design life.	Catastrophic	Site structures completely destroyed, significant stabilising or MAJOR damage to neighbouring property.	Very High

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

<u>**TABLE : A**</u>

Landslide risk assessment for Risk to life-Suitably Designed and Constructed Retention

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <10m³) from excavation through potentially weak soils directly adjacent to all shared boundaries		a) and b) Where an engineer designed, properly constructed retaining structure is built to suppor the excavtion, the likelihood of slide is barely credible	a) May engulf 100 % of pedestrian pavements adjacent to shared boundaries b) Likely to impact significant section of the road adjacent to shared boundaries		a) Person on pavement 1.0hrs/day b) Person in car	a) and b) Possible to not evacuate	a) May undermine pavement, engulfment possible b) May undermine road, engulfment car	
			Rare	**Prob. of Impact**	**Impacted**				
		a) All pedestrian pavements surrounding the site.	0.000001	0.70	1.00	0.04	0.5	1.00	**1.46E-08**
		b) Vehicles in all surrounding roadways adjacent to the site (Griffen Road, The Strand and Pacific Parade)	0.000001	0.60	0.80	1.00	0.5	1.00	**2.40E-07**

* hazards considered assuming adequate retention system constructed
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property-Suitably Designed and Constructed Retention Measures

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <10m³) from excavation through potentially weak soils directly adjacent to all shared boundaries	a) All pedestrian pavements surrounding the site.	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Low
		b) Vehicles in all surrounding roadways adjacent to the site (Griffen Road, The Strand and Pacific Parade)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Low

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

Appendix 5

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

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.

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.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

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.

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.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

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 judgements 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 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 Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – 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.

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.

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.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

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, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: 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^{-6}	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

APPENDIX C: – 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) For 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.