CROZIER GEOTECHNICAL CONSULTANTS

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Crozier Geotechnical Consultants is a division of PJC Geo-Engineering Pty

REPORT ON GEOTECHNICAL SITE INVESTIGATION

For

PROPOSED NEW RESIDENTIAL DEVELOPMENT

At

44 ROSE AVENUE, WHEELER HEIGHTS

Prepared For
Wheeler Heights Developments Pty Ltd

Project: 2017-083.1
January, 2019

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0	17th May 2017	Original Issue
1	23rd June 2017	Revised Issue – after receiving and review of DA Plans
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Date: 30th January 2019

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GEOTECHNICAL REPORT FOR PROPOSED NEW RESIDENTIAL DEVELOPMENT
44 ROSE AVENUE, WHEELER HEIGHTS, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed residential development at 44 Rose Avenue, Wheeler Heights, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Wheeler Heights Developments Pty Ltd.

The property No. 44 is situated on the low north side of Rose Avenue, within gently north-west dipping topography. The project site is a rear battle axe style block, currently occupied by a single storey brick dwelling with a spacious backyard.

It is understood that the proposed works involve demolition of the existing site structures and construction of a residential development. The residential development will consist of two storey unit structures with basement level car parking. The proposed development will require excavation of up to 3.80m depth that will generally be located approximately 2.80m to 3.30m from the west side boundary, 3.20m to 4.20m from the east side boundary and 5.10m from the rear north boundary with the exception of a section along the western boundary, which will extend to the common boundary for the driveway ramp.

A review of Northern Beaches Council (Warringah Councilôs LEP/DCP) identified that the property is located within óClass Aô landslip hazard zone (LSR_009). The proposed works involve excavation >2.00m depth and therefore will require a full geotechnical report as part of the Development Application (DA) process.

The report therefore includes a description of site and sub-surface conditions, a geotechnical assessment of the development, site mapping/plan, a geological section, site risk assessment in accordance with AGS March 2007 publication and provides recommendations for construction and stormwater disposal.

The investigation and reporting were undertaken as per the Tender P17-118, Dated: 5th April 2017.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) Drilling of three boreholes using a mini drill rig along with Dynamic Cone Penetrometer testing (DCP) to investigate the subsurface geology, depth to bedrock and identification of ground water.

The following plans and drawings were supplied for the work:

- Site survey plan by Bee and Lethbridge Pty Ltd, Plan Reference No. 20310, Sheet 1 ï 4, Dated: 30th January 2017.
- DA Plans by Barry Rush and Associates Pty Ltd, Job No. 1704, Drawing No. A00 ï A17, Dated: 18th January 2019, provided to CGC on 23rd January 2019.

2. SITE FEATURES:

2.1. Description:

The site consists of a rear battle axe style block situated within gently west sloping topography near the western edge of Collaroy Plateau with a natural drainage path passing through the site to the north-west.

The property is located on the low north side of Rose Avenue. The main portion of the property is accessed via a narrow corridor and is 60.96m from Rose Avenue with a front south and rear north boundary of 20.12m each, east side boundary of 64.10m and west side boundary of 61.06m as referenced from the provided survey plan. It contains a dilapidated single storey brick dwelling adjacent to a garage. The rear portion of the property contains dense vegetation surrounded by large trees

2.2. Geology:

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

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint pattern, many cliff areas are undercut by differential weathering. Slopes below these cliffs are often steep 15° to 23° with a moderately thick sandy colluvial soil profile that are randomly covered by sandstone boulders.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 27th and 28th April 2017 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of vegetation and existing structures. It also included the drilling of three auger boreholes (BH1 ï BH3) using a ôDingoô mini drill rig employing solid stem, spiral flight augers to investigate sub-surface geology.

Dynamic Cone Penetrometer (DCP) testing was carried out from ground surface adjacent to the borehole, through the base of the borehole and at one other location in accordance with AS1289.6.3.3 ï 1997, ñDetermination of the penetration resistance of a soil ï Perth Sand Penetrometerò to estimate near surface soil conditions.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed bore log and DCP sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

3.2. Field Observations:

The site is located on the low northern side of Rose Avenue within gently (-1° to -4°) north-west dipping topography. Rose Avenue contains a bitumen pavement with low concrete kerbs and gutters and ais gently (-5°) west sloping where it passes on the south side of the site. A grass road reserve lies between the bitumen road and the site.

The property at No. 44 Rose Avenue consists of a rear battle axe style block accessed from Rose Avenue. Access to the main portion of the block is provided by a narrow grassy corridor. At the south east corner of the main portion of the block, there is a brick and clad garage which appears to be in reasonable condition, with no signs of cracking or settlement on its external walls. Adjacent to the garage, there is a single storey brick dwelling. The building structure appears to be approximately 40 years old and is dilapidated, however it has only minor (<2mm wide crack) vertical cracks on its external walls.
At the rear of the property, there is a spacious backyard scattered with tall trees and dense vegetation along the east and west side boundaries.

The neighbouring properties to the east of the site block include No. 42, No. 40 and No. 40a Rose Avenue.

No. 42 Rose Avenue is adjacent to the siteçs corridor leading to the main portion of the site. The property contains a two storey brick dwelling at the front of the property with a swimming pool at its rear north western corner. The building structure appears to be approximately 20 years old and is in good condition, with no significant signs of cracking or settlement on its external walls. The property is at a similar ground level to the site along the common boundary, with the building located approximately 0.90m from the corridor boundary and 20.00m from the boundary of the main portion of the site.

No. 40 and No. 40a Rose Avenue are both rear battle axe style blocks located upslope to the east and are accessed via a concrete driveway that extends along part of the east boundary with the site. They contain two and three storey brick and brick rendered dwellings. The building structures appear to be approximately 20 years old and are in good condition, with no significant signs of cracking or settlement on external walls. The properties are at similar ground levels to the site along the common boundary, with the building at No. 40 located 3.50m from the common boundary and the building at No. 40a located 1.50m from the common boundary.

The neighbouring properties to the west of the site include No. 46, and No. 46a Rose Avenue.

No. 46 and No. 46a Rose Avenue are both rear battle axe style blocks containing one and two storey brick dwellings with carports at the rear portion of the properties. The building structures appear to be approximately 40 years old and are in good condition, with no significant signs of cracking or settlement on their external walls. The properties are at a similar ground level along the common boundary with the building at No. 46a located 7.50m from the common boundary and the building at No. 46 located 3.30m from the common boundary.

The neighbouring property to the north (No. 43 Lantana Avenue) consists of a rear battle style block accessed from Lantana Avenue. Access to the main portion of the block is provided by a narrow grass corridor. At the end of the corridor, there is a single storey brick dwelling which appears to be approximately 50 years old and is in good condition, with no significant signs of cracking or settlement on its external walls. At the rear of the property, there is a spacious backyard with dense vegetation at the south east corner of the property.

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

3.3. Field Testing:

The boreholes (BH1 ï BH3) were drilled at various locations within the rear portions of the property No. 44 Rose Avenue. The ôDingoô mini drill rig refusal was encountered at depths varying from 1.40m to 1.55m below the existing ground surface on interpreted low to medium strength sandstone bedrock.

Dynamic Cone Penetrometer (DCP) tests were carried out from ground surface adjacent to the boreholes and at one isolated location.

Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified as follows:

- **SAND (FILL/TOPSOIL) –** this layer was encountered from surface at all test locations to a maximum depth of 0.40m below the existing ground surface. It is classified as very loose to medium dense, fine to medium grained, moist with some concrete gravels;
- **SAND –** this layer was encountered below the fill/topsoil at all test locations to a maximum depth of 1.55m. It is classified as loose to dense, medium to coarse grained, moist to wet with some clay, quartz and sandstone gravels;
- **SANDSTONE BEDROCK ï** this layer was encountered below the sandy soils. Based on the results of DCP testing and refusal of the drill rig, the depth to the sandstone bedrock of a minimum of low strength was interpreted to vary from 1.55m depth (BH1) towards the southern portion to 1.40m depth (BH3) towards the northern portion of the site.

Details of the observed significant water seepage within various boreholes are tabulated below:

Table: 2 – Details of the Observed Significant Water Seepage

Borehole No.	Surface Level (m)	Depth Drilled (m)	Seepage Depth (mbgl*)/RL	Remarks
BH1	68.00	1.55	0.60/6.40	Observed during drilling
BH2	67.07	1.40	0.85/6.22	Observed during drilling
BH3	66.60	1.40	1.05/65.55	Observed during drilling

mbgl* - meter below ground level

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of sandy topsoil/fill of shallow thickness (Ò0.40m thick) across the site, underlain by residual sandy soils to a maximum depth of 1.55m below the existing ground surface. Residual soils overlie sandstone bedrock of at least low strength. Based on DCP testing in the areas of soil cover, the depth to sandstone bedrock of a minimum of very low strength is inferred to be varying from 1.255m to 1.50m within the area of the proposed development. This very low strength bedrock is expected to grade very quickly to medium to high strength bedrock though actual bedrock strengths are unconfirmed.

Sandstone bedrock outcrops were not observed at any location within the project site or within the neighbouring properties.

The proposed works involve demolition of the existing site structures and construction of a two storey residential unit structure with basement level car parking. The proposed development will require excavation of up to 3.80m depth that will generally be located approximately 2.80m to 3.30m from the west side boundary, 3.20m to 4.20m from the east side boundary and 5.10m from the rear north boundary with the exception of a section along the western boundary, which will extend to the common boundary for the driveway ramp.

It is expected that most of the excavation will extend through sandstone bedrock of low to medium and potentially high strength along with intersecting sandy residual soils and shallow (Ò0.40m thick) layer of topsoil/fill from surface. The excavation of medium to high strength bedrock will require the use of rock excavation equipment which has the potential to create significant ground vibrations, but the probability of vibration damage to the neighbouring houses is reduced due to the nature of the geology and the separation distances. However care will need to be taken to ensure that the excavation works do not create a vibration hazard for the neighbouring properties.

Considering the depth of the proposed excavations, geology of the site and separation distances from site boundaries, safe batter slopes of 1.0H:1.0V (fill/residual soils) are achievable for most parts of the project site. However a section along the western boundary, which will extend to the common boundary will need to be supported before excavation. The site conditions suggest that a pier support wall will be required to support the excavation perimeter to minimise lateral movements in the wall and adjoining property. A contiguous wall may be most suitable as the excavation will encounter seepage and sandy soils which will not stand unsupported. This wall can be installed via grout injection or CFA piers with liners. Temporary anchors may be required until permanent support measures are in place, although anchored support is expected to extend beyond site boundaries and this may result in access complications. It is recommended that installation of all support measures must be carried out under supervision of a trained geotechnical professional.

The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser strength than interpreted. For confirmation of bedrock strength to below proposed excavation levels will need an investigation utilizing cored boreholes in the actual excavation location. Bedrock strength through the excavation and at footing level can be confirmed by geotechnical inspection during initial excavation/construction works.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and a mini drill rig. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However the results of the investigation provide a reasonable basis for the analysis and subsequent design of the proposed works.

4.2. Site Specific Risk Assessment:

Based on our site investigation we have identified the following geological/geotechnical landslip hazards which need to be considered in relation to the existing site and the proposed works. The hazards are:

A. Landslip of surficial soils from excavation works.
B. Toppling/sliding of unstable block of rock formed by intersecting defects.

The hazards have 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.

The Risk to Life from both Hazards was estimated to vary from **1.07 x 10^{-5} to 2.08 x 10^{-6} for persons working within the excavation, whilst the Risk to Property was considered to be 'Low to Very Low'**. The hazard was therefore considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

4.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class ôAô due to the sandy nature of the soils and when footings are founded on bedrock at base of excavation
Type of Footing	Strip/Pad or Slab at base of excavation, piers external to excavation
Sub-grade material and Maximum Allowable Bearing Capacity	- Weathered, VLS Sandstone: 800kPa - Weathered LS Sandstone: 1000kPa - Weathered MS Sandstone: 2000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: - All footings should be founded off bedrock of similar strength to prevent differential settlement. - All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project. - Piers should be socketed at least one diameter into weathered bedrock to provide stability within the slope.	

4.3.2. Excavation:	
Depth of Excavation	- Up to 3.80m for basement
Distance to Neighbouring Properties	<u>East Boundary</u>: No. 40 ï 3.20m to 3.40m to the common boundary, building another 3.50m away, N0. 40a ï 4.20m to the common boundary, house another 1.50m away, No. 42 ï 3.00m to the common boundary, house another 20.00m away. <u>West Boundary</u>: No. 46 ï 3.30m to the common boundary, house another 3.30m away, No. 46a ï 3.30m to the common boundary, house another 7.50m away,

	New driveway of Development ï to the common boundary. North Boundary: No 43 ï open lawn, house >40m away.
Type of Material to be Excavated	Shallow layers of loose to medium dense sandy fill/topsoil to 0.40m depth
	Loose through dense Sand to 1.40m ï 1.55m depth (Residual Soils)
	ELS to VLS bedrock (Below 1.40m to 1.55m)
	LS ï MS/HS bedrock

Guidelines for batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill and natural soils	1:1	2:1
Extremely Low to Very Low strength bedrock	1:1	1.25:1
Medium strength, defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by engineering geologist.

Remarks: Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Topsoil and residual soils	Excavator with bucket
	ELS bedrock	Excavator with bucket
	VLS bedrock/ironstone	Excavator with bucket and ripper
	LS ï MS bedrock	Rock hammer and saw

VLS ï very low strength, LS ï low strength, MS ï medium strength, HS ï high strength

Remarks: It is recommended that the hard rock excavation perimeter be saw cut prior to rock hammering, this will generally reduce the amount of rock support required, reduce deflection of rock across boundary and under neighbouring structures and will provide a slight buffer distance to ground vibrations for the use of rock hammers.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength sandstone bedrock, to achieve the specified low level of vibration the below tabulated hammer weights and buffer distances are required:

Buffer Distance from Structure	Maximum Hammer Weight
2.0m	200kg
4.0m	500kg
5.0m	800kg
8.0m	1000kg

Onsite calibration will provide accurate vibration levels for the site specific conditions and will generally allow for larger excavation machinery or smaller buffer distances to be used. Calibration of rock excavation machinery will need to be carried out prior to commencement of bulk rock excavation works and will determine the need for full time monitoring.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	No. 37, No. 40, No. 40a, No. 41, No. 46 and No. 46a = 5mm/s
Vibration Calibration Tests Required	Recommended
Full time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • During installation of support measures • Following clearing of bedrock surface • Every 1.50m depth interval of the main excavation where unsupported • Where low to medium strength bedrock is exposed • At completion of the excavation.
Dilapidation Surveys Requirement	On neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.

Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.

4.3.3. Retaining Structures:

Required	Surficial soils above and near excavation crests be retained prior to bulk excavation to minimize the risk of a soil slide into the excavation. This can be achieved by either clearing the soils away from the excavation crests (batters), horizontal benches or by construction of a temporary and/or permanent retaining structure founded off the bedrock surface.
Types	Soldier piles and/or steel reinforced concrete/concrete block wall designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.

Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill (sandy) (loose)	18	$\phi' = 29°$	0.35	0.52	N/A
ELS bedrock	22	$\phi' = 38°$	0.15	0.20	400 kPa
LS bedrock	23	$\phi' = 40°$	0.10	0.15	600kPa

Remarks: In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

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. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

Grout/rock adhesion for temporary anchors	350kPa for LS rock

Remarks: Anchors should be installed at a minimum distance of 0.20m from excavation crest

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Significant water seepage encountered in four boreholes
Excavation likely to intersect	Water Table	No
	Seepage	Significant seepage up to 1.05m depth within sand layer. Minor (<0.50L/min), on defects and at soil/rock interface.
Site Location and Topography		On ridge in gentle topography, low northern side of the road
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not recommended
Remarks: Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow. This can result in relaxation of excavation faces causing instability. Therefore excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional. A stormwater diversion drain should be installed upslope of excavation crests to intercept stormwater runoff and prevent erosion and softening of the excavation faces. An excavation trench should also be installed at the base of excavation cuts to below floor slab levels to reduce the risk of long term dampness. 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 the Councilôs stormwater system off site.		

4.4. Conditions Relating to Design and Construction Monitoring:

To allow certification at the completion of the project it will be necessary for Crozier Geotechnical Consultants to:

1. Review and approve the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Supervise installation of support measures,
3. Inspect any exposed low to medium strength bedrock and the proposed excavation equipment prior to its use
4. Inspect excavation at 1.50m depth intervals
5. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,

6. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilization and drainage measures are in place.

Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

5. CONCLUSION:

The site investigation identified the presence of sandy topsoil/fill of shallow thickness (Ò0.40m thick) across the site, underlain by residual sandy soils to a maximum depth of 1.55m below the existing ground surface. Residual soils overlie sandstone bedrock of at least low strength. Based on DCP testing in the areas of soil cover, the depth to sandstone bedrock of a minimum of very low strength is inferred to be varying from 1.25m to 1.50m within area of the proposed development. This very low strength bedrock is expected to grade very quickly to medium to high strength bedrock though actual bedrock strengths are unconfirmed.

The proposed works involve construction of a two storey residential unit structures with basement level car parking. The proposed development will require excavation of up to 3.80m depth that will generally be located approximately 2.80m to 3.30m from the west side boundary, 3.20m to 4.20m from the east side boundary 5.10m from the rear north boundary with the exception of a section along the western boundary, which will extend to the common boundary for the driveway ramp.

As per investigation results most of the excavation will extend through sandstone bedrock of low to medium and potentially high strength along with intersecting sandy residual soils and shallow layer of topsoil/fill from surface. In view of the depth of the proposed excavations, geology of the site and separation distances from site boundaries, safe batter slopes of 1.0H:1.0V (fill/residual soils) are achievable for most parts of the project site. However a section (basement parking ramp) along the western boundary, which will extend to the common boundary, will need to be supported before excavation. The site conditions suggest that a contiguous pier wall with temporary anchors for lateral support may be the most suitable method of support. It is recommended that installation of all support measures must be carried out under supervision of a trained geotechnical professional.

The lower half of the excavation may intersect medium to high strength sandstone bedrock which will require rock excavation equipment (i.e. rock hammers). Where this occurs a geotechnical professional should be consulted to assess the bedrock strength and provide guidance on suitable equipment.

The risk from geological/geotechnical hazard which was identified in relation to the proposed works, is limited to minor landslip failure of surficial soils along a small section of the west side boundary is considered óAcceptableô, when assessed against the criteria of the AGS 2007.

The risks associated with the proposed development can be maintained within óAcceptableô levels with negligible impact to neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

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6. REFERENCES:

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Appendix 1

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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: -

Qc (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: -

Qc = (12 to 18) Cu

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

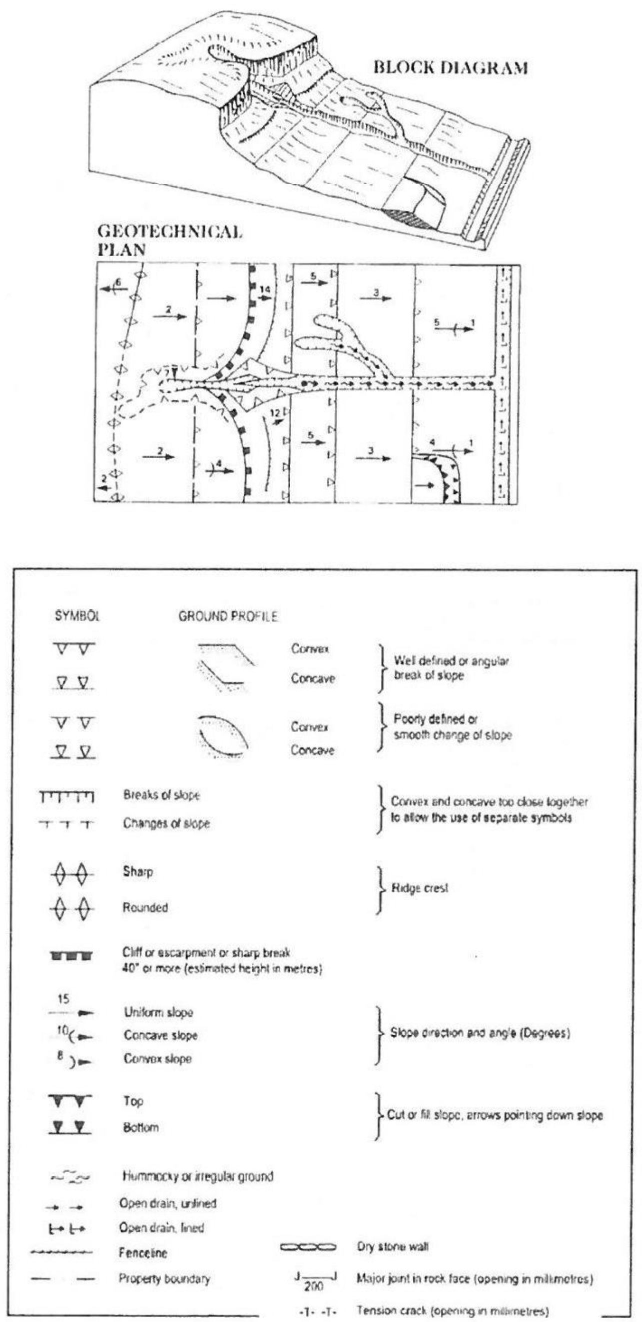
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

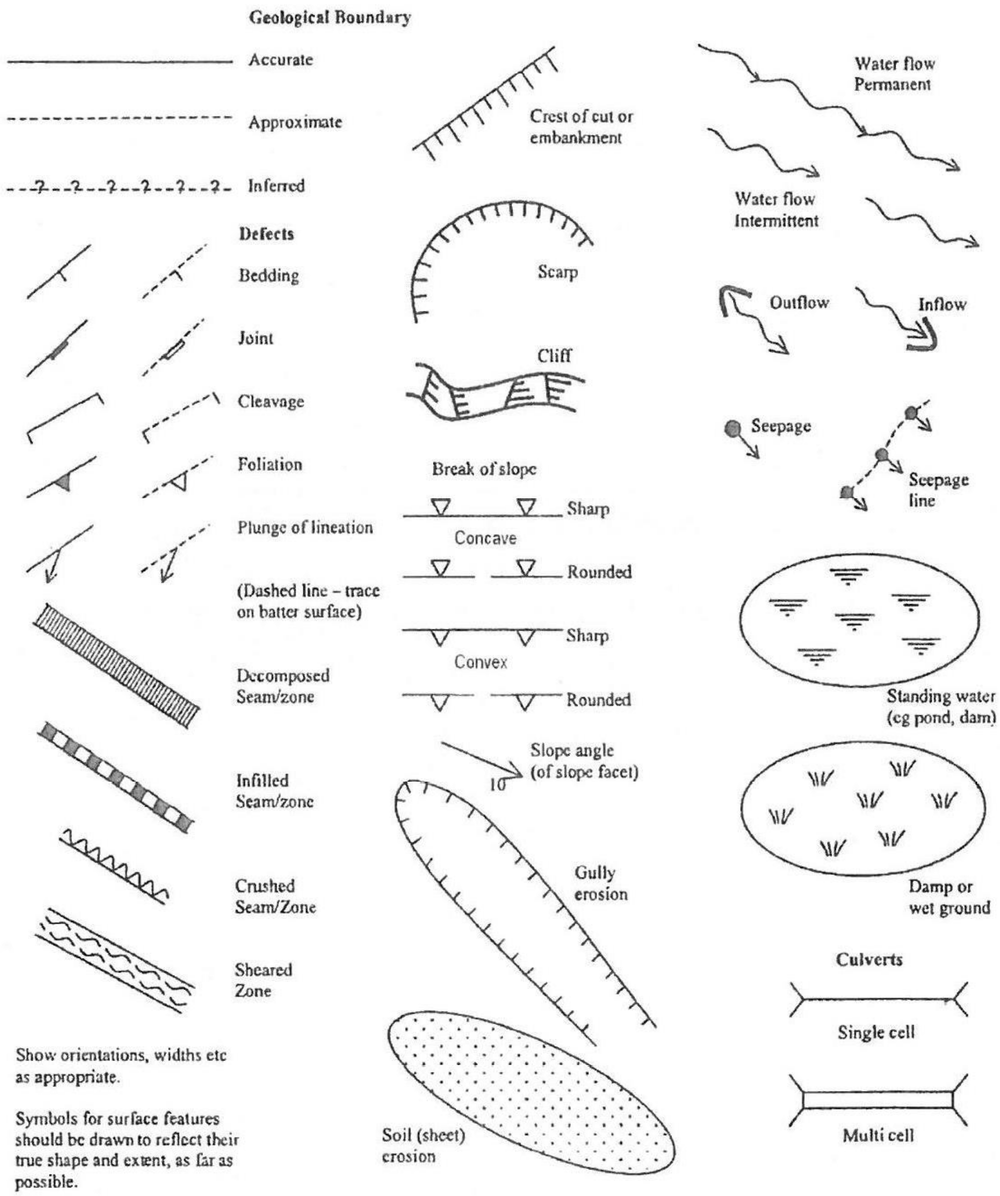
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.



Example of Mapping Symbols

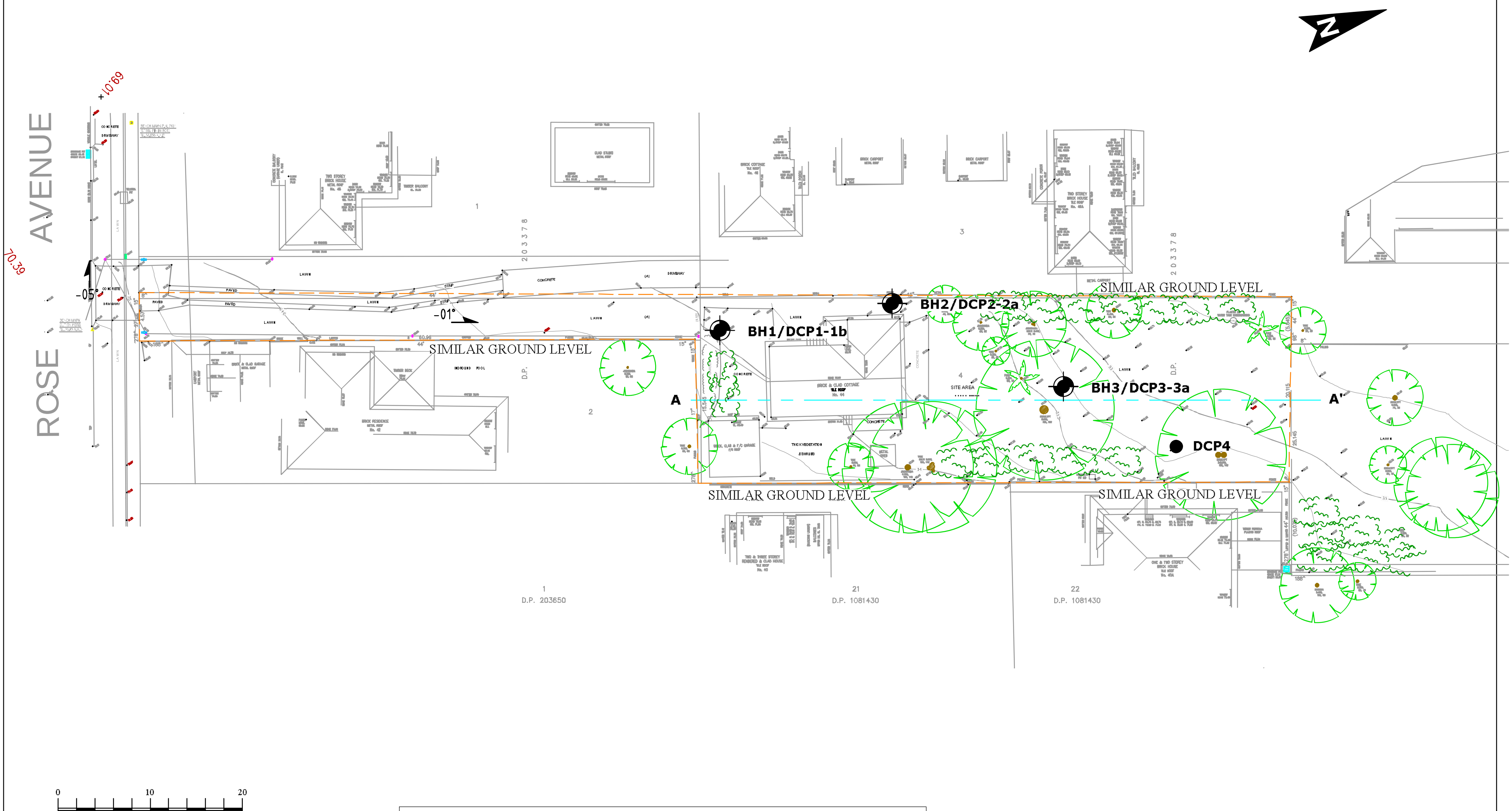
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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



VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

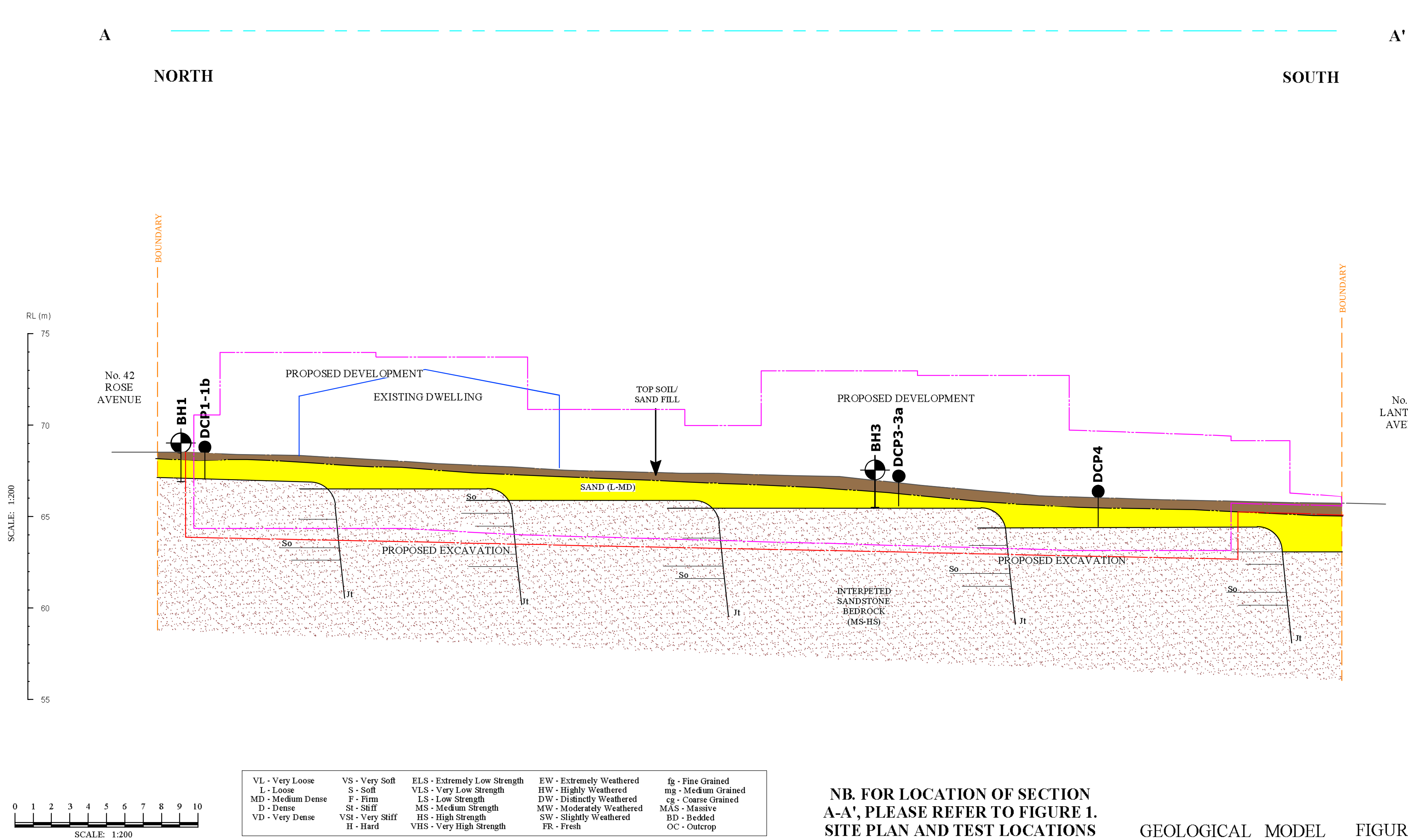
LEGEND

- DCP — DYNAMIC CONE PENETROMETER
- BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
- DENSE VEGETATION
- x° SLOPE ANGLE
- A — A' CROSS-SECTION REFERENCE LINE
- PROPERTY BOUNDARY

SCALE:	1:400 @ A3
DRAWING:	FIGURE 1
DATE:	30/01/2019
APPROVED BY:	TMC
DRAWN BY:	KB
PROJECT:	2017-083.1

PREPARED FOR:
Wheeler Heights Developments Pty Ltd

ADDRESS:
44 ROSE AVENUE
WHEELER HEIGHTS



VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

ELS - Extremely Low Strength
VLS - Very Low Strength
LS - Low Strength
MS - Medium Strength
HS - High Strength
VHS - Very High Strength

EW - Extremely Weathered
HW - Highly Weathered
DW - Distinctly Weathered
MW - Moderately Weathered
SW - Slightly Weathered
FR - Fresh

fg - Fine Grained
mg - Medium Grained
cg - Coarse Grained
MAS - Massive
BD - Bedded
OC - Outcrop

NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 2.

LEGEND

Symbol	Description	Symbol	Description	Symbol	Description	Symbol	Description
BH	AUGER LOCATIONS		GEOLOGICAL BOUNDARY	So	BEDDING		TOP SOIL/ SAND FILL
		A — A'	CROSS-SECTION REFERENCE LINE				SAND
DCP	DYNAMIC CONE PENETROMETER		PROPERTY BOUNDARY	Jt	JOINT		SANDSTONE BEDROCK

CROZIER GEOTECHNICAL CONSULTANTS
Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 30/01/2019
APPROVED BY: TMC
DRAWN BY: KB
PROJECT: 2017-083.1

PREPARED FOR:
Wheeler Heights Developments Pty Ltd

ADDRESS:
44 ROSE AVENUE
WHEELER HEIGHTS

TEST BORE REPORT

CLIENT: Wheeler Heights Developments Pty Ltd **DATE:** 27/04/2017 **BORE No.:** 1

PROJECT: New Residential Development **PROJECT No.:** 2017-083.1 **SHEET:** 1 of 1

LOCATION: 44 Rose Ave, Wheeler Heights **SURFACE LEVEL:** RL å 68.00m

Depth (m)	Description of Strata	Sampling		In Situ Testing	
	PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Type	Depth (m)	Type	Results
0.00	GRASS SAND (Fill) - Medium dense, dark grey, medium grained, moist sand fill with some concrete fragments				
0.20	SAND - Medium dense, grey, medium grained, moist sand * 0.45m loose				
		D	0.50		
	* 0.80m light grey and wet				
1.00		D	1.00		
	* 1.20m medium dense				
		D	1.50		
1.55	DINGO REFUSAL at 1.55m depth on interpreted low strength sandstone bedrock				
2.00					

RIG: Dingo Mini Drill Rig DRILLER: KB LOGGED: BL

METHOD: Spiral Flight Auger with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: Significant seepage at 0.60m depth

REMARKS: CHECKED: SR

TEST BORE REPORT

CLIENT: Wheeler Heights Developments Pty Ltd | **DATE:** 27/04/2017 | **BORE No.:** 2

PROJECT: New Residential Development | **PROJECT No.:** 2017-083.1 | **SHEET:** 1 of 1

LOCATION: 44 Rose Ave, Wheeler Heights | **SURFACE LEVEL:** RL å 67.07m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	Results	
0.00	GRASS SAND (Fill) - Very loose, dark brown, fine to medium grained, moist sand fill					
0.40	SAND - Loose, brown-grey, medium grained, wet sand * 0.50m grey	D	0.50			
	* 0.70m light grey with some clay					
1.00		D	1.00			
	* 1.10m coarse grained with some quartz gravels					
1.40	DINGO REFUSAL at 1.40m depth on interpreted low strength sandstone bedrock	D	1.40			
2.00						

RIG: Dingo Mini Drill Rig | DRILLER: KB | LOGGED: BL

METHOD: Spiral Flight Auger with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: Significant seepage at 0.85m depth

REMARKS: | CHECKED: SR

TEST BORE REPORT

CLIENT: Wheeler Heights Developments Pty Ltd
DATE: 27/04/2017
BORE No.: 3
PROJECT: New Residential Development
PROJECT No.: 2017-083.1
SHEET: 1 of 1
LOCATION: 44 Rose Ave, Wheeler Heights
SURFACE LEVEL: RL å 66.60m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling Type	Sampling Depth (m)	In Situ Testing Type	Results	
0.00	GRASS					
	SAND (Topsoil) - Very loose, dark brown, fine grained, moist sand topsoil					
	* 0.15m loose					
0.40	SAND - Medium dense, brown-grey, medium grained, wet sand					
		D	0.50			
	* 0.60m grey with some quartz gravels					
	* 0.90m some clay					
1.00		D	1.00			
	* 1.30m light grey, fine grained and moist					
1.40	DINGO REFUSAL at 1.40m depth on interpreted low strength sandstone bedrock	D	1.40			
2.00						

RIG: Dingo Mini Drill Rig
DRILLER: KB
LOGGED: BL
METHOD: Spiral Flight Auger with Tungsten Carbide Bit
GROUND WATER OBSERVATIONS: Significant seepage at 1.05m depth
REMARKS:
CHECKED: SR

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Wheeler Heights Developments Pty Ltd **DATE:** 27/04/2017

PROJECT: New Residential Development **PROJECT No.:** 2017-083.1

LOCATION: 44 Rose Ave, Wheeler Heights **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP1a	DCP2	DCP2a	DCP3	DCP3a	DCP4	
0.00 - 0.15	6	-	1	-	1	-	1	
0.15 - 0.30	6	-	1	-	2	-	2	
0.30 - 0.45	4	-	2	-	2	-	2	
0.45 - 0.60	2	-	2	-	3	-	4	
0.60 - 0.75	2	-	2	-	3	-	6	
0.75 - 0.90	2	-	1	-	3	-	7	
0.90 - 1.05	3	-	2	-	4	-	7	
1.05 - 1.20	3	-	3	-	9	-	13	
1.20 - 1.35		4		10 (B)		16	18	
1.35 - 1.50		25		Refusal at 1.25m on rock		10 (B)	20	
1.50 - 1.65		Disct at 1.40m on interpreted rock				Refusal at 1.35m on rock	10 (B)	
1.65 - 1.80							Refusal at 1.50m on rock	
1.80 - 1.95								
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								

TEST METHOD: AS 1289. F3.3, PERTH SAND PENETROMETER

REMARKS: **(B)** Test hammer bouncing upon refusal on solid object

-- No test undertaken at this level due to prior excavation of soils

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood	Spatial Impact	Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <$3m^3$) of surficial soils from excavation works	Excavation area	3.80m deep excavation, loose to medium dense fill and residual sandy soils to 1.55m depth overlying sandstone bedrock with significant seepage at 0.60m depth Possible 0.001	Excavation at the west side boundary, approximately 30m long section of new driveway/ramp of the southern development 0.10	Person working in excavtion 8hrs/day 0.33	Unlikely to not evacuate 0.25	Person in open space, unlikely buried 0.25	**2.08E-06**
B	Toppling/sliding of unstable block of rock formed by intersecting defects, from unsupported cliff face	Excavation area	Ground vibrations created by using a larger rock hammer (>250kg) may cause toppling/sliding of unstable block of rock from cut face Possible 0.001	Individual block may hit small portion of excavtion 0.10	Workers within the excavation approx. 8hrs/day, Mon - Sat 0.29	Likely to not evacuate 0.75	Person possible to be crushed 0.50	**1.07E-05**

* hazards considered in current condition and/or without suitable remedial/stabilisation measures
* likelihood of occurrence for design life of house (considered 100years)
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01)
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <3m3) of surficial soils from excavation works	Excavation area	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Toppling/sliding of unstable block of rock formed by intersecting defects, from unsupported cliff face	Excavation area	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* 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%.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 4

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