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

GEOTECHNICAL INVESTIGATION REPORT

for

PROPOSED NEW DWELLING

at

16 REDDALL STREET, MANLY, NSW

Prepared For

Mike Norman

Project No.: 2024-166

September 2024

Document Revision Record

Issue No	Date	Details of Revisions
0	27th September 2024	Original Issue

TABLE OF CONTENTS

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

GEOTECHNICAL INVESTIGATION FOR PROPOSED NEW DWELLING
16 REDDALL STREET, MANLY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation and assessment carried out for a proposed new dwelling at 16 Reddall Street, Manly, NSW. The assessment was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of The Quinlan Group on behalf of the client Mike Norman.

It is understood that the proposed works involve the demolition of existing site structures and the construction of a new three-storey dwelling featuring a Basement level broadly towards the middle of the site and a pool and cabana towards the site rear. The Basement finished floor level (FFL) is proposed to be at RL24.65, hence requiring bulk excavation below existing ground surface levels to a maximum depth of 2.70m, gradually reducing to nil at the front eastern boundary. The Basement Floor level is proposed to abut the side southern boundary, whilst setback from the northern and western boundary by 1.25m and 19.0m respectively. The proposed pool is estimated to require bulk excavation to 2.00m depth, abutting the side northern boundary.

With reference to Northern Beaches (Manly) Councils – Development Control Plan 2013 – Schedule 1 Map C, the site is located within Landslip Risk Class ‘G4’ which is classified as Ridge crests, major spur slopes and dissected plateau areas with typical slope angles of <15°.

The site is classified under Manly Local Environment Plan 2013 as being within ‘Class 5’ Acid Sulfate soils hazard zone therefore an assessment of Acid Sulphate Soils (ASS) is also required to support the DA.

The report provides an assessment of the site conditions and the proposed development with recommendations for the design and construction to ensure geotechnical stability and good engineering practice.

The investigation and reporting were undertaken as per the Fee Proposal No.: P24-386, Dated: 19 August 2024.

The investigation comprised:

a) Before You Dig Australia (BYDA) request and onsite review, along with test location clearing by an accredited service locating sub-contractor.
b) A detailed geotechnical inspection and mapping of the site and inspection of adjacent properties
c) Drilling of four auger boreholes to identify sub-surface geology using hand tools due to site access limitations along with Dynamic Penetrometer (DCP) to investigate sub-surface conditions.
d) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional.

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

- Architectural Drawings – The Quinlan Group, Drawing No.: NRM100/A, NRM101/A, NRM102/A, NRM103/A, NRM800/A, Dated: 13/08/2024.
- Survey Drawing – Hill & Blume, Drawing No.: 65463001A, Sheet 1 of 1, Dated: 30/05/2024.

2. SITE FEATURES:

2.1. Description:

The site (16 Reddall Street, Manly) (Lot 1, DP68066) is a rectangular shaped block located on the south-western side of Reddall Street within gentle north-east dipping topography at mid slope. The survey indicates ground surface levels within the site varies from a high of RL27.54 towards the western rear to a low of RL25.57 along the front eastern boundary. For the purposes of this report, the front Reddall Street roadway is referenced as the eastern boundary, with the other boundaries referenced accordingly.

From the provided survey plan, the site has front eastern boundary and rear western boundary of approximately 7.695m respectively, whilst the side northern and southern boundaries are approximately 48.785m and 48.055m respectively, covering a total site area of 375.7m^2. An aerial photograph of the site and its surrounds with boundary designations is shown below in Photograph 1, as sourced from NSW Government Six Map Spatial Data.

Photograph 1: Aerial site view and surrounds (NSW Government Six Map Spatial Data)

3.2. 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 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 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 are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defect 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 from the Sydney 1:100,000 Geological Series sheet 9130 provided below indicates the geology underlying the site and surrounding area.

Extract of Sydney (9130 Geology Series Map): 1:100,000 – Geology underlying the site

3. FIELDWORK:

3.1. Methods:

The field investigation comprised a geotechnical inspection and mapping of the site and adjacent properties on 4th September 2024 by a Geotechnical Engineer. It included a photographic record of the site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of ground conditions, existing site structures and limited inspection of neighbouring properties and structures.

The sub-surface investigation comprised the drilling of four auger boreholes (BH1 – BH4) throughout the site to investigate the sub-surface geology using hand tools as access to the required test locations within the site for a conventional drilling rig was unavailable, access to test down the majority of the south boundary was also not available.

Geotechnical logging of the subsurface conditions was undertaken by a Geotechnical Engineer by inspection of disturbed soil recovered from the augers. Logging was undertaken in accordance with AS1726:2017 'Geotechnical Site Investigations'.

Dynamic Cone Penetrometer (DCP) testing was carried out from the ground surface adjacent to the boreholes in general accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer" to estimate near surface soil conditions and depth to bedrock.

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

3.2. Field Observations:

The site is situated on the south-western side of Reddall Street within gentle north-east dipping topography at mid slope level. Reddall Street is formed with a bituminous sealed pavement that is gently south dipping and is separated from the site by a concrete kerb, pedestrian pathway and a nature strip comprising grass lawns. There were no signs of excessive settlement or cracking within the roadway to indicate any large-scale slope stability concern. Photograph 2 below provides a view of the site from Reddall Street.

Photograph 2: View of the front of the site, facing broadly west from Reddall Street (Retrieved from Google Street View)

The site contains a single storey masonry house formed atop a sandstone block base of anticipated construction age >80 years situated broadly towards the middle of the site. A rendered garage situated at roadway level is attached to the front of the house accessed via a concrete strip driveway between supported garden beds. A concrete pathway provides access to the house along the southern side of the garage and a concrete and bitumen pathway provides access to the site rear along the northern side of the garage/house. The remainder of the front contains retained garden beds of ≤0.50m height supporting vegetation.

A secondary single storey, clad granny flat is situated at the site rear separated from the house by a timber fence and a grass lawn. The main structures appeared in good condition with no obvious signs of significant cracking or excessive settlement to indicate any underlying geotechnical concern whilst the secondary granny flat was not visible through the timber fence.

The neighbouring property to the north (No. 14 Reddall Street) contains a three storey, rendered apartment building of anticipated construction >60 years, and setback from the shared site boundary by 2.20m. The property rear contains a brick paved courtyard/carport area. A timber boundary fence and rendered boundary wall separates the site with the neighbouring property. The property appears to contain similar ground surface levels to the site along the common boundary and the neighbouring apartment building appeared in good condition with no signs of cracking, ground movement or underlying geotechnical issues.

The neighbouring property to the south (No. 18 Reddall Street) contains a three-storey, rendered dwelling including a basement level of anticipated construction age >20 years, setback from the shared site boundary by 0.85m. The property rear appears to contain a swimming pool with surrounding tiled area estimated to be setback by 2.00m from the shared site boundary, with a grass lawn between the pool and the house. The property appears to contain lower ground surface levels to the site by up to 1.50m height. The visible features of the main structure appeared in good condition with no signs of cracking, ground movement or underlying geotechnical issues.

The neighbouring property to the west (No. 22 Cliff Street) contains two, masonry apartment complexes of anticipated construction age >50 years, with the rear building setback from the shared site boundary by an estimated 2.00m. The property appears to contain similar ground surface levels to the site along the common boundary, however the structures were not visible due to the boundary obstruction.

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

3.3. Ground Conditions:

Four boreholes (BH1 - BH4) were drilled using a hand auger across the site, broadly within the envelope of proposed works. A shallow layer of topsoil/fill was encountered from ground surface with extremely weathered bedrock encountered underlying the fill prior to hand auger refusal on interpreted sandstone bedrock of at least very low strength.

Dynamic Cone Penetrometer (DCP) tests were undertaken from the ground surface at four locations adjacent to the boreholes. Refusal on interpreted sandstone bedrock of at least very low strength was encountered between 0.50m (DCP4) and 0.75m depth (DCP1 and DCP3).

For a description of the ground conditions encountered at the individual borehole/DCP test locations, the Borehole Log and DCP results sheets should be consulted however the subsurface conditions at the site can be summarized as follows:

- **TOPSOIL/FILL –** This material was encountered within all boreholes from ground surface or underlying a bitumen pavement and extends to a maximum depth of 0.50m (BH3). It generally comprised a brown to grey, fine to medium grained, dry sandy fill with rootlets and gravels.
- **SAND/CLAYEY SAND** – Natural sand/clayey sand was encountered within all test locations from a minimum depth of 0.10m (BH1). The unit generally comprised a very dense, pale brown, medium grained, dry to moist sand with sandstone gravels towards the front of the site, whilst the site rear comprised a very dense, pale brown, medium grained, dry to moist clayey sand.
- **SANDSTONE BEDROCK** – Interpreted sandstone bedrock of at least very low strength was encountered in all test locations via borehole and DCP refusal from a minimum and maximum depth of 0.50m (DCP4) and 0.75m (DCP1 and DCP3).

No signs of a free-standing groundwater table or significant seepage were identified within any of the boreholes or on any returned DCP rods to approximately RL 24.82.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of topsoil/fill from ground surface or underlying the bitumen pavement to a maximum depth of 0.50m overlying natural sand/clayey sand from a minimum depth of 0.10m. This layer was underlain by interpreted sandstone bedrock of at least very low strength from a minimum and maximum depth of 0.50m (DCP4) and 0.75m (DCP1 and DCP3) respectively with the bedrock anticipated, though not confirmed to grade to low to medium strength quickly with intersection. However, shale bedrock and weak sandstone units have been encountered in nearby properties. A free-standing ground water table or significant seepage was not observed within the boreholes or on the DCP rods on retrieval and is not expected within shallow depths (≤3.0m) based on site location.

The proposed works involve the demolition of existing site structures and the construction of a new three storey dwelling including a basement level. The proposed works are anticipated to require maximum bulk excavation to 2.70m depth for the proposed basement level, gradually reducing to nil at the front eastern boundary. The Basement Floor level is proposed to abut the side southern boundary, whilst setback from the northern and western boundary by 1.25m and 19.0m respectively are proposed. A swimming pool and cabana structure is proposed at the site rear.

Based on the results of the investigation, the Basement excavation is anticipated to extend through a shallow layer of fill and natural sand prior to intersection of sandstone bedrock for the majority of the excavation. The bedrock is anticipated to be of at least very low strength however is anticipated to grade to low to medium strength and potentially high strength with depth.

Based on the proposed excavation depths and setbacks as well as the depths of the surficial soils overlying the bedrock across site, it is expected that safe batter slopes with respect to neighbouring boundaries, as detailed in Section 5.3 of this report may not be feasible along the house’s southern excavation edge and the proposed swimming pools northern excavation edge.

Pre-excavation support will be required where safe batter slopes are unachievable and may be considered where poor-quality bedrock is encountered. Pre-excavation support may consist of soldier pile (or similar) shoring walls. Pre-excavation support will need to be installed and taken through any soils and founded within competent bedrock or below the base levels of excavations. Careful control of pile drilling/support installation is required to avoid over excavation whilst all gaps in the wall must be sealed during excavation to prevent erosion between piles where supporting soil.

The actual strength and quality of the bedrock in the proposed excavation location is unconfirmed and requires core drilling, which will better define safe batter conditions and footing bearing capacities. Bedrock of at least low to medium strength can be excavated at steep to vertical batter slopes provided it is unfractured by the excavation works and does not contain unfavorable defects. Where these are encountered then support systems (i.e. rock bolts/shotcrete) can be implemented as excavation works progress. As access for testing is not available on the southern boundary it is recommended that geotechnical inspection occur following demolition and prior bulk excavation to expose and assess the bedrock surface. Where poor quality bedrock is exposed then further investigation or a conservative support approach (ie piles) should be implemented.

There were limited stability hazards identified in the investigation. However, there is a potential for poorly oriented defects in both residual soils or weathered bedrock or localized zones of highly weathered bedrock (particularly near the upper surface) in the proposed excavation to result in localized rockslide/topple failure with potential impact to the site however there is very low potential for impact to properties adjacent. Installation of pre-excavation support systems will prevent major stability hazards.

Deflection in the excavation will occur due to destressing as a result of the high north-south in situ horizontal stresses within the Hawkesbury Sandstone bedrock. Excavation of the bedrock will result in rock mass de stressing regardless of defects and support systems installed which could result in minor damage to the existing structure where founded at the crest of new excavations. Previous excavations within this geological unit in Sydney, to the depths proposed have been identified as having lateral deflection in the order of 1.25mm - 5mm at the centre, crest of an excavation face (Wong, 2013). This deflection is expected to dissipate with distance from the excavation and will be negligible/nil at approximately 1.5D (D= depth of excavation). As such, no noticeable impact to neighbouring structures is expected.

The fill/soil and extremely weathered to very low strength bedrock can be excavated using conventional earthmoving equipment (e.g. buckets and rippers) which appears possible for the entire basement level. If excavation of low up to high strength rock is required, this will require the use of rock excavation equipment which can produce ground vibrations of a level which can potentially cause damage to neighbouring structures. Therefore, selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use. It is recommended that a rock saw and small (≤250kg) rock hammers be proposed for use at this site to avoid the need for full time monitoring. Larger rock hammers may be preferred and if utilized, further assessment and potentially full-time monitoring would be necessary.

Based on the relatively shallow depth to bedrock and expectations of bedrock at the base of the excavations, any new footings are recommended to bear within competent sandstone bedrock of similar strength to avoid differential settlement. Preliminary allowable bearing pressures appropriate for the bedrock encountered underlying the site are provided in Section 6.3.1. The confirmation of consistent low strength bedrock or better below auger refusal depths will require core drilling.

A free-standing ground water table or significant seepage was not identified within the boreholes or observed on the DCP rods on retrieval to RL 24.82. As such, the water table will not be impacted by the proposed development. Whilst seepage appears limited this could increase during/following rainfall. As the basement are excavated into the site, they will accumulate seepage which will require control/removal unless a tanked structure is proposed.

The site is situated within 'Class 5' Acid Sulfate Soils hazard zone and based on the investigation results no Acid Sulfate Soils will be intersected whilst the proposed works are not anticipated based on excavation depths to lower or impact the water table in any adjacent Class 1, 2, 3 or 4 land. As such, the Preliminary Assessment conducted as per the methods of the Acid Sulfate Soils Manual 1998 confirms that an Acid Sulfate Management Plan is not required.

It is understood that a Sydney Water (SW) asset underlies the site across the western rear of the property, within approximately 7.50m of the proposed basement excavation. CGC has not undertaken any investigation into the construction/type/depth etc. of the asset however the BYDA plans indicate it as comprising a connected 150mm diameter Vitrified Clay pipe running through the western rear. The nearest manholes within the neighbouring property (No. 14 Reddall Street) indicates the pipe invert is located at approximately 1.90m depth. Based on both the asset depth and scope of proposed works, excavation and structures which will not intersect the 45° influence zone from the sewer invert. As such, based on Sydney Water's "Technical Guidelines for Building Over and Adjacent to Pipe Assets, 2015", it is anticipated that a Specialist Engineering Assessment (SEA) will not be required for the Construction Certificate (CC) phase of the proposed works, however Sydney Water should be contacted to confirm.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing structures within the site and adjacent properties provided all 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 using hand tools. This test equipment provides limited data from small, isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations and below auger refusal depths. However, provided the recommendations of this report are implemented in the design and construction phases the proposed development is considered suitable for the site.

4.2. Site Specific Risk Assessment:

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

A. Landslip (earthslide) of surficial soil around perimeter of excavation (<2.00m^3)
B. Landslip (rockslide/topple <5m^3) of bedrock around perimeter of excavation due to poorly oriented defects

A qualitative assessment of risk to life and property related to this hazard is presented in Table A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

The **Risk to Life** from **Hazard A** was estimated to be up to **1.41x 10^{-6}** for a single person, whilst the **Risk to Property** was considered to be **'Moderate'** for neighbouring properties.

The **Risk to Life** from **Hazard B** was estimated to be up to **7.03x 10^{-7}** for a single person, whilst the **Risk to Property** was considered to be **'Low'** for neighbouring properties

Based on the results in Hazard A, The Risk to Property is unacceptable without treatment. However, these risk levels are assessed for a single catastrophic slope failure based on the existing condition and protection of the rear retaining structures. The assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including detailed investigation, regular geotechnical mapping of the excavation and installation of determined support systems in timely manner, the likelihood of any failure becomes 'Rare' and as such the consequences and risk can be further reduced when assessed against the criteria of the AGS.

4.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings on bedrock at base of excavation or off bedrock surface
Type of Footing	Strip/Pad or Slab
Sub-grade material and Maximum Allowable Bearing Capacity – shallow footings	- Very low strength sandstone: 800 kPa - Low strength sandstone: 1000 kPa - Medium strength sandstone: 2000 kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – Rock site The hazard factor (z) for Sydney is 0.08.

Remarks:

*Requires confirmation via additional investigation including core drilling

All footings should be founded off bedrock of similar strength to reduce the potential for differential settlement, allowance for differential settlement should be designed for if the structure is variably founded.

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 'certified' at the end of the project.

Individual footings should be founded within/on material of similar bearing and settlement characteristics to reduce the potential for differential settlement.

4.3.2. Excavation:	
Depth of Excavation	Up to 2.70m depth for Basement level Estimated 2.00m depth for pool

Property Separation:

The table below shows the properties potentially affected by the proposed excavation, excavation depth and the separation distances to the shared property boundary/structure. It is considered, based on elevations and setbacks, that the risks associated with the excavation are limited to the northern, southern and western boundaries.

Table 1: Property separation distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m)	Separation Distances (m)	
				Boundary	Structure
North	No. 14 Reddall Street	Apartment building	Up to 2.70m for Basement, 2.00m for Pool	1.25m for Basement, 0.00m for Pool	2.20m

South	No. 18 Reddall Street	House, Pool, Retained garden bed	Up to 2.70m for Basement, 2.00m for Pool	0.00m for Basement, 4.50m for Pool	House setback 0.85m, Pool setback 2.00m, Garden bed directly adjacent to boundary
East	Reddall Street	Pedestrian pathway, Bitumen roadway	0.00m	0.00m	Pathway setback 0.60m, Roadway setback 3.60m
West	No. 22 Cliff Street	Apartment building	2.00m for Pool	>8.00m	2.00m

Type of Material to be Excavated	≤ 0.75m	Fill/ Topsoil and Natural Sand
	From a minimum 0.50m	Sandstone bedrock – VLS – MS/HS

Guidelines for un-surcharged batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	1.5:1	2.0:1
Very low (VLS) strength, or fractured bedrock	0.75:1*	0.5:1*
Low to medium strength (LS -MS), defect free sandstone bedrock	Vertical*	Vertical*
Low to medium strength (LS -MS), defect free shale bedrock	Vertical*	0.25:1*

*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 or rock cuts and invoke the need to implement additional support measures.

Batter slopes are not achievable and so unsupported excavation for the main walls is not suitable.

Where safe batter slopes are not implemented, the stability of the excavation cannot be guaranteed until permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval.

Equipment for Excavation	Fill/natural soils	Excavator with bucket or hand tools
	VLS bedrock	Excavator with bucket and ripper
	LS to MS/HS bedrock	Rock hammer and rock saw

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

Remarks:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across

<table>
<tr><td colspan="2">boundary of detached sections of bedrock near surface.
Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Hawkesbury Sandstone bedrock, to achieve a low level of vibration (5mm/s PPV) the below hammer weights and buffer distances are generally required:
<table><tr><th>Maximum Hammer Weight</th><th>Required Buffer Distance from Structure</th></tr><tr><td>300kg</td><td>2.00m</td></tr><tr><td>400kg</td><td>3.00m</td></tr><tr><td>600kg</td><td>6.00m</td></tr><tr><td>≥1 tonne</td><td>Up to 20.00m</td></tr></table>Onsite calibration and full time vibration monitoring will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of equipment and review of dilapidation surveys and excavation location is necessary to determine need for full time monitoring. Where monitoring is determined as necessary then it should be maintained directly between the excavation activity and the structure being monitored, as such the monitor may require relocation during excavation.</td></tr>
<tr><td>Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))</td><td>Neighbouring dwellings = 5mm/s within 10m of excavation,
Sydney water sewer asset located across the rear of the site:
- Maximum PPV for intermittent vibrations: 10mm/s
- Maximum PPV for continuous vibrations: 5mm/s</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td>If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td>Pending proposed excavation equipment and vibration calibration testing results, if required</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td>Yes, recommended that these inspections be undertaken as per below mentioned sequence:
• Upon demolition and initial clearing of soils
• During installation of excavation support where determined necessary
• At 1.50m depth intervals of excavation
• Where ground conditions are exposed that differ to those than expected</td></tr>
<tr><td>Dilapidation Surveys Requirement</td><td>Recommended 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.</td></tr>
<tr><td colspan="2">Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil</td></tr>
</table>

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	Any fill/surficial soils within the proposed excavation will require support prior to excavation where safe batters cannot be constructed. MS bedrock is generally self-supporting if no defects, however, poor rock mass will require retention. This is only determinable through cored boreholes or regular geotechnical inspection during excavation works.
Types	Contiguous pile wall where pre-excavation support required due to sandy soils. Steel reinforced concrete/concrete block wall where temporary support or stable batters can be implemented 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/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill/Natural Soils	18	$\phi' = 28°$	0.35	0.52	N/A
VLS to LS bedrock	23	$\phi' = 38°$	0.10	0.15	300 kPa
MS bedrock (defect free)	24	$\phi' = 38°$	0.00	0.01	600 kPa

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 supporting 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).

<table>
<tr><th colspan="3">4.3.4. Drainage and Hydrogeology</th></tr>
<tr><td colspan="2">Groundwater Table or Seepage identified</td><td>No to RL 24.82</td></tr>
<tr><td rowspan="2">Excavation likely to intersect</td><td>Water Table</td><td>No</td></tr>
<tr><td>Seepage</td><td>Minor seepage (<0.5L/min) on defects and at soil/rock interface</td></tr>
<tr><td colspan="2">Site Location and Topography</td><td>South-western side of Reddall Street within gentle north-east dipping topography.</td></tr>
<tr><td colspan="2">Impact of development on local hydrogeology</td><td>Negligible</td></tr>
<tr><td colspan="3">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 the Council's stormwater system off site.</td></tr>
</table>

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 the structural drawings, for inclusion of the recommendations of this report,
2. Review excavation methodology and equipment prior to hard rock excavation,
3. Conduct inspections as per the recommendations of Section 5.3 in this report including inspection of excavation conditions or core drilling and all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness, and the stability prior to the placement of steel or concrete.
4. Inspect the completed development to ensure all retention and stormwater systems are complete and connected and that construction activity has not created any new landslip hazards

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. 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 fill and natural sand/clayey sand in all test locations up to a maximum depth of 0.75m. Sandstone bedrock was subsequently encountered from a minimum depth of 0.50m respectively. A freestanding groundwater table was not identified within the boreholes and is considered unlikely based on topography and elevations.

The proposed works involve the demolition of existing site structures and the construction of a new three storey dwelling broadly towards the middle with a swimming pool and cabana towards the rear. The proposed Basement level will require bulk excavation up to 2.70m depth gradually reducing to nil at the front eastern boundary, whilst the proposed pool will require 2.00m depth excavation.

The proposed excavation is anticipated to extend through a relatively shallow layer of surficial soils before encountering sandstone bedrock for the majority of the excavation initially of very low strength however expected to grade too low to medium strength and potentially high strength with depth and hard rock excavation equipment is likely to be required,

It is recommended that a preliminary vibration limit (Maximum Peak Particle Velocity, PPV) of 5mm/s PPV be set at the founding level for neighbouring structures fand 3mm/s PPV for the Sydney Water assets for all excavation work on this site to maintain comfort levels and provide a very low probability of structural damage.

Safe batter slopes with respect to neighbouring properties do not appear achievable for all northern and southern excavation edges and the pool/deck areas western excavation edge. Therefore, pre-excavation support will need to be installed in the form of a contiguous pile wall (or similar). This wall will need to be installed and taken through any soils and founded within competent bedrock or below the proposed excavation base level.

It is recommended that all new footings extend through any topsoil/fill and natural soils encountered and be founded on competent sandstone bedrock of at least very low strength to avoid variable settlement within the new structure.

There is expected to be negligible impact to the local hydrogeology with the water table not intersected or expected within the depth of proposed works. Some intersection of seepage is expected, and open excavations will provide minor dewatering along defects near the surface. The soils underlying the site are not considered to be Acid Sulfate Soils and lowering of the water table is not envisaged therefore any Acid Sulfate Soils Management Plan will not be required.

The risks associated with the proposed development as well as the existing site conditions can be maintained within an 'Acceptable' Risk Management Criteria 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.

Prepared By:	Reviewed By:
SD	
Sores Demirbag	Troy Crozier
Geotechnical Engineer	Principal
B.E. (Hons.) Civil	MIEAust., CPEng
	MAIG, RPGeo
	Registration No.: 10197

6. 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. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
3. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
4. Australian Standard AS 2870 – 2011, Residential Slabs and Footings
5. AS1170.4 – 2007, Part 4: Earthquake actions in Australia

Appendix 1

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: -

$$Q_c \text{ (MPa)} = (0.4 \text{ to } 0.6)\ N \text{ blows (blows per 300mm)}$$

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

$$Q_c = (12 \text{ to } 18)\ C_u$$

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

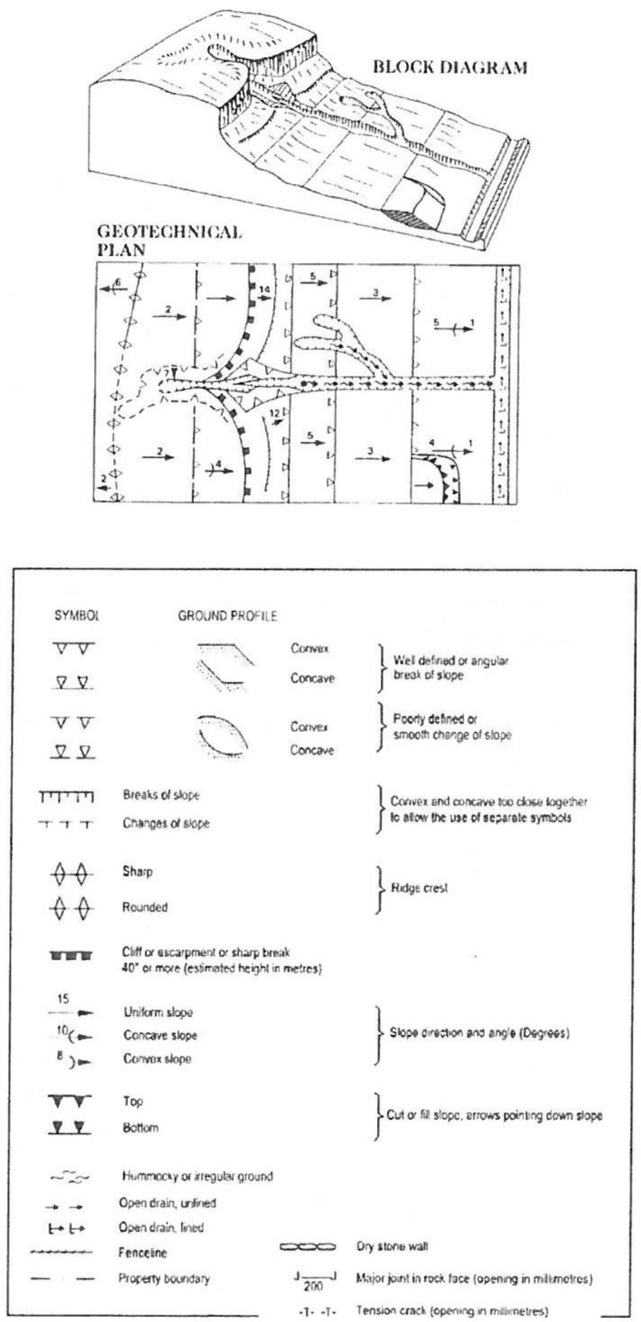
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

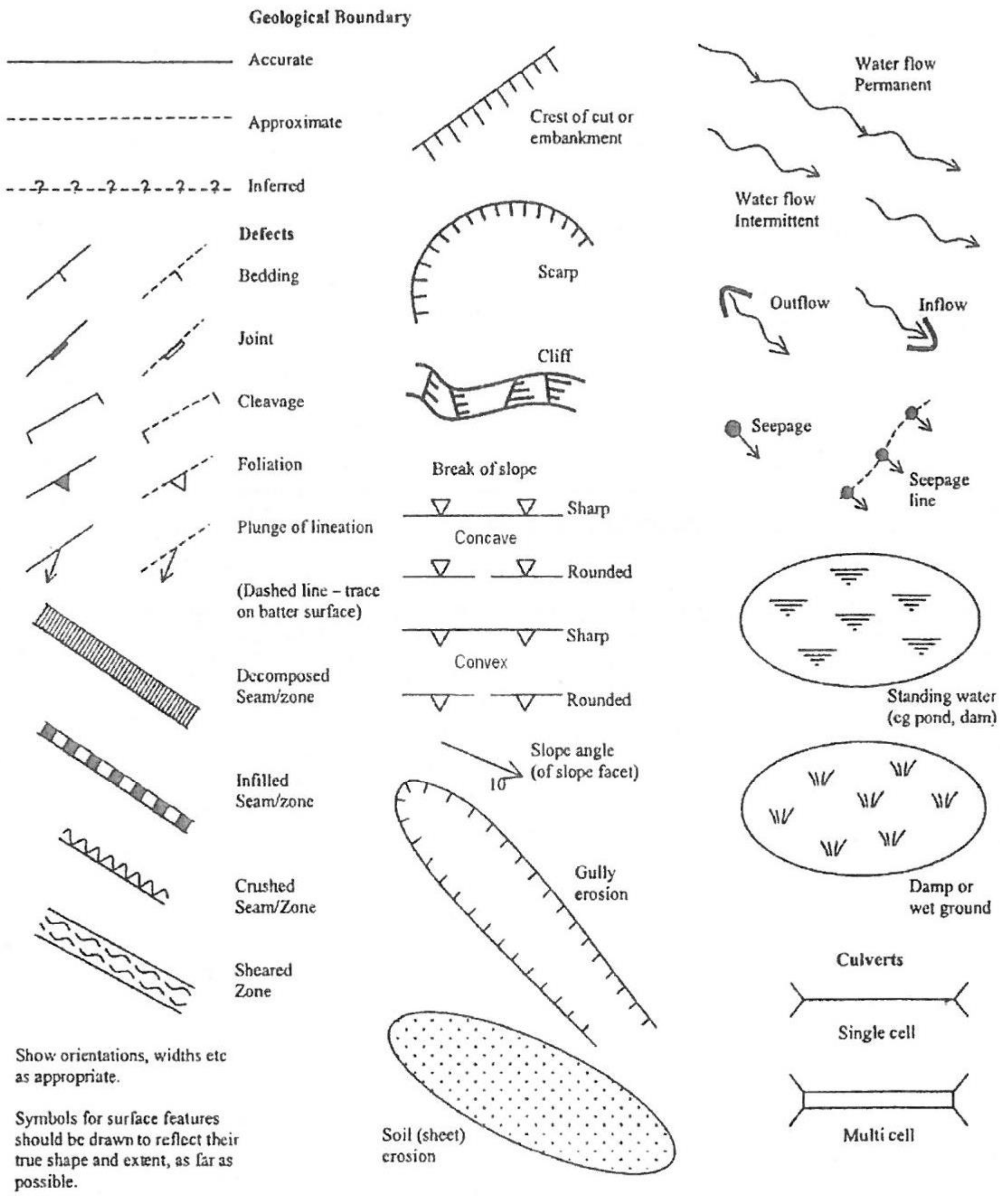
Site Inspection

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.



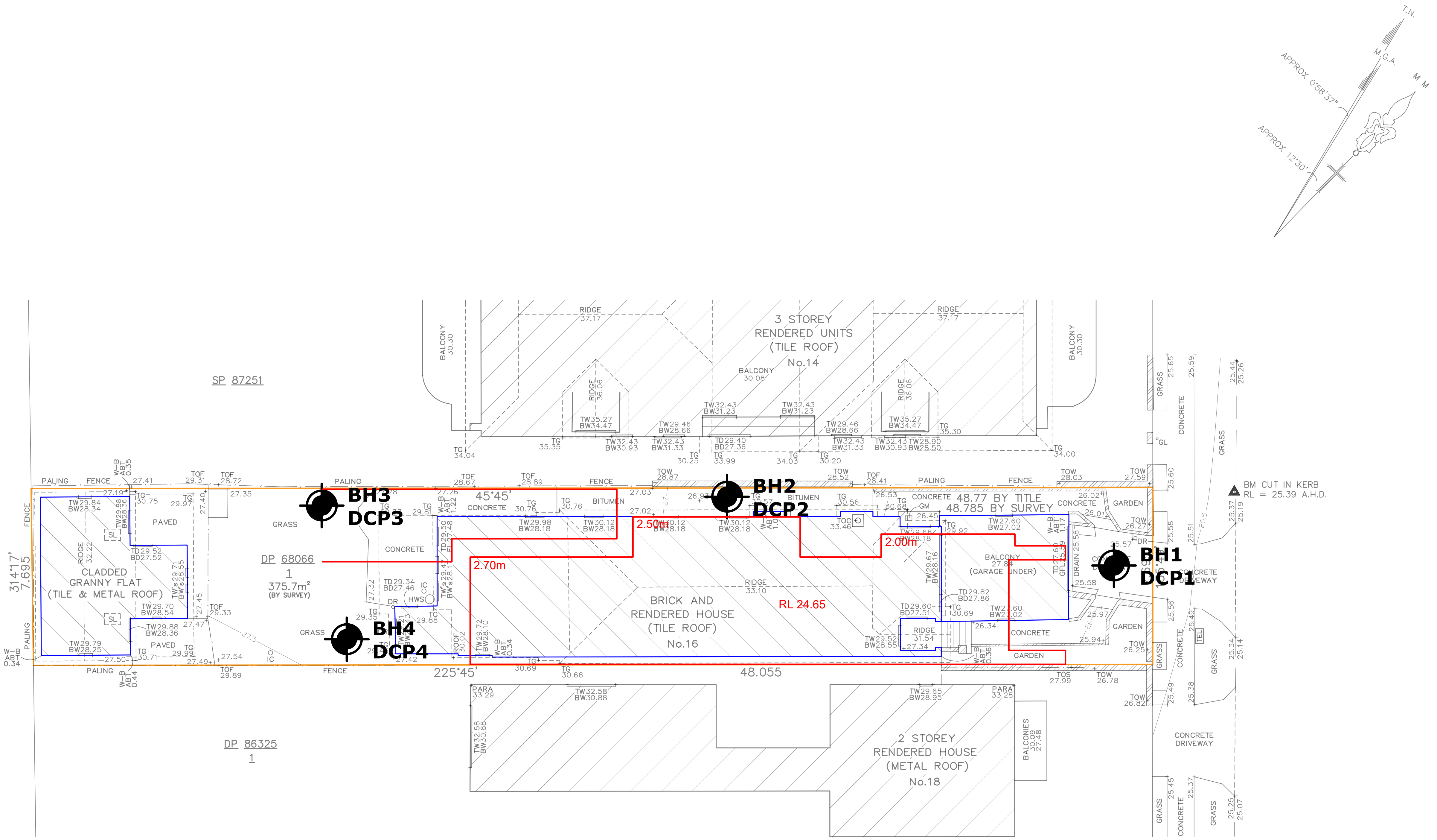
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

SITE PLAN & TEST LOCATIONS

FIGURE 1.

CROZIER GEOTECHNICAL CONSULTANTS

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

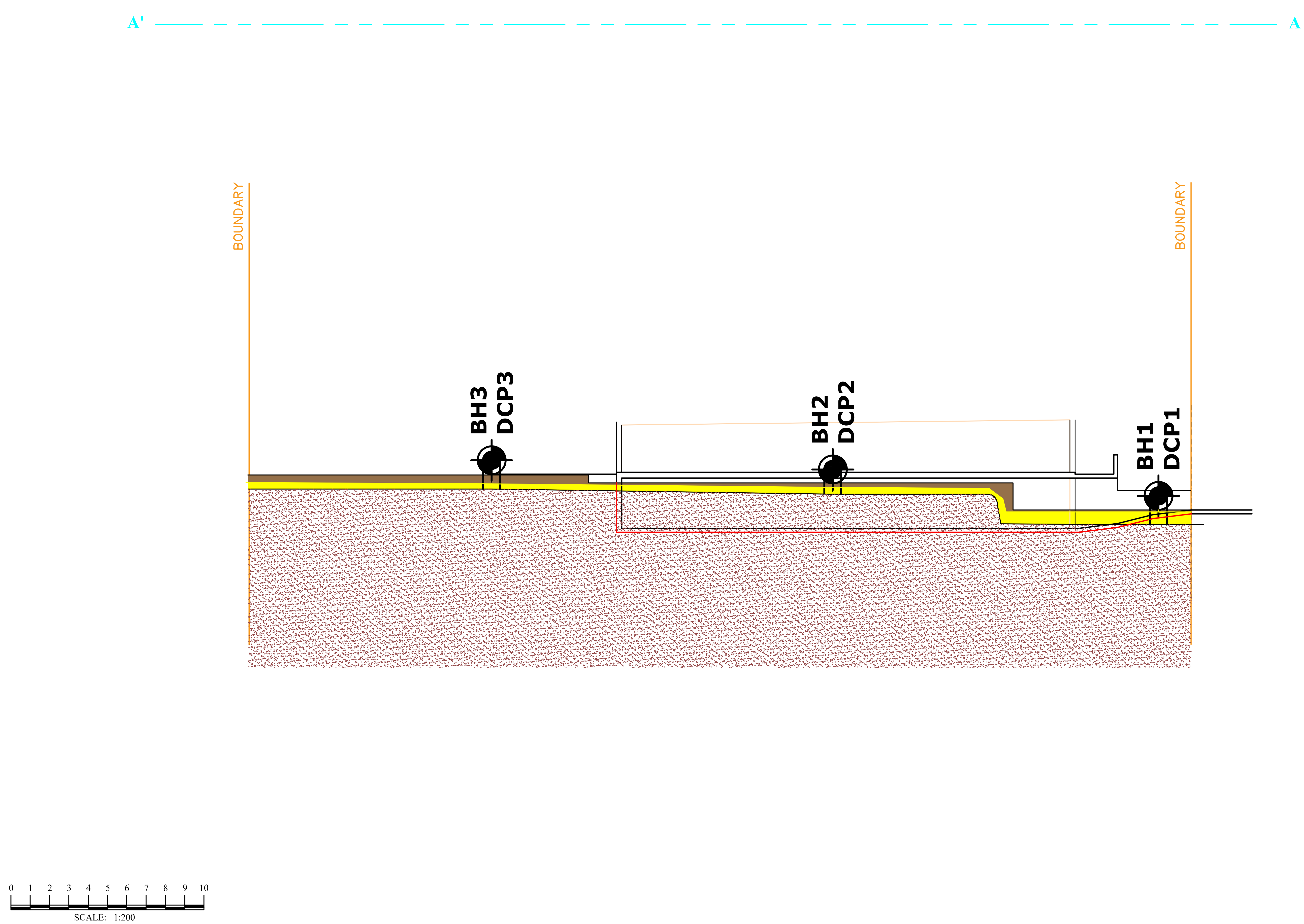
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
	EXISTING STRUCTURES
	PROPOSED EXCAVATION AND APPROXIMATE DEPTHS
	PROPERTY BOUNDARY
A —— A'	SECTION LINE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	09/2024
APPROVED BY:	TMC
DRAWN BY:	SD
PROJECT:	2024-166

PREPARED FOR:
Mike Norman

ADDRESS:
16 Reddall Street, Manly, NSW



SITE SECTION

FIGURE 2.

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

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
	PROPOSED EXCAVATION
	TOPSOIL/FILL
	SAND
	SANDSTONE BEDROCK
A —— A'	SECTION LINE

SCALE:	1:200 @ A3	PREPARED FOR: Mike Norman
DRAWING:	FIGURE 2	
DATE:	09/2024	
APPROVED BY:	TMC	ADDRESS: 16 Reddall Street, Manly, NSW
DRAWN BY:	SD	
PROJECT:	2024-166	

BOREHOLE LOG

CLIENT: Mike Norman **DATE:** 4/09/2024 **BORE No.:** 1

PROJECT: New Development **PROJECT No.:** 2024-166 **SHEET:** 1 of 1

LOCATION: 16 Reddall Street, Manly, NSW **SURFACE LEVEL:** RL 25.57

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: Brown to grey, fine to medium grained, dry to moist sandy fi				
0.10	SP	SAND: Very dense, pale brown to orange, medium to coarse grained, moist sand with sandstone gravels	D	0.10 0.20		
0.70		Hand Auger refusal at 0.70m depth on interpreted sandstone bedrock of at least very low strength				

RIG: Not Applicable DRILLER: A.C

METHOD: Hand Auger LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mike Norman | **DATE:** 4/09/2024 | **BORE No.:** 2

PROJECT: New Development | **PROJECT No.:** 2024-166 | **SHEET:** 1 of 1

LOCATION: 16 Reddall Street, Manly, NSW | **SURFACE LEVEL:** RL 26.93

Depth (m) 0.00	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.05		BITUMEN PAVEMENT				
0.20		FILL: Brown, fine to medium grained, dry sandy fill with building refuse				
0.50	SP	SAND: Very dense, pale brown to orange, medium grained, moist sand with sandstone gravels				
		Hand Auger refusal at 0.50m depth on interpreted sandstone bedrock of at least very low strength				

RIG: Not Applicable

DRILLER: A.C

METHOD: Hand Auger

LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS:

CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mike Norman **DATE:** 4/09/2024 **BORE No.:** 3

PROJECT: New Development **PROJECT No.:** 2024-166 **SHEET:** 1 of 1

LOCATION: 16 Reddall Street, Manly, NSW **SURFACE LEVEL:** RL 27.30

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSOIL/FILL: Medium dense, dark brown to brown, fine to medium grained, dry to moist sandy fill with rootlets				
0.15		… brown sand with building refuse (disturbed natural)				
0.50	SC	CLAYEY SAND: Very dense, pale brown, medium grained, dry to moist clayey sand	D	0.50 0.60		
0.70		Hand Auger refusal at 0.70m depth on interpreted sandstone bedrock of at least very low strength				
1.00						

RIG: Not Applicable DRILLER: A.C

METHOD: Hand Auger LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mike Norman **DATE:** 4/09/2024 **BORE No.:** 4

PROJECT: New Development **PROJECT No.:** 2024-166 **SHEET:** 1 of 1

LOCATION: 16 Reddall Street, Manly, NSW **SURFACE LEVEL:** RL 27.42

Depth (m)	**Classification**	**Description of Strata** **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	**Sampling**		**In Situ Testing**	
			Type	**Tests**	**Type**	**Results**
0.00		TOPSOIL/FILL: Very loose, brown to grey, fine to medium grained, dry to moist sandy fill with rootlets				
0.30	SC	CLAYEY SAND: Medium dense, pale brown, medium grained, moist clayey sand				
0.50		Hand Auger refusal at 0.50m depth on interpreted sandstone bedrock of at least low strength				

RIG: Not Applicable DRILLER: A.C

METHOD: Hand Auger LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: CHECKED: T.M.C

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Mike Norman
PROJECT: New Development
LOCATION: 16 Reddall Street, Manly, NSW

DATE: 4/09/2024
PROJECT No.: 2024-166
SHEET: 1 of 1

	Test Location									
Depth (m)	1	2	3	4						
0.00 - 0.10	5	3	2	1						
0.10 - 0.20	18	3	4	1						
0.20 - 0.30	26	23	4	0						
0.30 - 0.40	15	19	4	3						
0.40 - 0.50	16	10	5	6						
0.50 - 0.60	16	(B) at 0.55m	7	(B) at 0.50m						
0.60 - 0.70	8		24							
0.70 - 0.80	16		(B) at 0.75m							
0.80 - 0.90	(B) at 0.75m									
0.90 - 1.00										
1.00 - 1.10										
1.10 - 1.20										
1.20 - 1.30										
1.30 - 1.40										
1.40 - 1.50										
1.50 - 1.60										
1.60 - 1.70										
1.70 - 1.80										
1.80 - 1.90										
1.90 - 2.00										
2.00 - 2.10										
2.10 - 2.20										
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <2m³) from soils at crest of excavation		Potentially up to 0.75m of soil	a) Excavation setback from the northern boundary by 0.00m for Pool and 1.25m for Basement, Building setback 2.20mm, impact 1% b) Excavation setback from the southern boundary by 0.00m, House setback 0.85m, impact 1%		a) Person in house 18hrs/day avge. b) Person in house 18hrs/day avge.	a) Likely to not evacuate b) Likely to not evacuate	a) Person in house, minor damage only b) Person in house, minor damage only	
			Likely	**Prob. of Impact**	**Impacted**				
		a) Apartment building of No. 14 Reddall Street	0.01	0.10	0.05	0.7500	0.75	0.05	**1.41E-06**
		b) House of No. 18 Reddall Street	0.01	0.10	0.05	0.7500	0.75	0.05	**1.41E-06**
B	Landslip (rockslide/topple <5m³) of bedrock around perimeter of excavation due to poorly oriented defects		Potentially up to 2.00m of exposed bedrock	a) Excavation setback from the northern boundary by 0.00m for Pool and 1.25m for Basement, Building setback 2.20mm, impact 5% b) Excavation setback from the southern boundary by 0.00m, House setback 0.85m, impact 10%		a) Person in house 18hrs/day avge. b) Person in house 18hrs/day avge.	a) Likely to not evacuate b) Likely to not evacuate	a) Person in house, minor damage only b) Person in house, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Apartment building of No. 14 Reddall Street	0.001	0.25	0.10	0.7500	0.75	0.05	**7.03E-07**
		b) House of No. 18 Reddall Street	0.001	0.25	0.10	0.7500	0.75	0.05	**7.03E-07**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems

* likelihood of occurrence for design life of 100 years

* Spatial Impact - Probability of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.

* considered for person most at risk, where multiple people occupy area then increased risk levels

* for excavation induced landslip then considered for adjacent premises/buildings founded off 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). Based on likelihood of person knowing 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

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <2m³) from soils at crest of excavation	a) Apartment building of No. 14 Reddall Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
		b) House of No. 18 Reddall Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
B	Landslip (rockslide/topple <5m³) of bedrock around perimeter of excavation due to poorly oriented	a) Apartment building of No. 14 Reddall Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Low
		b) House of No. 18 Reddall Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	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%.

* Cost of site development estimated at $1,000,000

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}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	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.