

REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

28 ETHEL STREET, BALGOWLAH, NSW

Prepared For

Sara and Richard Crampton

Project No.: 2021-198

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**GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS
28 ETHEL STREET, BALGOWLAH, NSW**

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 28 Ethel Street, Balgowlah, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Studio Esteta on behalf of the clients Sara & Richard Crampton.

It is understood the rear of the existing site house is to be partially demolished with the construction of a two storey northern extension. A proposed connected garage is to be constructed within the rear of the site and will require bulk excavation to a maximum of approximately 1.9m depth. A partly in ground swimming pool is also proposed within the north eastern corner of the block and will require minor bulk excavation to approximately 0.6m depth.

According to the Manly Development Control Plan 2013 Section 2.1.13.1

a) A Site Stability Report is required with a DA when the proposed development involves:

- i) any land identified on the LEP Landslide Risk Map. In this regard a DA for development on land identified on the LEP Landslide Risk Map must consider certain matters under LEP clause 6.8;*
- ii) any excavation greater than 1m below natural ground level for a basement or basement car parking area;*

The site is located within a Landslip Risk Class G4 as identified within Northern Beaches (Manly) Councils – Development Control Plan 2013 – Schedule 1 Map C and within a Class 5 Acid Sulphate Soils zone.

The geotechnical report submitted with the Development Application (DA) must detail how the development may be achieved to ensure geotechnical stability is maintained using good engineering practices. The report must also include a risk assessment for existing/potential instability as per the AGS March 2007 publication. Therefore, this report includes a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provides recommendations for design, planning and construction.

The investigation and reporting were undertaken as per the Proposal P21-355, Dated: 23rd July 2021. The investigation and reporting were prepared to assist in the Development Application and preliminary design and construction tendering.

The investigation comprised:

- a) A detailed geotechnical inspection and mapping of the site and inspection of adjacent properties by a Geotechnical Engineer.
- b) Onsite service location and clearing of borehole locations by an accredited contractor.
- c) Drilling of three boreholes and excavation of one test pit using hand tools along with Dynamic Cone Penetrometer (DCP) testing adjacent to the borehole/test pit locations to investigate subsurface conditions.
- d) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional.

The following plans were supplied and relied upon in the preparation of the investigation and this report:

- Architectural Drawings – Esteta, Project No.: 2106, Drawing No.: A002, A102 – A104, Dated: 05/2021
- Survey Drawing – Dunlop Thorpe & Co., Reference No.: 17364, Dated: 5/5/2011
- Scope of Works – Esteta, via email, Dated 12/07/2021

2. PROPOSED WORKS

The proposed works involve the partial demolition of the rear of the existing house with the construction of a two storey rear extension and connected garage structure. Further internal alterations are planned for the Ground Floor level of the existing structure. It is understood that the First Floor level is to overlie the proposed northern extension only and not the original structure.

The proposed Garage Floor is to be formed at an FFL of RL 72.64m and will connect to the Ground Floor level of the main house via internal stairs. The garage structure will be accessed via a new driveway connected to Whittle Avenue. Bulk excavation will be required to a maximum of approximately 1.9m depth for the garage reducing to the north to $\leq 0.50\text{m}$, with the majority of excavation required within southern portions of the building footprint. The proposed garage structure will have a northern side setback of 2.0m and a western side setback of 1.0m.

A partly inground swimming pool is also to be installed within the rear of the site. The pool will be positioned within the north eastern corner of the block and will comprise a 3.0m x 6.0m x 1.7m deep precast concrete plunge pool. The pool deck will be formed at the Ground Floor level (RL75.14m) with the pool shell to require minor excavation within southern and western portions to $\leq 0.6\text{m}$ depth. The proposed swimming pool will have a 1.1m northern side setback, with the surrounding retaining walls and pool paving to extend to the eastern boundary.

3. SITE FEATURES:

3.1. Site Description:

The site is a rectangular shaped block with front (south) and rear (north) boundaries of 12.19m and side east and west boundaries of 39.635m, as referenced from the provided Survey Plan. The site is located on the corner of Ethel Street and Whittle Avenue, within gentle ($\approx 4^\circ$) north west dipping topography. An aerial photograph of the site and its surrounds is provided below (Photograph 1), as sourced from NSW Government Six Map spatial data.



Photograph 1. Aerial photograph of site and surrounds

The site contains a single storey brick cottage with a northern deck and rear metal shed. A concrete strip driveway extends along the western boundary of the site, with small gardens and grass located within the front and rear of the block. An exposed bedrock outcrop is exposed along the western boundary of the site,

comprising medium to coarse grained, medium strength sandstone. Views of the front and rear of the site are shown as Photograph 2 & 3.



Photograph 2. View of the rear of the main structure, facing south



Photograph 3. View of the front of the site, facing north

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet indicates that the site is underlain by Hawkesbury Sandstone 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. This unit was identified outcropping within the rear of the site.

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 defects orientations being south-east and north-east. Numerous sections of cliff are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15° to 23°) and are randomly covered by sandstone boulders. Interpreted sandstone bedrock and boulder outcrops were identified within the site. An extract of the relevant Geology Series Sheet is provided below.



Extract 1:100,000 Sydney Series Geological reference extract

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection, mapping of the site and limited inspection of adjacent properties on the 9th September 2021 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 existing features and ground conditions.

It also included the drilling of three auger boreholes (BH1 – BH3) and one test pit (TP1) using hand tools to investigate sub-surface geology.

DCP testing was carried out from the ground surface adjacent to the boreholes/test pit in 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 confirm depths to bedrock.

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

4.2. Field Observations:

The site is situated on the high corner of Ethel Street and Whittle Avenue, within gentle north west dipping topography. Both Ethel Street and Whittle Avenue are formed with bitumen pavement and concrete kerb/gutter, except for the low western side of Whittle Avenue which grades into soil and vegetation. Ethel Street dips gently west, whilst Whittle Street dips very gently north. There were no signs of significant cracking within the surrounding road reserves to suggest any underlying movement.

The site contains a one storey brick cottage positioned towards the front of the site which appears to be in good condition with only minor horizontal cracking identified within the rear wall of the structure. Vehicle access is provided along the western boundary via a concrete stripped driveway from Ethel Street. The front of the block contains a small grassed lawn and surrounding gardens, with a rendered brick and timber boundary wall extending around the southern and western boundary within the front of the site.

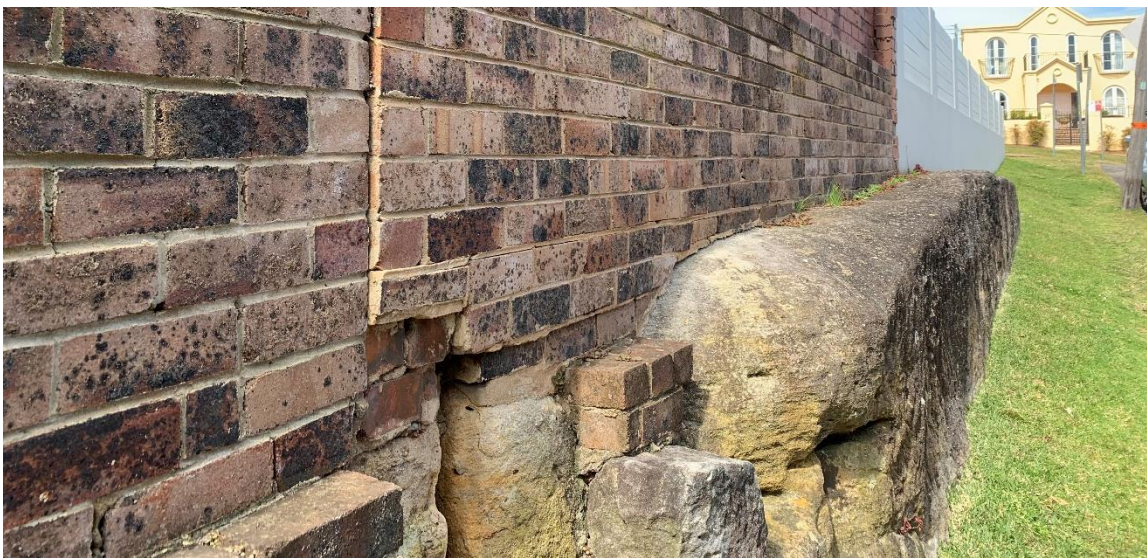
The rear of the site contains an open grassed lawn comprising an upper southern region and lower northern region, with a metal shed located in the north eastern corner of the block. A timber deck is positioned at the rear of the house and extends to the grassed lawn. Remnants of a previous brick structure are located within

the upper southern region of the rear, as vertical brick walls are positioned adjacent to the western boundary and appear to underlie portions of the grass. A single skin brick 1.55m high boundary wall is positioned along the western and northern boundaries within the rear and has undergone significant deflection and settlement within western portions. The north western corner of the wall has a $\leq 150\text{mm}$ deflection and significant outward rotation as shown in Photograph 4.



Photograph 4: Outward rotation within the western boundary wall within the rear of the site, with significant deflection and $\leq 150\text{mm}$ separation from adjacent northern boundary wall, facing north.

Exposed sandstone bedrock was identified within the site and surrounding areas. A $\approx 10\text{m}$ long section of outcropping bedrock was observed below the western boundary fence, with the rendered boundary wall, brick wall and north western portion of the main structure all bearing on the exposed bedrock. This outcrop is shown in Photograph 5. Exposed sandstone bedrock was also identified within the rear lower northern portion of the site.



Photograph 5: Exposed sandstone bedrock along the western boundary of the site, with brick wall of main structure and western boundary wall formed on outcrop.

Stormwater drainage and gutter systems were observed across the site structure, with down pipes discharging into drainage pits across the site, one drain was identified within the lower northern portion of the rear. A sewer main intersects the north portion of the site, with an inspection pit located adjacent to the western boundary within Whittle Avenue road reserve. Based on available DBYD information it is understood the sewer main comprises a 225mm diameter vitrified clay pipe, with the adjacent inspection cap indicating a pipe invert level between 1.1m and 1.8m depth.

Sydney Road is located approximately 15m to the west of the site and forms a triple carriageway motorway which has been excavated $\leq 7.0\text{m}$ into the sandstone bedrock below the site levels. Sub vertical sandstone bedrock sidewalls extend along either side of the motorway.

The neighbouring property to the east (No.26 Ethel Street) contains a two storey brick and weatherboard modern house positioned 1.8m from the common boundary with the site. A single storey brick structure is located within the rear of the property and is positioned on the common boundary with the site. An inground swimming pool is also positioned within the rear of the property, with the surrounding ground level appearing to be approximately 0.5m higher than the site adjacent to the common boundary.

The neighbouring property to the north (No.19 Whittle Avenue) contains a one and two storey brick house formed with a sandstone block base, the house is classified as a heritage item (I285) according to the Manly LEP 2013 and is of an estimated 1930's construction. The structure is located approximately 1.0m from the common boundary with the site, with the front of the property containing sealed concrete surfaces with a

metal carport within the front north western corner. Inspection of the rear of the property was not possible from the site.

4.3. Ground Conditions:

For a description of the ground conditions encountered at the borehole/test pit/DCP test locations, the Borehole Log and DCP results sheets should be consulted however a very broad summary of the subsurface conditions encountered is provided below.

- **TOPSOIL/FILL** – this layer was encountered from surface at all test locations to a maximum depth of 0.80m (BH3) below the existing ground surface. This layer comprised dark brown, medium grained, moist, silty sand.
- **CLAYEY SAND/SANDY CLAY** – Interpreted extremely weathered material was identified in BH3 and TP1 from a minimum of 0.50m depth (TP1). Clayey sand grading to sandy clay to a maximum drilled depth of 1.20m was encountered in BH3. The clayey sand layer is classified as very loose, grey, medium to coarse grained, moist to wet, extremely weathered sandstone.
- **SANDSTONE BEDROCK** – Bedrock grading from extremely low strength to at least very low strength was encountered at all test locations. Sandstone bedrock of at least very low strength was interpreted between 0.31m (DCP2) and 1.10m depth (DCP3).

A test pit was excavated in the rear lawn to expose shallow bedrock underlying the fill soils. Inspection of the bedrock surface condition and nature showed extremely low strength sandstone at 0.50m depth grading to sandstone of at least very low strength from 0.80m depth.

Minor seepage was encountered at the soil – bedrock interface within BH3. A freestanding groundwater table or significant seepage was not identified underlying the site.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified the presence of silty sand topsoil and fill to a maximum depth of 0.80m (BH3). Extremely weathered/extremely low strength sandstone bedrock was encountered in BH3 & TP1 from a minimum of 0.50m depth (TP1), this extremely weathered material grades from clayey sand to sandy clay to a maximum depth of 1.20m (BH3). Sandstone bedrock of at least very low strength was encountered between 0.31m (DCP2) and 1.10m depth (DCP3).

The proposed works include the partial demolition of the rear of the house and the construction of a two storey rear extension, along with a rear garage structure and swimming pool. The garage structure will be positioned within the northwestern corner of the site and will require bulk excavation to a maximum of approximately 1.9m depth, the installation of the swimming pool shell will also require bulk excavation to a maximum of approximately 0.6m depth.

It is understood that excavation will largely be concentrated to the upper southern portion of the rear of the site for the proposed garage structure. Shallow sandstone bedrock $\leq 0.40\text{m}$ depth was identified within this region (DCP1 & DCP2). It is therefore anticipated that sandstone bedrock of very low strength quickly grading to low/medium strength will be encountered through the majority of the proposed excavation.

The investigation using hand tools could not penetrate bedrock harder than very low strength. Bedrock of at least low 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. There is a potential for poorly oriented defects or localized zones of highly weathered bedrock (particularly near the upper surface) to result in localized rock slide/topple failure with potential impact to the site. However, based on excavation depths and separation distances there is a very low potential for impact to property boundaries and neighbouring structures.

Therefore, geotechnical inspection following initial clearing of the bedrock surface is required to confirm site conditions along with inspection at regular depth intervals during excavation. Through selection of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within 'Acceptable' levels for all situations.

Sandstone outcrop/bedrock of low to medium strength was identified along the western boundary of the site. Excavation of hard bedrock (low to high strength) will require rock excavation equipment (i.e. rock hammer). The use of rock hammers has the potential to create ground vibrations which could impact neighbouring structures including the heritage listed neighbouring residence at No.19 Whittle Avenue (I285). 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 ($\leq 250\text{kg}$) rock hammers be proposed for use at this site to avoid the need for full time monitoring.

The existing house structure appeared to be in a stable condition with no major signs of cracking or settlement, except for within the existing north western laundry block which is to be demolished. It is understood that the northern extension to the house is to be independently founded, with no new loading conditions from the proposed First Floor level imposed onto the original structure. Considering the shallow depth of sandstone bedrock interpreted to underlie the whole site, it is likely that the existing structure is founded on this unit. It is recommended that all new footings for the proposed northern extension, garage structure and swimming pool should extend to bear onto sandstone bedrock of at least low strength.

Where batter slopes are not feasible due to proximity of site boundaries or site structures, support prior to excavation will be required. Shoring walls will need to be installed around any of the excavation perimeters which do not comply with the safe batter slope requirements detailed in Section 5.3.2. This does not appear to be required for any of the pool excavation sides.

Existing landslip hazards were not identified however the excavation will create potential stability hazards though these are not expected to impact adjacent properties but will need to be considered during construction. The existing northern boundary wall is in an unstable condition and should be removed or reconstructed as part of or prior to demolition and excavation works.

It is understood that a Sydney Water (SW) sewer underlies the rear of the site, with a local sewer pipe identified extending from the rear of the main structure to the sewer main. It appears that the excavation and proposed footings for the garage and swimming pool will likely be within a 1.0V : 1.0H Influence Zone of the sewer pipe and as such subject to the Sydney Water Technical Guidelines - Building over and adjacent (BOA) to pipe assets.

CGC has not undertaken any investigation into the construction/type/depth etc. of the asset however DBYD plans indicate it is a 225mm Vitrified Clay pipe which will be highly susceptible to deformation and impact

from excavation works. Based on previous experience it is recommended that Sydney Water be contacted as soon as possible to determine what requirements may exist in order to protect the asset. For if the proposed works are to be completed within this influence zone then to meet the Sydney Water requirements for BOA works, a specialist engineering assessment (SEA), including an appraisal of the impact of the proposed permanent and temporary works on the pipe assets using the appropriate site investigation, engineering modelling or analysis method, has to be submitted for acceptance.

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 limited scope of investigation using augering techniques only. This investigation 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.

5.2. 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. Toppling/sliding of unstable block of rock formed by intersecting defects within garage excavation.
- B. Landslip of surficial soils from excavation works for swimming pool and adjacent retaining walls.

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 **Hazard A and B** were estimated to be up to 7.18×10^{-8} for a single person, whilst the **Risk to Property** from the hazards were considered to be up to 'Low'.

Although the risk to property levels are considered to be ‘Unacceptable’ against the AGS Guidelines, the assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented the likelihood of any failure becomes ‘Rare’ and as such the consequences reduce and risk levels become within the ‘Acceptable’ risk management criteria. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class ‘A’ for footings at base of excavation into rock
Type of Footing	Strip/Pad or Slab or Piles
Maximum Allowable Bearing Capacity	- Weathered, ELS-VLS Bedrock: 700kPa - Weathered LS Bedrock: 1000kPa
Site sub-soil classification as per <i>Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia</i>	B _e – rock site
Remarks: All footings should be founded off bedrock of at least low 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 ‘certified’ at the end of the project.	

5.3.2. Excavation:				
Depth of Excavation			Maximum of 1.90m depth for the proposed garage structure, maximum of 0.60m for the swimming pool.	
Property Separation:				
Boundary	Adjacent Property	Bulk Excavation Depth	Separation Distances	
			Boundary	Structure
North	No.19 Whittle Avenue	Up to 1.90m, however excavation base at approximate level as property	>2.00m	Gardens on boundary, house a further 1.00m

South	Existing site house	Up to 1.90m	>0.00m	Original structure to be retained adjacent to cut
West	Whittle Avenue road reserve	Up to 1.90m, however excavation base at approximate level as road reserve	1.00m	Grass road reserve, road a further 1.20m.
East	No.26 Ethel Street	Up to 1.90m for garage,	4.50m	Brick secondary structure on boundary
		Up to 0.60m for swimming pool	Nil	
Type of Material to be Excavated		Fill/topsoil to 0.50m depth		
		Extremely low strength bedrock to a maximum of 1.20m depth		
		Very low to low/medium strength bedrock from a minimum of 0.33m depth		
Guidelines for <u>unsurcharged</u> batter slopes are tabulated below:				
Material		Safe Batter Slope (H:V)		
		Short Term/Temporary	Long Term/Permanent	
Fill/topsoil		1.5:1	2:1	
VLS – LS bedrock (fractured)		Vertical	0.25:1*	
MS, defect free bedrock		Vertical*	Vertical*	
Remarks:				
*Dependent on assessment by geotechnical engineer.				
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		Fill /ELS	Excavator with bucket	
		VLS bedrock	Excavator with bucket and ripper	
		LS-MS/HS bedrock	Rock hammer and rock saw	
ELS – extremely low strength, VLS – very low strength, LS – low strength, MS – medium 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 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:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
≥1 tonne	20.00m

Onsite calibration 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.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Existing site structures = 5mm/s No.26 Ethel Street = 5mm/s No.19 Whittle Avenue = 3mm/s Services = 3mm/s
Vibration Calibration Tests Required	If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed
Full time vibration Monitoring Required	Pending proposed excavation equipment and vibration calibration testing results, if required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • For assessment of batter slopes • During installation of the excavation support system • At 1.50m depth intervals of excavation • Where unexpected ground conditions are identified or any other concerns are held. • At completion of the excavation • Following footing excavations to confirm founding material strength
Dilapidation Surveys Requirement	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.
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, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.	

5.3.3. Retaining Structures:					
Required	New retaining structures/excavation support wall will be required as part of the proposed development, including around the proposed swimming pool area.				
Types	Steel reinforced concrete/concrete block wall designed as per AS4678-2002 Earth Retaining Structures where safe batters are achievable.				
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 (K₀)	
Fill/topsoil	18	ϕ' = 28°	0.35	0.52	N/A
VLS to LS bedrock	23	ϕ' = 38°	0.10	0.15	300kPa
MS bedrock (defect free)	24	ϕ' = 40°	0.00	0.01	600kPa
Remarks: In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided to allow release of groundwater seepage. 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 (K₀) earth pressure coefficients and incorporate surcharge loading 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).					

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.5L /min) estimated
Site Location and Topography		North side of Ethel Street and high east side of Whittle Avenue, within gentle north west dipping topography.
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Possible via dispersion only
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. 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 preferably discharges to the Council's stormwater system off site.		

5.4. 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 and hydraulic design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect the installation of excavation support measures if required,
 - Inspect any medium strength bedrock and the proposed equipment prior to its excavation and at 1.50m depth intervals of excavation,
3. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,

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

6. CONCLUSION:

The site investigation identified fill soils overlying sandstone bedrock of extremely low strength generally grading to at least very low strength across the rear of the site. Sandstone bedrock of at least very low strength was identified between 0.33m (DCP1) and 1.20m depth (BH3). Extremely weathered sandstone was identified in TP1 and BH3 from a minimum of 0.50m depth (TP1).

The proposed works involve the partial demolition of the existing house with the construction of a two storey rear extension. A rear garage structure and adjacent swimming pool are also to be constructed which will require bulk excavation to a maximum of approximately 1.9m depth.

Shallow bedrock underlies the site and proposed excavation footprint, therefore suitable rock excavation equipment (i.e. rock saws and rock hammers $\leq 250\text{kg}$) along with an appropriate excavation methodology should be used.

It appears batter slopes may be feasible to support the excavation sides however further details of the proposed pool excavation will be required to confirm this. Achievable safe batter slopes will also be subject to the presence of defects within the bedrock encountered during excavation.

There were no existing/credible landslip hazards identified and the proposed works are relatively minor from a geotechnical perspective and should not create any new instability provided the recommendations of this report are implemented.

The risks associated with the proposed development can be maintained within 'Acceptable' levels (AGS 2007) with negligible impact to the 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.

Prepared By:



Josh Cotton
Geotechnical Engineer

Reviewed By:



Troy Crozier
Principal
MAIG, RPGeo – Geotechnical and Engineering
Registration No.: 10197

7. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. 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.
3. E. Hoek & J.W. Bray 1981, "Rock Slope Engineering" By The Institution of Mining and Metallurgy, London.
4. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall.
5. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin

Appendix 1

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:

<u>Soil Classification</u>	<u>Particle Size</u>
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:

<u>Classification</u>	<u>Undrained Shear Strength kPa</u>
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:

<u>Relative Density</u>	<u>SPT</u> "N" Value (blows/300mm)	<u>CPT</u> 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 separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by 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.

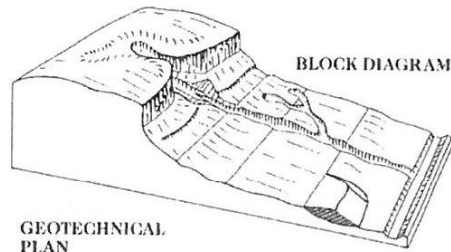
Reproduction of Information for Contractual Purposes

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.

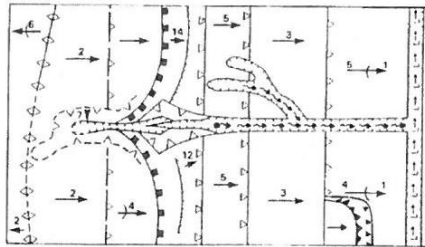
Site Inspection

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.

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007



GEOTECHNICAL
PLAN



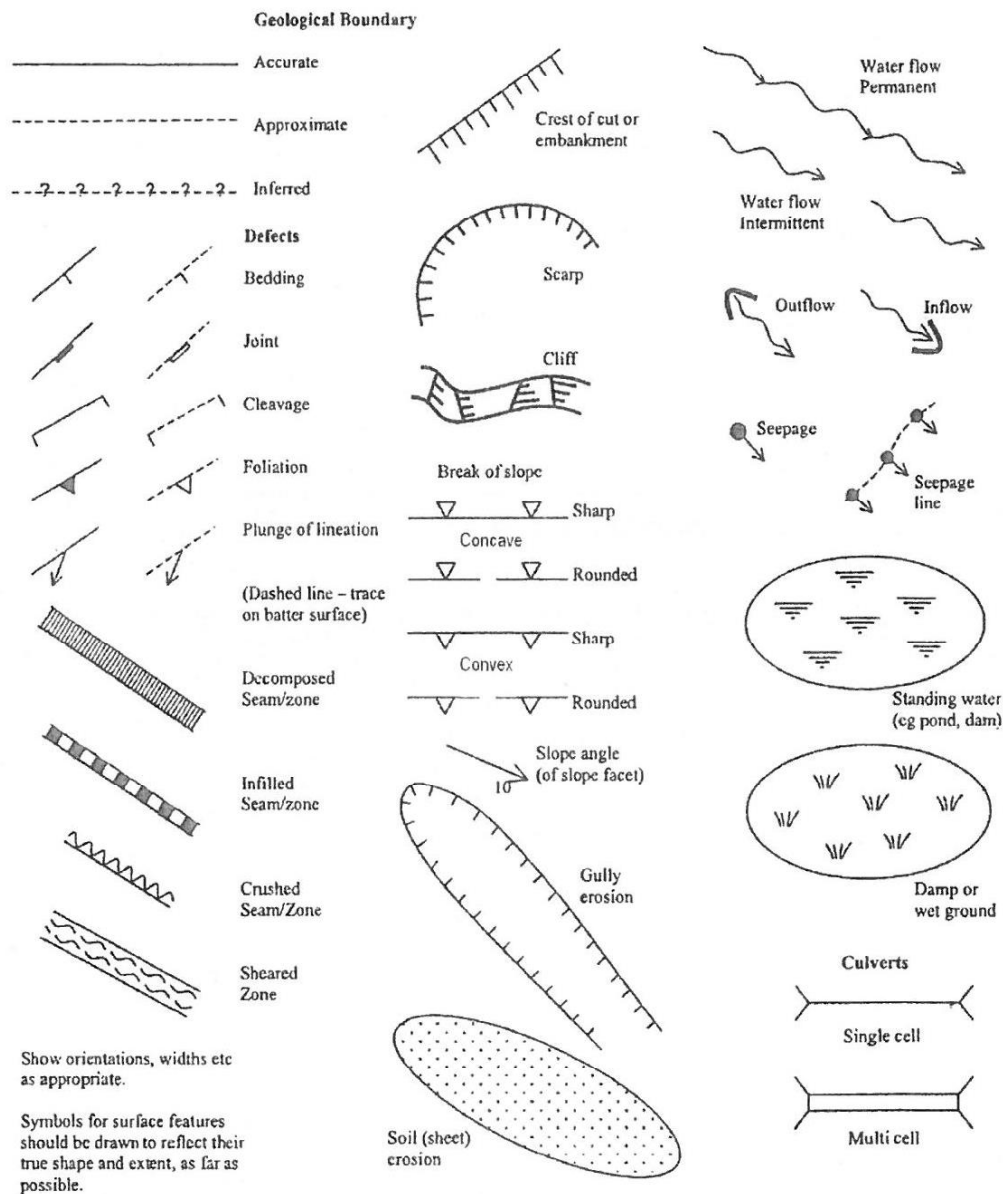
SYMBOL	GROUND PROFILE	
		Convex
		Concave
		Convex
		Concave
		Breaks of slope
		Changes of slope
		Sharp
		Rounded
		Cliff or escarpment or sharp break 40° or more (estimated height in metres)
		Uniform slope
		Concave slope
		Convex slope
		Top
		Bottom
		Hummocky or irregular ground
		Open drain, unlined
		Open drain, lined
		Fence line
		Property boundary
		Dry stone wall
		Major joint in rock face (opening in millimetres)
		Tension crack (opening in millimetres)

Example of Mapping Symbols

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

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

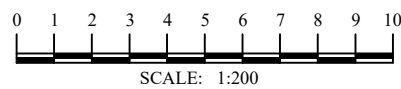
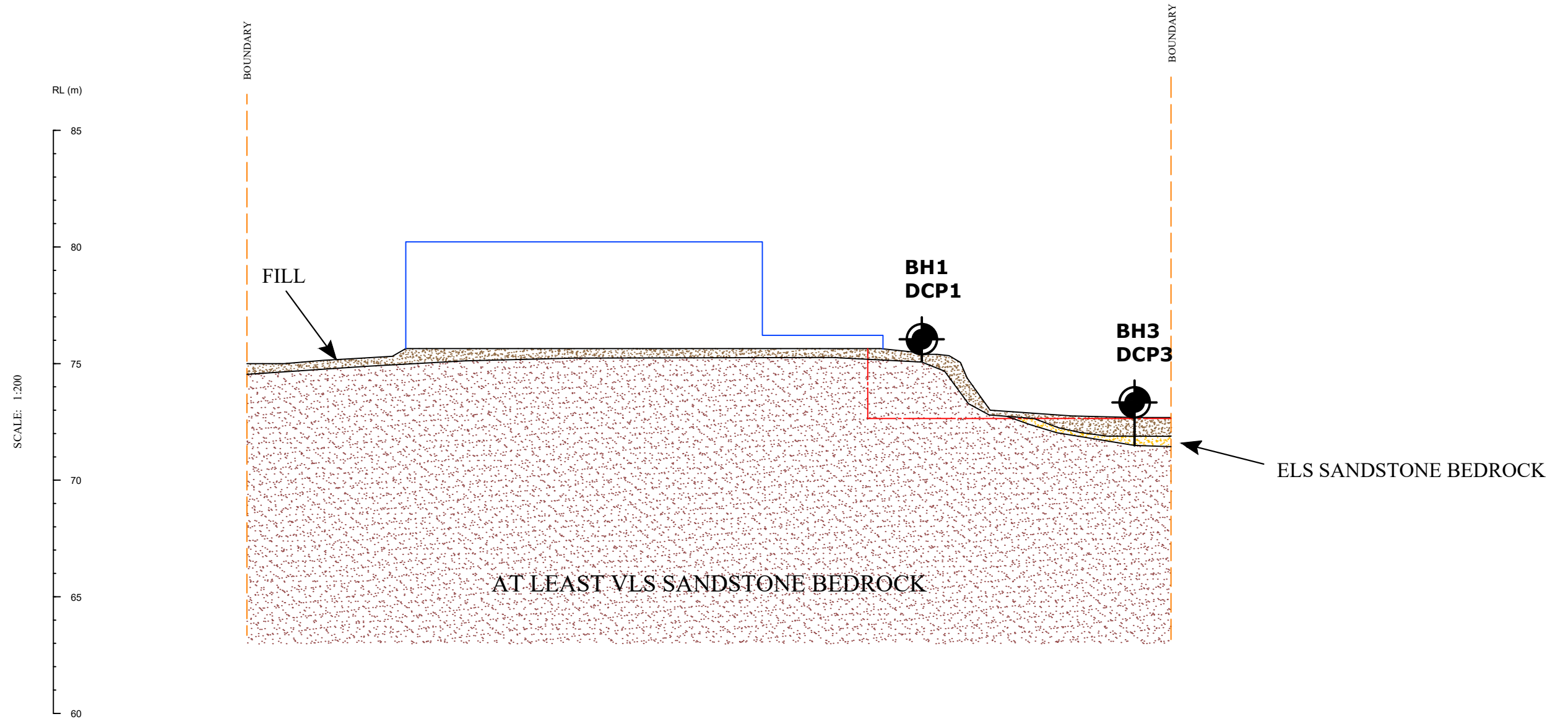


Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2

A

A'



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

GEOLOGICAL MODEL
FIGURE 2.

LEGEND

A — A'

CROSS-SECTION
REFERENCE LINE



AUGER /
DYNAMIC CONE
PENETROMETER
LOCATION



PROPOSED
EXCAVATION
OUTLINE



EXISTING
STRUCTURE



PROPERTY
BOUNDARY



FILL



SANDSTONE
BEDROCK >VLS



ELS SANDSTONE
BEDROCK

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 13/09/2021

APPROVED BY: TMC
DRAWN BY: JC
PROJECT: 2021-198

PREPARED FOR:
SARA & RICHARD CRAMPTON

ADDRESS:
28 ETHEL STREET,
BALGOWLAH

BOREHOLE LOG

CLIENT: Sara & Richard Crampton

DATE: 9/09/2021

BORE No.: 1

PROJECT: Alterations and Additions

PROJECT No.: 2021-198

SHEET: 1 of 1

LOCATION: 28 Ethel Street, Balgowlah

SURFACE LEVEL: RL= 74.42m

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		FILL/TOPSOIL: Dark Brown, medium grained, moist, silty sand				
0.30		...dark grey with sandstone gravel				
0.37		AUGER REFUSAL @ 0.37m depth on sandstone bedrock of at least very low strength				
1.00						
2.00						

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: JC

GROUND WATER OBSERVATIONS: None encountered during auger drilling

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Sara & Richard Crampton

DATE: 9/09/2021

BORE No.: 2

PROJECT: Alterations and Additions

PROJECT No.: 2021-198

SHEET: 1 of 1

LOCATION: 28 Ethel Street, Balgowlah

SURFACE LEVEL: RL= 73.77m

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: Dark brown, medium grained, moist, silty sand with gravel				
0.20		...bricks				
0.30		AUGER REFUSAL @ 0.30m depth on sandstone bedrock of at least very low strength				
1.00						
2.00						

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: JC

GROUND WATER OBSERVATIONS: None encountered during auger drilling

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Sara & Richard Crampton

DATE: 9/09/2021

BORE No.: 3

PROJECT: Alterations and Additions

PROJECT No.: 2021-198

SHEET: 1 of 1

LOCATION: 28 Ethel Street, Balgowlah

SURFACE LEVEL: RL=72.65m

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		FILL: Dark brown, medium grained, moist, silty sand				
0.80	SC	Clayey SAND: Very loose, grey, medium to coarse grained, moist to wet, possible extremely weathered sandstone		0.90		
1.00			D	1.00		
1.10	CL	Sandy CLAY: Firm, light grey, medium to high plasticity, moist to wet, extremely weathered sandstone	D	1.10		
1.20		...pale yellow brown, highly weathered sandstone				
		AUGER REFUSAL @ 1.2m depth on sandstone bedrock of at least very low strength				
2.00						

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: JC

GROUND WATER OBSERVATIONS: None encountered during auger drilling

REMARKS:

CHECKED: TMC

TEST PIT REPORT

Client: Sara & Richard Crompton

DATE: 9/09/2021

Test Pit No.: 1

Project: Alterations & Additions

PROJECT No.: 2021-198

SHEET: 1 of 1

Location: 28 Ethel Street Balgowlah

SURFACE LEVEL: RL=73.05m

Depth (m)	Description of Strata PRIMARY SOIL - strength / density, colour, grain size / plasticity, moisture, soil type incl. secondary constituents, other	Sampling		In Situ Testing		Photography
		Type	Results	Type	Results	
0.00	FILL: Dark brown, medium grained, moist, silty sand					
0.50						
	Clayey SAND: Very loose, grey, medium to coarse grained, moist to wet, extremely weathered sandstone					
0.65						
	SANDSTONE BEDROCK of at least very low strength					
1.00						
2.00						
3.00						

EXCAVATOR: N/A

DRILLER: AC

BUCKET SIZE: Shovel

LOGGED: JC

GROUND WATER: None encountered during excavation

CHECKED: TMC

TEST PIT SIDE WALL STABILITY: Good



DYNAMIC PENETROMETER TEST SHEET

CLIENT: Sara & Richard Crampton
PROJECT: Alterations and Additions
LOCATION: 28 Ethel Street, Balgowlah

DATE: 9/09/2021
PROJECT No.: 2021-198
SHEET: 1 of 1

Depth (m)	Test Location							
	DCP1	DCP2	DCP3	DCP4				
0.00 - 0.10	2	1	1	1				
0.10 - 0.20	2	0	0	1				
0.20 - 0.30	2	6	1	1				
0.30 - 0.40	6*B @0.33m	5*B @0.31m	2	1				
0.40 - 0.50			1	3				
0.50 - 0.60			1	12*B @0.53m				
0.60 - 0.70			1					
0.70 - 0.80			1					
0.80 - 0.90			1					
0.90 - 1.00			2					
1.00 - 1.10			11*B @1.1m					
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								

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

A	Landslip (Rockslide/topple <2m³) of bedrock around perimeter of excavation for garage structure due to poorly oriented defects		excavation up to 1.5m depth in sandstone bedrock, likely reducing to nil to the north	a) excavation 6.0m from existing house to be retained, impact 2% b) excavation 4.0m from the common boundary, secondary structure on common boundary, impact 10% c) excavation approximately 7.0m from boundary, house a further 1.0m, impact 1%		a) Person in house, 20hr/day avg. b) Person in secondary structure, 5hr/day avg. c) Person in house, 20hr/day avg.	a) Likely to not evacuate b) Likely to not evacuate c) Likely to not evacuate	a) Person in building, damage to building only b) Person in building, damage to building only c) Person in building, damage to building only	
			Possible - Rare	Prob. of Impact	Impacted				
		a) No.28 Ethel Street existing house	0.001	0.02	0.02	0.8333	0.75	0.1	2.50E-08
		b) No.26 Ethel Street secondary structure	0.001	0.10	0.05	0.2083	0.75	0.1	7.81E-08
		c) No.19 Whittle Avenue house	0.0001	0.01	0.01	0.8333	0.75	0.1	6.25E-10
B	Landslip of soils from swimming pool and retaining wall excavations (<0.5m³)		excavation up to 0.5m depth in fill soils	a) excavation to approximately 1.0m from the common boundary, secondary structure on boundary, impact 20%		a) Person in secondary structure, 5hr/day avg.	a) Unlikely to not evacuate	a) Person in building, damage to building only	
			Possible	Prob. of Impact	Impacted				
		a) No.26 Ethel Street secondary structure	0.001	0.10	0.30	0.2083	0.25	0.10	1.56E-07

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

* for excavation induced landslide 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 (Rockslide/topple <2m ³) of bedrock around perimeter of excavation for garage structure due to poorly oriented defects	a) No.28 Ethel Street existing house	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
		b) No.26 Ethel Street secondary structure	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
		c) No.19 Whittle Avenue house	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Landslip of soils from swimming pool and retaining wall excavations (<0.5m ³)	a) No.26 Ethel Street secondary structure	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%.

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.

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007
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		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10 ⁻³	5x10 ⁻³	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10 ⁻⁴		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10 ⁻⁵	5x10 ⁻⁵	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10 ⁻⁶	5x10 ⁻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%		Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	40%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%	1%	Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

- Notes:** (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.
- (3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.
- (4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

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.

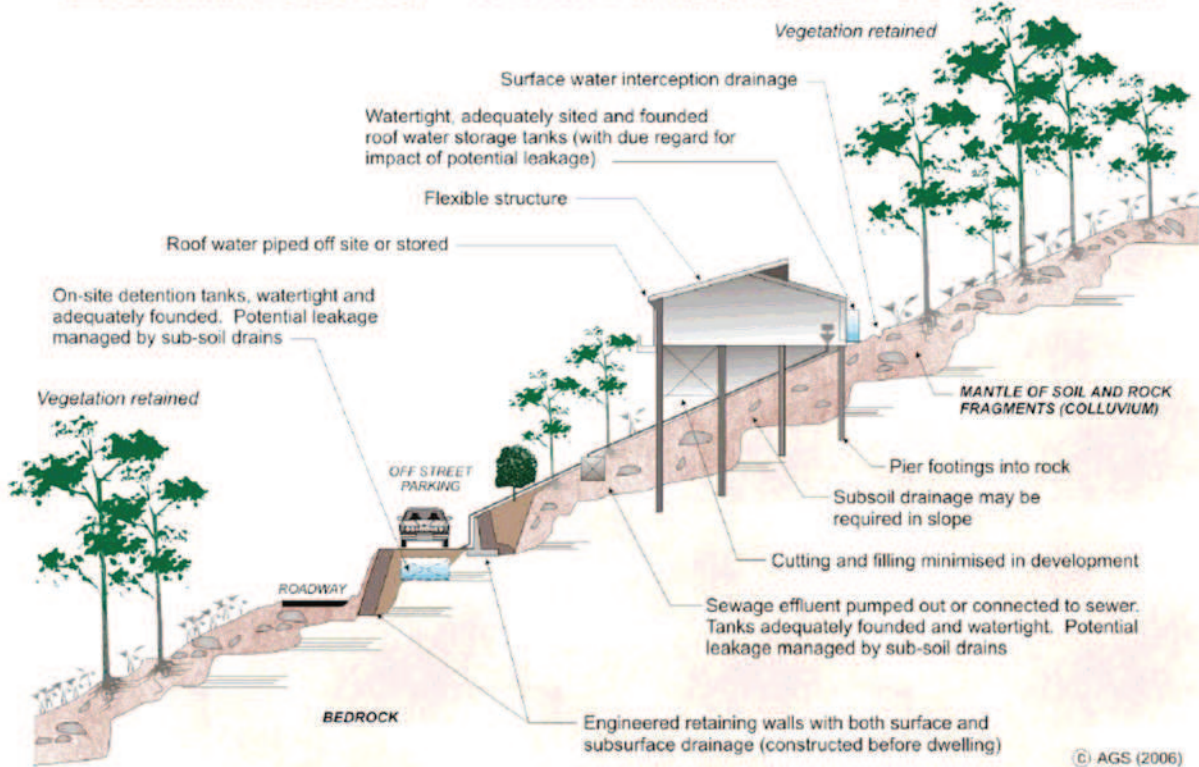
Appendix 5

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF **GOOD** HILLSIDE PRACTICE



EXAMPLES OF **POOR** HILLSIDE PRACTICE

