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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED NEW DWELLING

at

2 WYADRA AVENUE, CURL CURL, NSW

Prepared For

Mark Aubrey

Project No.: 2020-229.1

March, 2022

Document Revision Record

Issue No	**Date**	**Details of Revisions**
0	18th March 2022	Original issue

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GEOTECHNICAL REPORT FOR PROPOSED NEW DWELLING 2 WYADRA AVENUE, CURL CURL, NSW.

1. INTRODUCTION:

This report details the results of a geotechnical assessment carried out for the proposed construction of a new residential dwelling. The assessment was undertaken by Crozier Geotechnical at the written request of the client Mark Aubrey.

The proposed works are understood to involve the construction of a new one and two storey residential dwelling with attached carport structure. The proposed works appear to require excavation below existing ground levels to a maximum anticipated depth of approximately 3.0m for a lift shaft however the majority of the dwelling excavation will be <1.0m in depth.

Reference to the Northern Beaches Council – Warringah Local Environmental Plan 2011 and Landslip Risk Map (Map Sheet_010), identified that the site is located within land classified as Class 'B' detailed as "Flanking Slopes 5° to 25°" and also Class 'C' "Slopes >25°". The majority of No. 2 will be located within Class 'C' land.

Crozier Geotechnical Consultants previously supplied a report for DA submission related to the initial sub - division of No. 2 Wyadra Ave from the rear of No. 16 Ellen St and the construction of a new residential house (Geotechnical Site Investigation for Proposed New House at 16 Ellen Street, Freshwater, Project No. 2015-174, Dated: 25th September 2015), as well as a further report for DA submission related to the sub division boundary alteration and construction of a new residential house (Geotechnical Site investigation for Proposed Sub Division and Development at 2 Wyadra Avenue, Curl Curl, NSW, Project No. 2020-229, Dated: 12th November 2020)

This recent assessment involved:

- Review of previous investigation data from No. 16 Ellen St. and No. 2 Wyadra Ave. related to the previous sub-division and development proposals
- Review of updated development proposal plans

The following plans and diagrams were supplied and relied upon for assessment and reporting;

- Site survey by Stutchbury Jaques Pty Ltd, Reference No. 10358/19, Dated 22/06/2020
- Architectural Design drawings by Peter Stutchbury Architecture, Project Name: Ledge House, Drawing No.: 101-104, 200-202, 300-301, Dated: 14/12/2021

1.1 Proposed Works

It is understood that the proposed works will involve the construction of a new two storey residential dwelling within the site. This structure will require excavation for the ground floor, the lift shaft and for OSD stormwater tanks. These excavations will extend to maximum depths below ground level of 2.40m, 3.00m and 1.25m respectively. The excavations for the lift shaft and the OSD tanks will be setback from the shared western boundary by 0.90m, all other setbacks of note are in excess of 5.0m. Figure 1 below displays the footprint of the proposed dwelling overlain onto the supplies survey, highlighting excavation depths along the perimeter.

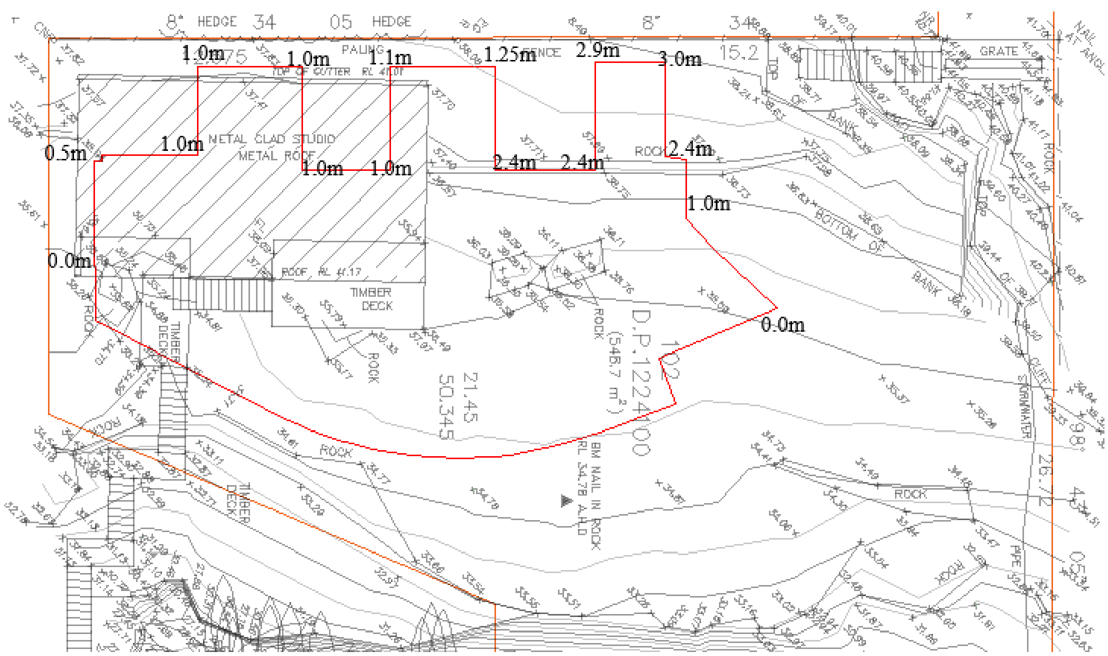

Figure 1: Outline of proposed works with approximate excavation depths

2. SITE FEATURES:

2.1. Description:

The property No. 2 Wyadra Avenue is a rear battle-axe style block with concrete access driveway passing down the northern side of No. 25 Loch Street to the main portion of the block, which extends east and south across the rear of No. 16. The main portion of the block is gently to moderately east dipping down to the crest of an up to 8.0m high cliff line that strikes north-south through the rear edge of the site and also through the neighbouring properties either side, including No. 14 Ellen Street. Below the cliff base the narrow remainder of the site is gently sloping extending into No. 16.

2.2 Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. This rock unit was identified in outcrops within 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. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint pattern, many cliff areas are undercut by differential weathering. Slopes below these cliffs are often steep 15 to 23° with moderately thick sandy colluvial soil profile that are randomly covered by sandstone boulders.

3. FIELD WORK

3.1 Methods:

Previous field investigation comprised a walk over inspection and mapping of No. 16 (which included the site) and adjacent properties on 28th August 2015 by a Geotechnical Engineer with photographic record of conditions and the drilling of two auger boreholes (BH1 – 2) using a hand auger to investigate sub-surface geology in areas of soil cover within the rear of the property. Dynamic Cone Penetrometer (DCP) testing was carried out adjacent to the boreholes and in other areas, in accordance with AS1289.6.3.3 – 1997, “Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer” to estimate near surface soil conditions and confirm depths to bedrock.

An additional, more recent assessment included an inspection of the rear of No. 14 and the site (No. 2) by a Senior Engineering Geologist on 10th November 2020.

3.2. Field Observations:

The site (No. 2) is located on the low east side of Wyadra Avenue, at mid-slope level with the majority of the property formed down slope to the rear of No. 25 Loch Street, which extends to the ridge crest. Access to the property is provided via a suspended concrete driveway with kerbs and drains that extends from the eastern end of Wyadra Ave and ends at the rear eastern boundary of No. 25.

Timber stairs provide access from the driveway down to the block, which is gently to moderately east dipping with soil slopes covered in low weed vegetation and numerous low (<1.0m) sandstone outcrops considered to be generally bedrock. The rear eastern side of the property is formed with outcropping medium strength sandstone that is considered to be bedrock that forms the crest of the cliff that extends across the site near the rear boundary. Adjacent to the western side of the outcrop is a shallow elongated depression that appears to define a defect within the bedrock below.

View of site (No. 2) from access driveway

Outcrop along eastern side of block

The cliff is up to 8.0m in height and formed through generally massive sandstone with few bedding or joint defects in the upper portion and with more bedding defects in the lower 3.0m. An undercut is formed at the base of the upper cliff, that extends up to 4.0m laterally into the cliff face towards the east. Some spalling at face of the cliff has also separated some small sections of rock.

Rear of No. 16 showing cliff line at rear of No. 2

Spalling and fractured rock near cliff crest.

At the rear north-west corner of the property there is another 4.0 – 5.0m high cliff line that extends from the northern neighbouring property to the sites north-west corner. This cliff contains a large overhang in the corner of the site which has been previously underpinned via several large mass concrete underpinning blocks. From this overhang the cliff curves to strike west and reduces in height with low cliffs and terraces stepping down in a southerly direction into the site.

Outcrop cliff and underpinning at north-wets corner of No. 2

No. 16 Ellen Street is located down slope within moderate east dipping topography from below the base of the lower cliff within No. 2. The property consists of a gently east sloping front half containing a three storey cement rendered residence with lawn and driveway at front. To the rear of the house, at upper level, there are a series of rock walls supporting terraced garden beds and lawns along with a timber decking.

The property No. 14 Ellen St contains a three storey cement rendered house structure on the front half within a gentle slope with a garden and lawn at upper level to the rear. The cliff line that cuts across No. 2 extends through this property and is generally near vertical on the north side and becomes partially fragmented and covered in vegetation to the south. Above the cliff this property is also gently to moderately sloping and contains a single storey raised cottage structure adjacent to the rear upslope western boundary. Below this cottage outcropping bedrock as a low (1.0m) cliff with terrace above was mapped which also continues across into No. 2.

Cliff to rear of No. 14

Undercut below cliff face as viewed from No. 14

The neighbouring property to the north and north-west contains a multi-level concrete structure related to Stewart House. This structure is located above the cliff line that enters the north-west corner of the site with the building located approximately 10m from the site boundary.

The neighbouring properties upslope (No. 23 and No. 25 Loch Street) both contain residential houses on the front halves of the blocks with a secondary dwelling and swimming pool at the rear of No. 23 and an undeveloped slope within the rear of No. 25.

4. COMMENTS:

4.1. Geotechnical Assessment:

The inspections identified shallow sandstone bedrock across most of No. 2 with extensive outcrops that also extend across neighbouring properties including the rear of No. 14 and No. 16.

The outcrop to the north-west of the site has been underpinned where it is undercut and is considered stable. The large sandstone cliff that extends across the rear edge of the site and extends through No. 14 to the south appears generally stable. However, the depression as seen to the rear of the outcrop in No. 2 along with the increased undercut dimension as seen from No. 14 warrant the installation of support systems to ensure long term stability. This could be undertaken following a more detailed geotechnical inspection/investigation via the installation of a blade wall along the boundary alignment between No. 14 and No. 16 or via two separate underpinning blocks, as seen within the overhang in the north-west corner of the site. This work should be engineer designed and installed by an experienced contractor.

The proposed works involve the construction of a new two storey dwelling with attached carport structure and will include a lift shaft and OSD water tanks. The lift shaft will require excavation to approximately 3.0m below existing ground levels whilst additional excavations will be required for the First Floor level of the dwelling however these will typically be <1.0m in depth.

The proposed works are located to the west of any potential instability in the existing cliff line therefore they will not be affected even if the interpreted/described cliff instability occurs. The development will involve an excavation, however based on site conditions the excavation will extend generally through outcropping sandstone bedrock of at least low to medium strength, therefore the potential to create instability is limited. The risk associated with the excavation can be managed and maintained within 'Acceptable' levels with negligible impact to adjacent properties or structures through geotechnical inspection during excavation and installation of support measures if determined as necessary by those inspections.

The management of ground vibrations will also be a critical aspect of the development works however through the use of suitable excavation equipment, which is anticipated due to the access limitations, the potential for creation of ground vibrations of detrimental level at adjacent structures is 'Very Low'. It is recommended that underpinning of the lower cliff occurs prior to rock excavation in the site.

4.2. Site Specific Risk Assessment:

Based on our assessment we have identified two geological/geotechnical hazards which need to be considered in relation to the existing site. The hazards are:

A. Landslip (rock topple 20m^3) from cliff due to undercut with rotation of cliff crest
B. Landslip (rock fall <1m^3) due to natural detachment of spalled section of rock on cliff
C. Landslip (rock slide/topple <2m^3) within bedrock due to excavation for lift

A qualitative assessment of risk to life and property related to these hazards is presented in Table A and B, Appendix: 2, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 3.

The **Risk to Life** from the hazards was estimated at up to **8.33 x 10^{-6}** whilst the **Risk to Property** was considered to be up to **'Moderate'**. The hazards were therefore considered to be marginally 'Acceptable' when assessed against the criteria of the AGS 2007. However, these hazards are considered without the implementation of the recommendations of this report. Therefore, where geotechnical inspection and installation of support systems along with the removal of the spalled rock occur, the risk levels will reduce significantly.

4.3. Design & Construction Recommendations:

4.3.1. New Footings:

The results of the investigations suggest that the site is underlain by sandy soil of variable but generally shallow depth overlying generally medium strength sandstone bedrock which also outcrops. The new house will be founded at the base of an excavation into bedrock which is expected to expose at least medium strength sandstone whilst the parking platform and carport will also be located over outcrop but this part of the site may also contain detached sections and boulders. Therefore, care will be required to ensure all footings are supported off insitu bedrock.

Footings founded in medium to high strength bedrock may be designed for a maximum allowable bearing capacity of 2000kPa, which is expected to be more than sufficient for the proposed works. Inspection by a geotechnical engineer of the base of the excavation and excavated footings is recommended to confirm bearing capacity and the insitu nature of the foundation. All footings should be founded off similar strength bedrock to prevent differential settlement, and this is expected to be achieved via strip/pad footings.

As footings are all expected to be founded within the bedrock it is considered a Class 'A' site as per the Australian Standard for Residential Slabs and Footings AS2870 – 2011.

Under the Australian Standard Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia the site Sub-soil classification would be B_e – rock site.

4.3.2. Excavation:

It is understood that the proposed works for the development include a localised excavation to 3.0m depth for the lift shaft, which will extend within 1.0m of the western rear boundary but will be well away from any other boundary. The excavation will therefore extend to within approximately 10m of the swimming pool within No. 23 Loch Street but will be very narrow and isolated. The excavation will be up to 3.0m depth in its western side however the depth will reduce quickly to nil in the east due to the natural ground surface slope. Additional excavations will be required for the OSD Water tanks as well as First Floor level however these are anticipated to be <1.0m depth.

The excavation will intersect shallow sand/fill underlain by low to medium strength sandstone with high strength rock expected in the excavation along with very high strength ironstone layers. Isolated low strength weathered or shale layers may also be present.

The excavation of soil and any extremely low strength, extremely weathered bedrock may be readily achieved using conventional earth moving equipment or hydraulic excavators with the assistance of ripping for the very low strength bedrock and thin ironstone bands. This method of excavation through soils and weathered bedrock will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.

The majority of the excavation will extend through medium strength sandstone bedrock with the possibility of high strength ironstone bands and occasional low strength shale and siltstone horizons. It will therefore require the use of rock excavation equipment (i.e. rock hammer / breaker / saw / grinder).

The selection of excavation machinery must take into account the following information: Vibration levels from rock breakers can be excessive (Peak Particle Velocities (PPV) greater than 50mm per second) and cause damage to adjacent structures, particularly if high to very high strength iron cemented sandstone bands or major south-east to north-east sub-vertical joints are encountered.

The Australian Standard (AS2187.2) makes reference to several standards used by British and United States authorities to assess damage as a result of ground vibrations from explosions, which produce transient vibration events. From these standards it can be seen that the values to create cosmetic damage, which is defined as hairline cracks (<0.1mm width) in AS2870-2011, Table: C1, are significantly higher than those at which humans find ground vibrations disturbing (>5mm/s). However, rock hammering produces intermittent vibrations which are more continuous than transient events, therefore lower damage thresholds would be expected.

Humans perceive ground vibrations at very low levels (0.5mm/s particle velocities) whilst steady state vibrations, as created by continuous uninterrupted rock hammering, are disturbing to persons above a value of 5mm/s PPV (Wiss 1981). This is especially the case where good relations with neighbours are not held.

It is therefore recommended that a **vibration limit (Maximum Peak Particle Velocity, PPV) of 5mm/s** be set at the founding level of all occupied neighbouring structures for all excavation work on this site with 8mm/s PPV recommended for un-occupied structures (i.e. swimming pools).

Vibration characteristics are site and equipment specific therefore vibration characterisation tests for any rock breaker/hammer will need to be undertaken using vibration monitoring equipment by a geotechnical specialist where a rock hammer >250kg 'dead-weight' is proposed for use. These tests are conducted prior to rock excavation work being carried out to define the equipment's characteristics, confirm appropriate buffer distances and site vibration characteristics.

Full time vibration monitoring may be required pending the results of the calibration testing. The geotechnical engineer should be notified of the proposed excavation equipment and methodology prior to excavation commencement. Visual monitoring at the commencement of the excavation and during vibration calibration of the equipment should take place via site inspection (Senior Engineering Geologist) to ensure that excavation techniques used by the operator keep vibration levels down to an acceptable level.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Hawkesbury Sandstone bedrock, to achieve 5mm/s PPV level of vibration the below hammer weights and buffer distances are required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.5m
400kg	4.0m
600kg	7.0m
900kg	10.0m

Rock sawing of the excavation perimeter 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.

Upper horizons in the bedrock may be detached along bedding and joint defects. Where these sections are impacted via rock hammering the opposite end, potentially located below neighbouring structures, will deflect more than expected. The rock sawing of the excavation perimeter prior to rock hammering will significantly reduce the risk of this hazard.

It is recommended that dilapidation surveys be undertaken 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.

Care will be required to ensure that excavated sections of rock are not allowed to travel down slope from the excavation and may require the placement of a catch fence or similar.

4.3.3. Excavation Support:

Recommended maximum batter slopes for excavation through fill and natural soils/rock on this site are presented below in Table: 1. Where these batters cannot be implemented then the excavation will require temporary support until permanent retaining walls can be completed. If suitable measures are not implemented then the stability of this excavation until permanent retaining walls are completed cannot be guaranteed. It is expected that these batters will be achieved through most of the site due to limited soil and shallow bedrock present.

Table 1 - Batter Slopes

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill and natural soils	1:1	2:1
Low strength or fractured sandstone	0.75:1	0.75:1
Medium strength, defect free sandstone	vertical	vertical

Vertical batters can be used where the excavation extends through medium to high strength sandstone bedrock which will generally remain self-supporting, though this will be dependent on weathering, the orientation of joints/defects and bedding. As such geotechnical inspection of excavated rock faces is required and may determine the need for installation of support systems, though this appears very unlikely based on site observations.

Based on the proximity of the cut to the rear boundary, any rock bolts may need to extend across into the neighbouring properties, therefore this option is not preferred. Design for the support structures will be on an individual basis as identified during inspections and should be undertaken by a geotechnical engineer to limit potential exposure at property boundaries.

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. Should boulders be identified near the crest of the excavation then these may need to be slightly over excavated, possibly underpinned and supported to ensure that no long term movement occurs that could create point loads on walls or rock face instability.

During excavation works, regular inspections should be undertaken by an appropriately qualified geotechnical professional to assess the subsurface conditions and advise on underpinning works, excavation batter slopes or rock face support requirements. It is recommended that these inspections be undertaken upon clearing of all soils from the bedrock surface and then once the rock excavation extends to its mid-level and then when it reaches its base.

4.3.4. Retaining Structures:

Whilst the medium strength bedrock will be self supporting, new retaining walls may be required as part of the proposed works. Where defects are identified in the geotechnical inspections then rock support may be required however this is considered very unlikely based on the scale of the proposed excavation. Backfilled retaining walls utilizing the coefficient for low strength sandstone may be implemented throughout as permanent support.

These structures will need to be "engineer designed" retaining wall systems designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures. Pressures acting on retaining walls can be calculated based on the parameters listed in Table: 2 for the materials likely to be retained.

Table: 2 - Retaining Structures Design Parameters

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (Ko)	
Fill (sandy) (loose to medium dense)	18	$\phi' = 29°$	0.35	0.52	2.85
Low strength rock (jointed)	22	$\phi' = 38°$	0.10	0.20	600 kPa

* Ultimate design values

In suggesting these parameters it is assumed that the retaining walls will be fully drained and it is envisaged that suitable subsoil drains would be 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.

Medium strength bedrock is suitable for the use of vertical galvanized steel dowels for lateral/rotation restraint for retaining wall systems. Medium strength bedrock is suitable for a grout/rock adhesion of 600kPa for fully grouted steel dowels and a footing bearing pressure of 2000kPa. The location of individual dowels should be assessed by the geotechnical engineer, they should generally extend a minimum of 600mm depth. However final design will be determined by the structural engineer.

4.3.5. Drainage and Hydrogeology:

The site is situated at mid-slope level within moderate sloping topography which contains extensive sandstone bedrock outcrops and cliff lines. Minor groundwater seepage was identified over the bedrock surface however no groundwater table will be intersected in the proposed development works therefore it is not expected to result in any significant impact to local hydrogeology.

Groundwater seepage can be expected at the soil rock interface and on geological defects within the bedrock. This seepage may be under slight artesian pressures due to water head from joints in the rock mass further upslope. 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. Drainage trenches/collection should also be placed in/over open defects in the bedrock exposed in the excavation. This may require some excavation in the joints. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to an engineered stormwater system and discharged to the Council's stormwater system off site.

Due to the shallow depth to bedrock, onsite disposal via infiltration into existing soil profile will not be possible, however a dispersion system may be suitable if combined with onsite detention or an alternative system designed by a Hydraulics Engineer.

4.4. Conditions Relating to Design and Construction Monitoring:

To allow Crozier Geotechnical Consultants to provide certification as part of construction, building and post-construction activity, it will be necessary for Crozier Geotechnical Consultants to;

1. Review the structural drawings and new stormwater disposal design for compliance with the recommendations of this report,
2. Conduct inspection of cliff overhang and installation of support systems as determined necessary along with confirmation of removal of spalled rock from cliff face.
3. Inspect all new footings and earthworks 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, as per Section 4.3,

5. CONCLUSION:

The site and adjacent properties are underlain by generally shallow layers of sandy soil overlying sandstone bedrock from ≤ 0.70m depth with outcropping sandstone bedrock of at least medium strength visible in numerous locations.

The proposed works involve the construction of a new two storey residential dwelling with attached carport and will include a lift shaft. The lift shaft will require a localized excavation to approximately 3.0m below existing ground levels and is situated within 1.0m from a shared boundary. The proposed First Floor Level and water tanks of the dwelling will require some additional excavations however these appear to be <1.0m in depth.

The inspections identified two natural landslip hazards related to the existing site and one potential hazard related to the proposed residential development.

Whilst the risk assessment identified some of the risk levels as being marginally “Acceptable’ when compared to the AGS criteria, it is recommended that an underpinning system be implemented to the overhang based on future detailed geotechnical inspection. Provided this occurs and the excavation/construction inspections detailed within this report the risk from all hazards will be well within the “Acceptable” risk level for the life span of the proposed development, taken as 100 years.

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Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

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

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

Standard Penetration Tests

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

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7
  as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

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

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

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

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

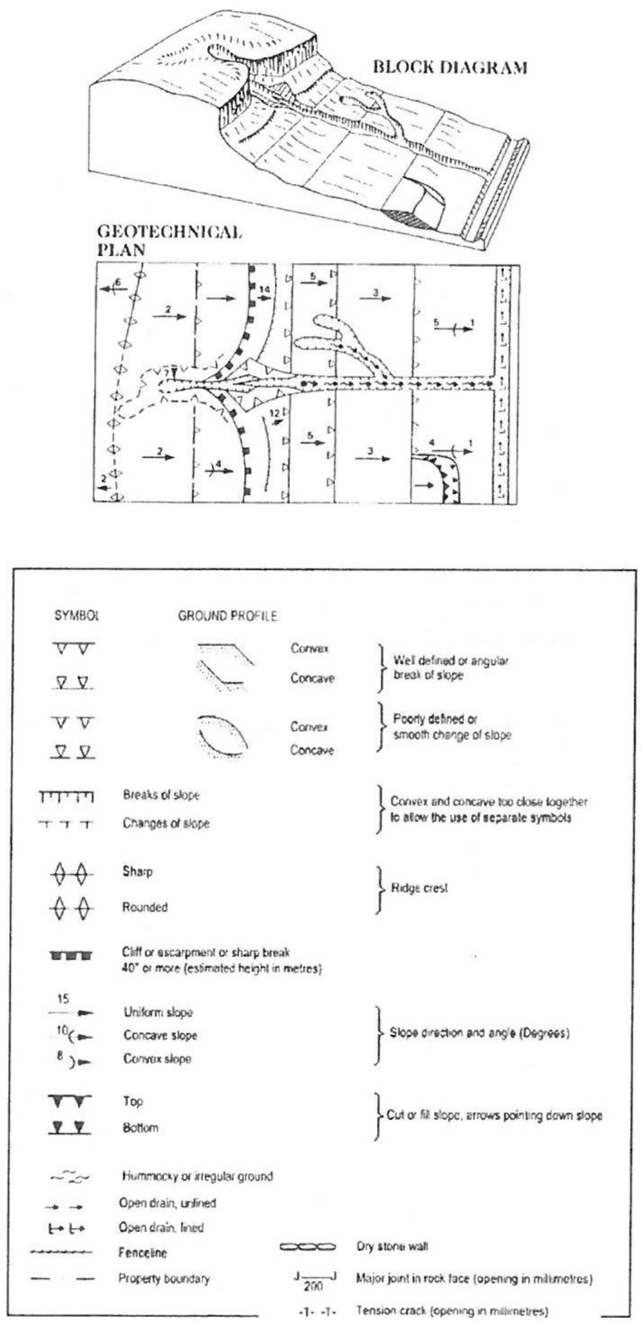
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

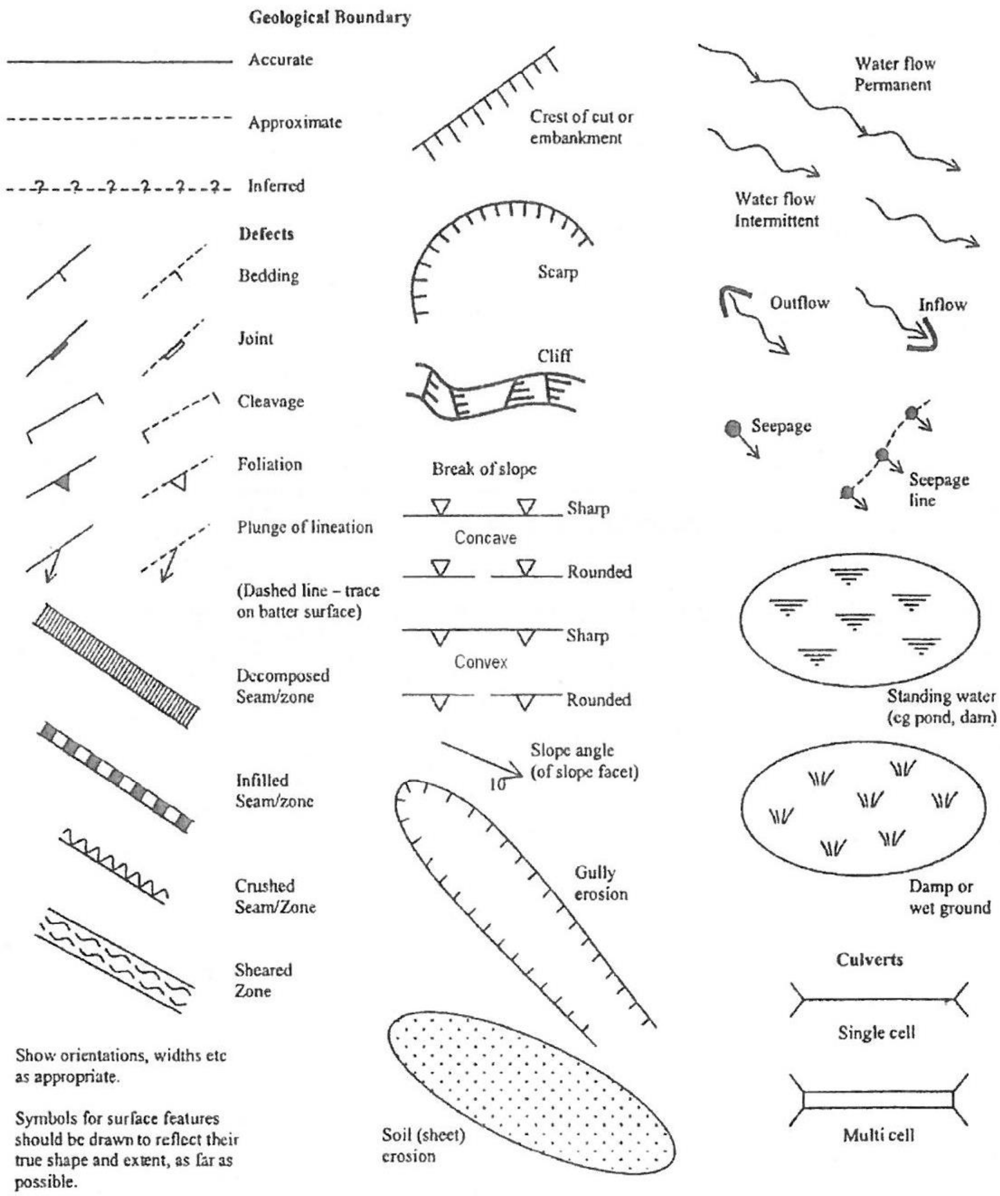
Site Inspection

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.



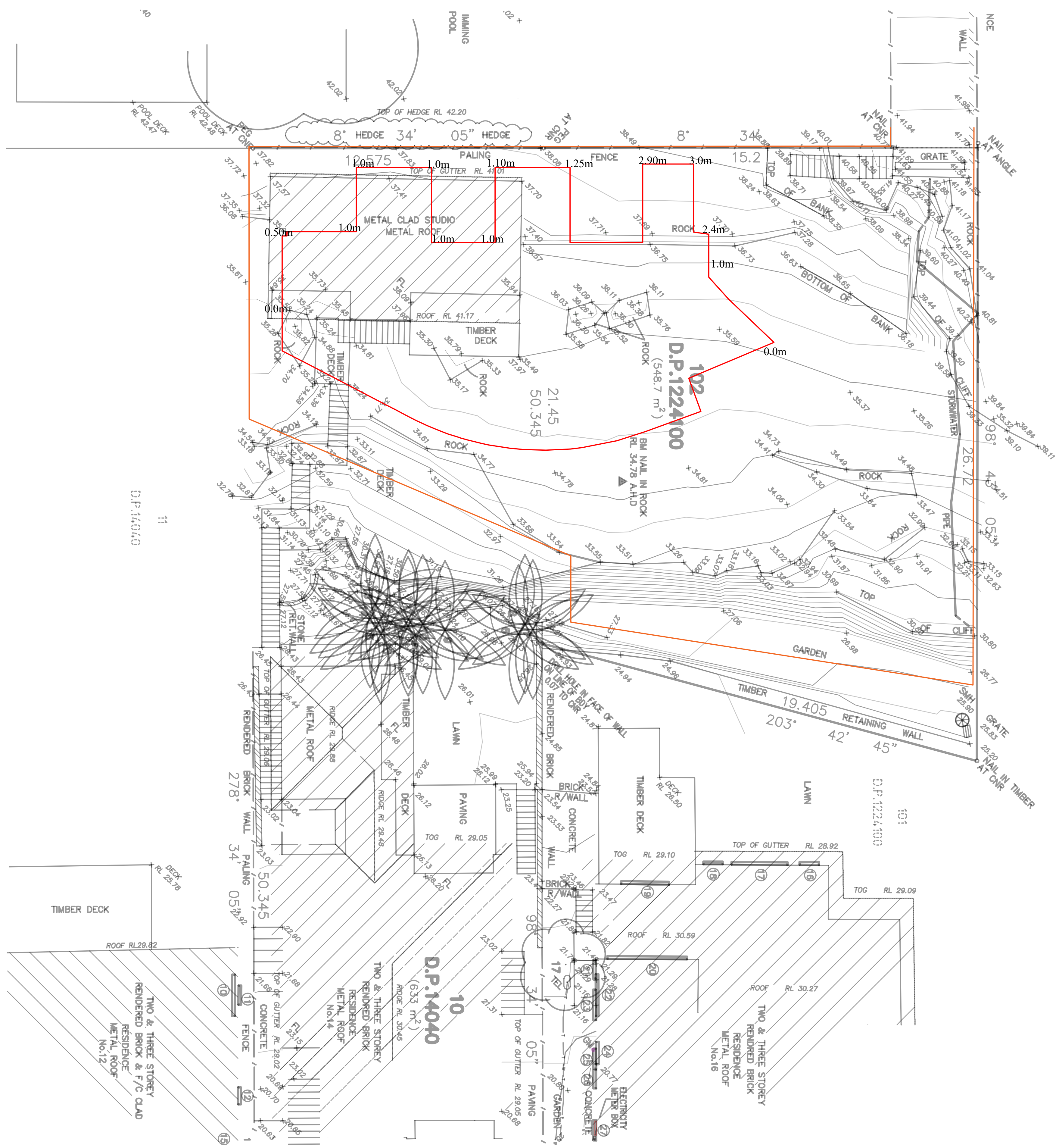
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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

FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

PROPERTY BOUNDARY

PROPOSED WORKS WITH ESTIMATED EXCAVATION DEPTHS

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	03/2022
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2020-299

PREPARED FOR:

ADDRESS:

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 (rock topple 50m³) from rotation of cliff edge due to overhang		Overhang of dimension when to potentially result in rotation	a) impact rear eastern 1/4 of property b) lawn and rear gardens, impact 20%		a) Person in rear edge of property following future development 1hrs/day avge. b) Person in lawn/garden 2hr/day avge.	a) Possible to not evacuate b) Possible to not evacuate	a) Person in open space, possible crush b) Person in open space, crushed	
			Possible	**Prob. of Impact**	**Impacted**				
		a) No. 2 Wyadra	0.001	1.00	0.25	0.0833	0.5	0.8	**8.33E-06**
		b) Rear Lawn No. 16 or No. 14	0.001	1.00	0.20	0.0417	0.5	1	**4.17E-06**
B	Landslip (rock fall <1m³) spall failure from cliff crest		Spalling is seperating sections of rock with weathering increasing potential for failure	a) rear lawn and garden at base of cliff, impact 0.5%		a) Person in garden 1hrs/day avge	a) Almost Certain to not evacuate	a) Person impacted	
			Likely	**Prob. of Impact**	**Impacted**				
		a) Rear Lawn No. 16	0.01	1.00	0.005	0.0417	1	1.00	**2.08E-06**
C	Landslip (rock slide/topple <5m³) due to excavation		Medium strength bedrock outcropping, shows no potentially de-stabilising defects	a) pool located approx. 5.00m from excavation, impact 5% b) rear garden slope located 1.0m from excavation, impact 1% c) house located at base of excavation, impact 5%		a) Person in pool 1hrs/day avge b) Person in garden 1hrs/day avge c) Person in house 20hrs/day	a) Almost Certain to not evacuate b) Possible to not evacuate c) Almost Certain to not evacuate	a) Structure minor impact only b) person in open space, unlikely buried c) person in structure, minor impact	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Rear pool of No. 23 Loch St	0.0001	0.01	0.050	0.0417	1	0.01	**2.08E-11**
		b) Rear of No. 25 Loch St	0.0001	0.20	0.010	0.0417	0.5	0.05	**2.08E-10**
		c) New Site Development	0.0001	1.00	0.050	0.8333	1	0.10	**4.17E-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 - Probaility 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 (rock topple 50m³) from rotation of cliff edge due to overhang	a) No. 2 Wyadra	Possible	The event could occur under adverse conditions 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 .	Moderate
		b) Rear Lawn No. 16 or No. 14	Possible	The event could occur under adverse conditions 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 .	Moderate
B	Landslip (rock fall <1m³) spall failure from cliff crest	a) Rear Lawn No. 16	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Low
C	Landslip (rock slide/topple <5m³) due to excavation	a) Rear pool of No. 23 Loch St	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		b) Rear of No. 25 Loch St	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		c) New Site Development	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 .	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

Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

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

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

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

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

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

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

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

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

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

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

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

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

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

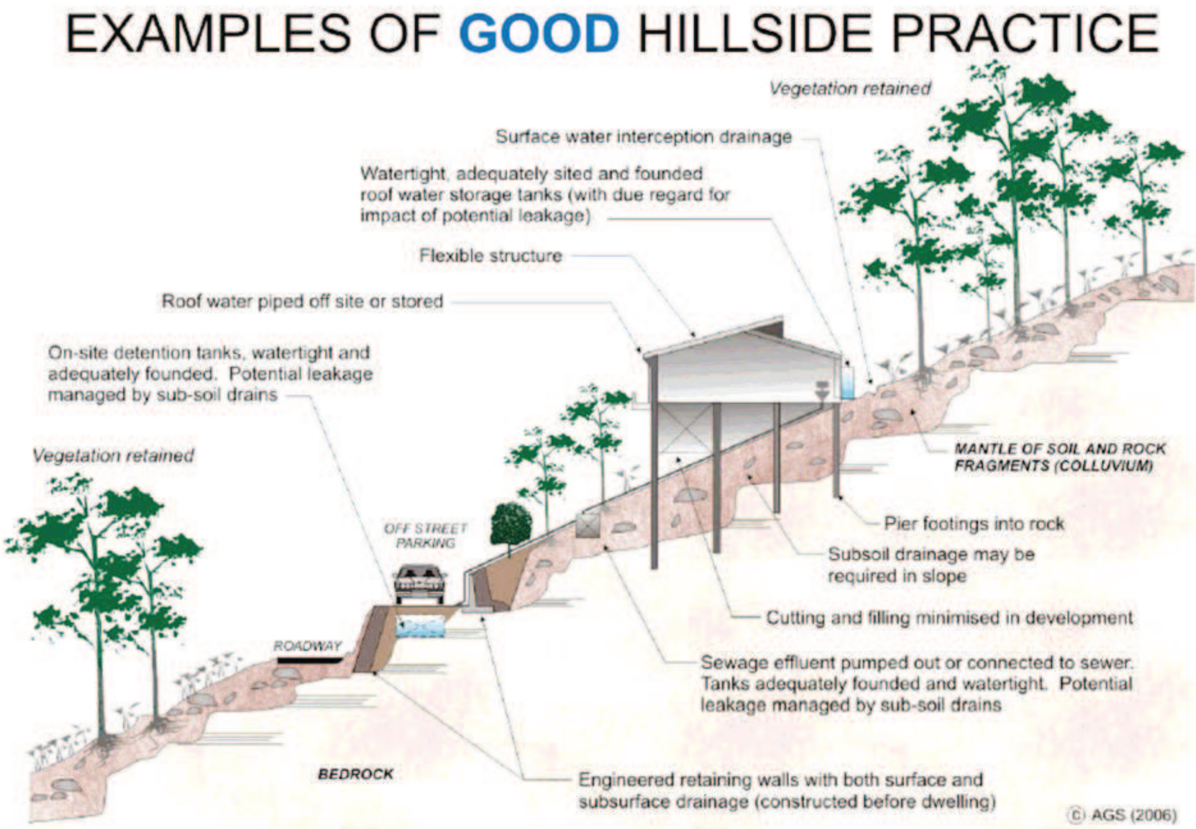
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
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
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
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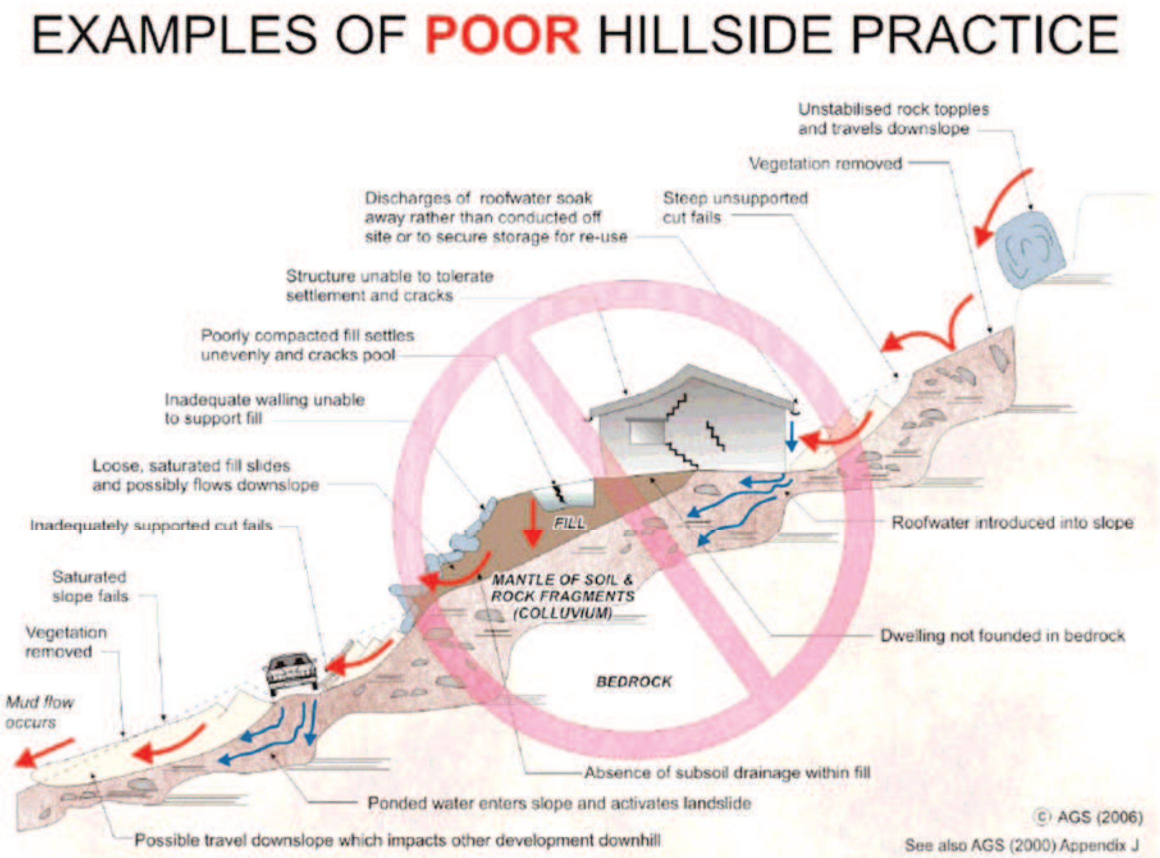
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
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See also AGS (2000) Appendix J