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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

52A CONSUL ROAD, BROOKVALE, NSW

Prepared For

Jacqueline and Michael Anderson

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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDDITIONS
52A CONSUL ROAD, BROOKVALE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation and assessment carried out for proposed alterations and additions at No. 52A Consul Road, Brookvale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of H & C Design on behalf of the clients Jacqueline and Michael Anderson.

The proposed works involve alterations and additions to the existing structures which includes partial demolition of the existing site dwelling and surrounding structures for a proposed extension, addition of a new First Floor level and construction of a new drainage void below the proposed dwelling. The proposed development will require a bulk excavation to 2.70m depth for drainage void within the Lower Ground Floor and 1.10m depth for proposed extension along with a drainage void within the Ground Floor.

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

This site is located within landslip risk Class 'B' (Southern portion) and Class 'C'(Northern portion) within the Landslip Risk Map _ Sheet LSR_008A. A review of the preliminary checklist and the proposed works identified that the Development Application (DA) involves works which exceed the preliminary assessment guidelines.

Therefore, a geotechnical report including a landslip assessment is required in support of the DA. This geotechnical report is provided for DA submission and details how the development may be achieved to ensure geotechnical stability and good engineering practice. It includes a risk assessment for both property and life as per the AGS March 2007 publication. This report also includes a description of site and subsurface conditions, in-situ test results, site mapping/plan, a geological section/model, a geotechnical assessment of the proposed works and recommendations for the proposed design and construction.

The geotechnical investigation included:

a) DBYD request, onsite review prior to site investigation.
b) Drilling of three boreholes using hand tools along with seven Dynamic Cone Penetrometer (DCP) tests to determine the subsurface geology, depth to bedrock, indication of underlying boulders and identification of groundwater,
c) Detailed geotechnical mapping of the entire site and adjacent land, with identification of geotechnical conditions and hazards including landslip related to the existing site and surroundings,
d) A photographic record of site conditions,
e) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional.

The following documents have been supplied by the Architect and relied upon for the investigation and reporting:

- Architectural Drawings – H & C Design, Job No.: 230402, Sheets: 01-13, Dated: August 2023
- Survey – Ramsay Surveyors, Ref: 8719, Sheet 1 of 1, Dated: 11/04/2023

2. PROPOSED DEVELOPMENT:

The proposed works involve alterations and additions to an existing site dwelling which includes an extension towards the north, west and east including construction of a new drainage void into the slope on the side of the Lower Floor Plan. The proposed development for the new Ground Floor will require a bulk excavation predominantly for the northern extension to approximately 1.10m depth and will extend 4.0m to the north. Furthermore, minor excavation will be required for the new footings for additional extension towards the western and eastern side of the dwelling. Excavation for the Lower Ground Floor drainage void will require excavation to approximately 2.70m depth.

An additional floor level is also proposed to be added to the dwelling. The upper level extension towards the north will require minor excavation for its new footings, with bulk excavation not required. It is further understood that the proposed development will require removal of two large trees located towards the high northeastern side of the site.

The new drainage void towards the northern and western side of the existing Lower Ground Floor will extend towards the north and west by approximately 0.85m and 0.60m respectively and will require a maximum bulk excavation to approximately 2.70m depth. The base excavation level for the drainage void is proposed to be within similar Finished Floor Level (FFL) as the existing/proposed Ground Floor Level (R.L 43.66).

Furthermore, the existing surrounding structures and landscape is proposed to be redesigned for the construction of a new outdoor terrace/BBQ area with retaining wall towards the northwest and for the installation of a new 5000 litre tank towards the low western side of the site which not does appear to require any bulk excavation.

3. SITE FEATURES:

3.1. Description:

The site is a broadly trapezoidal shaped block on the high northern side of Consul Road within very steep (≈33˚) southwest dipping topography and covers an area of 641.5m^2 as referenced from the provided survey plan. From the provided survey sketch the ground levels within the site are indicated as extending from a high of approximately RL52.93m towards the rear side of the dwelling to a low of approximately RL42.50m within the front eastern corner of the site, however the rear of the site was not surveyed and appears to rise significantly above this level to the base of a sandstone cliff line which rises an additional ~4.0m at the rear boundary.

From the provided survey plan, the site has a front southern boundary and rear northern boundary of 18.40m and 15.83m respectively whilst the side western and eastern boundaries are 35.82m and 45.195m respectively. An aerial photograph of the site location is shown below in Photograph 1, as sourced from NSW Government Six Map spatial data.

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

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet indicates that the site is underlain by Hawkesbury Sandstone which is of middle Triassic Age. The Hawkesbury Sandstone rock unit typically comprises of medium to coarse grained quartz sandstone with minor lenses of shale and laminate. Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium.

The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant joint defect orientations being sub-vertical and south-east and north-east striking. 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 (15º to 23°) and are randomly covered by sandstone boulders.

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

4. FIELD WORK:

4.1. Investigation Methods:

The field investigation comprised a geotechnical inspection of the site and adjacent land on 12th September 2023 by a Geotechnical Engineer. It involved a photographic record of site conditions as well as geotechnical assessment of the site and adjacent land with examination of existing site structures and inspection of neighbouring structures as well as soil slopes and boulder distribution.

It included the drilling of three boreholes (BH1-BH3) using a hand auger due to access restrictions to investigate sub-surface geology.

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

Dynamic Cone Penetrometer (DCP) testing was carried out from the ground surface adjacent to boreholes and at four additional locations. DCP tests were undertaken 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 indicate depths to bedrock/boulders.

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

4.2. Field Observations:

The site is situated on the high northern side of Consul Road within very steep south west dipping topography. Consul Road comprises a relatively flat bituminous sealed pavement that is separated from the site via a concrete kerb, gutter and south and southwest dipping grass lawn. There were no signs of excessive settlement or cracking observed within the structures adjacent to the site to suggest any significant movement or underlying geotechnical issues.

Minor sandstone rock outcrops were observed at the front western side of the grass lawn and were assessed as at least low strength sandstone bedrock without signs of major defects. Similarly, a very steep south dipping natural gully is located adjacent to the sites eastern boundary which consists of sandstone terraces, boulders and dense vegetation as seen in Photograph 2.

Photograph 2: View of the water course adjacent to the site, looking north/northwest

From the roadway, a gently southeast dipping concrete driveway extends up into the site providing access to a relatively flat concrete paved area that is anticipated to have been excavated/backfilled to level out.

The concrete pavement extends towards the north to a set of concrete steps providing access to the site's one and two storey split level brick and clad dwelling located towards the middle of the site. Concrete steps provide access to the site's rear concrete/tiled path and eastern grass lawn as seen in Photograph 3. The eastern grass lawn is anticipated to have some backfill due to its relatively flat topography. A brick retaining wall (1.0m high) and dry sandstone wall with dense vegetation is situated on the northern side of the stair retaining the eastern grass lawn which can also be seen in Photograph 3. The site dwelling appeared to be in a good condition with no signs of cracks or deformations.

Photograph 3: View of the site dwelling and its surrounds, looking northwest

The concrete/tiled path extends towards the western side of the site providing an access to the western grass lawn and garden area. It is understood that fill was placed within the existing lawn/garden area to level out the landscape and provide the existing appearance. The lawn dips south and forms a terraced landscape supported via 1.0m high timber log walls before flattening to a brick paved area as seen in Photograph 4. Furthermore, a terraced landscape with brick retaining wall (0.60m – 0.80m high) is situated towards the northern side of the grass lawn which extends towards the eastern side of the site. The high rear portion of the brick wall was observed to be constructed over a sandstone rock shelf which extends towards the west and into neighbouring property (52 Consul Road), however the shelf disappears as it extends toward the east within the site. The underlying sandstone shelf appeared to be stable with few joint defects, however the brick wall appeared to have been impacted severely due to the lack of drainage and age as seen in Photograph 5.

Photograph 4: View of the terraced garden and paved area, looking south

Photograph 5: View of the Sandstone outcrop and brick retaining wall, looking north

A steep south dipping dense vegetation area extends from the rear brick retaining wall to the northern boundary before a 4.0m high sandstone cliff line extends along the northern boundary and out towards the west as seen in Photograph 6. The cliff consists of sub-horizontal bedding defects and seams with sandy infill at approximately 1.50m intervals, however no geotechnical concern was encountered during our inspection. Sandstone rock shelf/clifflines up to 1.0m in height (dipping south by 15º - 20º) were observed within the rear dense vegetation area which appeared to be in a good condition and did not exhibit geotechnical concerns. Sandstone boulders varying from 3.0m^2 to 0.2m^2 are scattered throughout the rear area as seen in Photograph 7, however all the observed boulders appeared buried beneath the existing ground surface and did not exhibit any geotechnical concerns which could impact the future development within the site. A small scale drainage gully was observed towards the rear eastern side of the site which was observed to be dipping southeast merging with the gully located on the eastern side from the site boundary. The small gully within the site shows signs of only minor seepage and medium to large sandstone boulders.

Photograph 6: View of the rockface, looking north

Photograph 7: View of the sandstone boulder, looking south

The neighbouring property to the west (52 Consul Road) consists of a two storey rendered brick residence towards the middle of the property with a terraced landscape with dense steep dipping vegetation towards the rear and a concrete driveway and steep south dipping grass lawn towards the front. Sandstone shelves were observed towards the rear of the property's dwelling which extended into the site and can be seen in Photograph 8. The property levels are approximately 1.0m below the site with similar south dipping topography. The property dwelling and surrounding structures appeared to be in a good condition and did not exhibit signs of geotechnical concerns.

Photograph 7: View of the rock shelf (marked) within the neighbouring property (No. 52 Consul Road)

The neighbouring property to the north (54 Consul Road) consists of a three storey dwelling toward the middle of the property with concrete driveway, pavement and dense vegetation surrounding the property. The property is situated approximately 4.0m – 5.0m above site levels at the common boundary. The inspection was very limited to the property dwelling and structures, therefore the conditions within the property cannot be confirmed.

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

4.3. Ground Conditions:

The boreholes (BH1-BH3) were drilled adjacent to the existing site dwelling with all encountering a relatively variable layer of fill and residual soil before refusing within residual soils (BH1) and extremely weathered sandstone (BH2 and BH3) from ground surface.

Dynamic Cone Penetrometer (DCP) tests were carried out from the ground surface adjacent to the boreholes and at four additional locations with interpreted extremely weathered sandstone and very low strength sandstone encountered at a minimum depth of 0.20m (DCP4a) and at a maximum depth of 2.70m depth (DCP1)

Based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **FILL** – This layer was encountered in the boreholes extending from ground surface to a maximum depth of 1.10m(BH1). Where intersected in the borehole it was classified as a loose, dark grey, moist silty sand with sandstone gravels/cobbles.
- **RESIDUAL SOILS –** Natural clayey sand was intersected in all borehole locations from a minimum depth of 0.10m depth (BH2) and a maximum depth of 1.60m depth (BH1). It was identified as very loose gradually transition to medium dense, pale brown to pale grey, moist medium to coarse grained.
- **SANDSTONE BEDROCK – T**his unit was encountered within all the boreholes and from a minimum depth of 0.30m depth (BH2) and maximum depth of 1.0m (BH1). Similarly, sandstone shelves and cliff lines were observed predominantly towards the rear side of the site and within the neighbouring property towards the west (52 Consul Road). It consisted of bedding defects dipping towards the south by 15º – 20º. The outcrops were identified as least very low to low strength sandstone with only minor defects within the short chiff line. Overhangs, parting defects and clayey sand seams were observed over the face of the tall cliff line at northern boundary of the site, however the cliff was observed to be stable.

A free standing ground water table was not intersected within the investigation range. Significant seepage was not encountered in the test locations however signs of minor seepage were observed within the existing gully.

5. GEOTECHNICAL ASSESSMENT:

5.1. Comments:

The site investigation identified the presence of silty sand fill to a maximum depth of 1.10m depth (BH1, R.L 44.9) and a minimum depth of 0.10m depth (BH2, R.L 43.75) over residual soil. The residual soil which was identified as pale grey to pale brown clayey sand extended to a maximum depth 2.50m (DCP1, R.L 43.5) and a minimum depth of 0.30m (BH2, R.L 43.45). The residual soil transitions to extremely weathered sandstone bedrock within BH2 and BH3. The extremely weathered sandstone is interpreted to transition into at least very low strength sandstone which was observed predominantly towards the northern side of the site, while outcropping low strength sandstone was observed both within the site and in neighbouring properties. Due to the dipping topography of the site moderate seepage is anticipated across the soil and rock interface, particularly after a wet weather

The proposed works involve alterations and additions of the existing site dwelling including the partial demolition of the existing site dwelling for a proposed extension of the Ground Floor towards the north, west and east, the addition of a new First Floor level predominantly towards the north, construction of a new drainage void within the existing Lower Ground Floor and Ground Floor. The proposed development will require a bulk excavation to 2.70m depth for the drainage void within the Lower Ground Floor and 1.10m depth for the proposed extension for the Ground Floor Level.

To prevent future differential settlement all the new footings for the proposed extensions should be founded within the sandstone bedrock which may require piers in locations of deep soil. The underlying very low strength sandstone can provide an allowable end bearing capacity of ~800kPa, while footings in low strength sandstone may be designed with an allowable bearing capacity of 1000kPa.

Proposed extension for the new Ground Floor will require bulk excavation to 1.10m depth and is proposed to extend 4.0m to the north whilst the Lower Ground Floor works require excavation to approximately 2.70m depth. A sandstone rock shelf and shallow sandstone bedrock will be encountered during the excavation for proposed Ground Floor, similarly the proposed extension will require demolition of the existing brick retaining wall and sandstone bedrock. Due to access limitation towards the rear of the site, the proposed excavation will only be possible via hand tools or a small excavator. A new retaining wall which could be in the form of gabion basket wall, can be used to replace the existing brick wall which should be secured over sandstone bedrock encountered at a very shallow depth, due to site dipping topography and surface seepage.

The excavation towards the west is proposed to extend within an area containing existing timber log retaining walls and grass lawn and will encounter fill soils and residual soil to a maximum depth of 2.50m gradually transitioning to extremely weathered sandstone. Due to the proximity of the proposed excavation to the existing site dwelling a small-scale excavator or hand tools are recommended to be used for excavation works. Furthermore, the excavation work toward the north for the proposed Lower Ground Floor drainage void is proposed to extend beneath the existing Ground Floor Level by 0.80m laterally. This can undermine the existing footings and erode the surrounding soils. Therefore, a new structural wall should be constructed to underpin any exposed footings and propping should be installed prior to excavation to prevent damage to the structure.

Support such as shotcrete will be required to secure the exposed rock face within the excavation face of the drainage void as the excavation is expected to encounter extremely weathered to very low strength sandstone. A safe batter slope (1.0H:1.0V) appears to be achievable for the excavation toward the west, therefore pre-excavation support does not appear to be required. However, due to the presence of fill and residual soils a retaining structure in the form of block wall or concrete wall will need to be installed post excavation. Where at least Medium Strength bedrock with no poorly oriented defects is encountered, it will be free standing and can be excavated near vertically without the need for additional support measures. Where defects are encountered additional support may be required (i.e. rock bolts) to maintain stability.

In the case where pre-excavation support is required, a soldier pile wall would be suitable if taken down through any surficial soils and founded within competent bedrock.

Where possible the fill/soil and extremely low to very low strength bedrock can be excavated using conventional earthmoving equipment, however low to high strength sandstone bedrock will require the use of rock breaking equipment (e.g. rock hammers). The use of rock hammers can create ground vibrations which could damage the neighbouring, adjacent structures. Care will be required during the demolition, construction and excavation works to ensure the neighbouring properties, structures and services are not adversely impacted by ground vibrations. Small scale equipment (i.e. rock hammer ≤250kg) along with rock saw and a good excavation methodology can be used to maintain low vibration levels and avoid the need for full time vibration monitoring. Alternatively, the use of a rock grinder or hand tools can be used to maintain low ground vibrations. However, the scale of the excavation is expected to make this uneconomic.

It is recommended that vibration calibration tests be undertaken prior to bedrock bulk excavation where hammers >250kg are proposed to assess ground vibration characteristics. CGC should be consulted regarding excavation equipment and the contractor's methodology prior to the bulk excavation.

Two large trees toward the northeastern side of the site are to be removed as part of proposed development. Some small boulders (0.3m^2) were observed surrounding the trees which were partially buried beneath the surface. The removal of the trees will loosen the surrounding soils, however the surrounding boulders are not expected to be impacted as the boulders appear to be secured well beneath the ground surface. If possible, the surrounding boulders are recommended to be removed prior to removal of the tree and new saplings should be planted to replace the removed trees. A experienced geotechnical engineer or geologist should be consulted after the removal of the trees to inspect for any loose boulder that can cause a threat to the proposed development.

The remaining boulders onsite appeared to be relatively stable however, the bulk excavations proposed will likely impose risk onto the stability of such boulders. Therefore, careful excavation methodology and frequent geotechnical inspections by CGC to ensure boulder stability will be required.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on 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 hand tools. This test equipment provides limited data from small, isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations and below DCP refusal depths. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

5.2. Site Specific Risk Assessment:

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

A. Landslip (Earthslide) of surficial soil excavation works around perimeter of excavation (<1.50m^3)
B. Collapse (Rockslide) of bedrock around perimeter of excavation (<2m^3)

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

The **Risk to Life** from **Hazard A** was estimated to be up to **5.63 x 10^{-7}** for a single person, whilst the **Risk to Property** was considered to be **'Very Low'** for site dwelling.

The **Risk to Life** from **Hazard B** was estimated to be up to **1.69 x 10^{-7}** for a single person, whilst the **Risk to Property** was considered to be **'Very Low'** for site dwelling.

The assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including detailed investigation, regular geotechnical mapping of the excavation and installation of determined support systems in timely manner, the likelihood of any failure becomes 'Rare' and as such the consequences and risk can be further reduced when assessed against the criteria of the AGS.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' if footings bear onto bedrock
Type of Footing	Strip/Pad/Piles
Sub-grade material and Maximum Allowable Bearing Capacity	- VLS Sandstone: 800kPa - LS Sandstone: 1000kPa - MS Sandstone: 2000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – rock site

Remarks:

*Higher bearing pressures require further geotechnical testing including cored boreholes

All footings for the proposed structure should be founded off material of similar strength to prevent differential settlement. The structure should be founded off bedrock of at least low strength.

Ancillary structures founded in soil will be susceptible to soil creep.

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:

Table 1: Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	54 Consul Road	Site Dwelling	2.70m	>20.0	Dwelling +10.0
South	Common Driveway	Road Reserve	2.70m	13.0	Road Reserve +20.0
East	Common Driveway/ Consul Road	Driveway	2.70m	3.80	Road Reserve +10.0
West	52 Consul Road	Lawn	2.70m	4.50	Dwellings setback >10m

Type of Material to be Excavated	Fill and residual soils to depths ≤ 2.50m.

	Sandstone bedrock, Extremely weathered and VLS-MS

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

Material	Safe Batter Slope (H: V) Short Term/Temporary	Safe Batter Slope (H: V) Long Term/Permanent
Fill/residual soils/extremely weathered sandstone	1:1*	2:1*
Very Low to Low strength or fractured bedrock	0.5:1*	1:1*
Medium Strength (MS), defect free bedrock	Vertical	Vertical*

* Dependent on assessment by engineering geologist/geotechnical engineer

Remarks:

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

Equipment for Excavation	Fill/natural soils/extremely weathered sandstone	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-HS bedrock	Rock hammer, saw or grinder

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

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Neighbouring houses = 5mm/s vector sum
Vibration Calibration Tests Required	If medium to high strength bedrock is exposed and large rock hammers (≥250kg) are proposed for use.
Full time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Upon initial demolition and clearing of bedrock surface • Inspection of any temporary and permanent batter slopes • Following removal of trees to north east

	• At 1.50m depth intervals of excavation through bedrock • Where ground conditions are exposed that differ to those than expected • At completion of the excavation and for footings.
Dilapidation Surveys Requirement	Recommended within 10m of excavation perimeter to avoid 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. It is recommended that a drainage excavation extend to below floor slab levels to reduce the potential for dampness problems in the completed structure.	

5.3.3. Retaining Structures:	
Required	New retaining structures are required as part of the proposed development
Types	Gabion basket wall, Shotcrete /concrete block walls post excavation where temporary batters are possible, Bored pier pre-excavation support where temporary batters unachievable. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure or Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill/Natural Soils	18	$\phi' = 30°$	0.41	0.50	N/A
Extremely weathered – VLS bedrock	22	$\phi' = 38°$	0.32	0.20	200kPa
LS or fractured bedrock	23	$\phi' = 40°$	0.25	0.36	400kPa

Remarks: In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.
Retaining structures near site boundaries or supporting existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	At soil - rock interface and surface seepage
Site Location and Topography		High northern side of the road within very steep south dipping topography
Remarks: Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow down slope. This can result in relaxation of excavation faces causing instability prior to installation of permanent retention systems. Therefore, excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional. A stormwater diversion drain should be installed upslope of excavation crests to intercept stormwater runoff and prevent erosion and softening of the excavation faces. An excavation trench should also be installed at the base of excavation cuts to below floor slab levels to reduce the risk of long term dampness. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer.		

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 the structural drawings, for inclusion of the recommendations of this report,
2. Conduct inspections as per the recommendations of Section 5.3 in this report

3. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
4. Inspect the completed development to ensure all retention and stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. CGC cannot complete the certification (Form 3) if it has not been called to site to undertake the required inspections.

6. CONCLUSION:

The site investigation identified the presence of a variable layer of fill soils, residual clayey sand soils, extremely weathered sandstone and subsequently sandstone bedrock. The bedrock was observed outcropping predominantly toward the northern side of the site and was predominantly assessed as at least to Very Low Strength (VLS) to Low Strength (LS) sandstone. A freestanding groundwater table was not encountered and is not anticipated within 10m of existing site levels based off topography and elevations, however moderate seepage is expected to be encountered during any excavation work at the soil and rock interface and on defects in the bedrock.

The proposed works involve alterations and additions to the existing site structure including the addition of new First Floor Level and extension of the dwelling towards the north, west and east which will include construction of a new drainage void within Lower Ground Floor and Ground Floor. The proposed development will require a bulk excavation to 2.70m depth for the drainage void within Lower Ground Floor and 1.10m depth for proposed extension and the drainage void within Ground Floor

Large to small sandstone boulders were observed toward the northern side of the site which did not pose an immediate threat to the proposed development work. However, inspection will be required post excavation work to ensure the boulders will remain stable.

The proposed excavation will extend through fill, and potentially residual soils prior to intersection of sandstone bedrock, initially anticipated to be predominantly LS however expected to grade to MS and will likely feature some seam and joint defects.

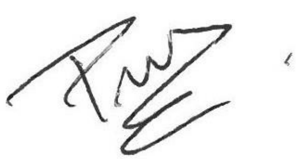
Due to excavation depths and setbacks, it is envisioned that safe temporary batter slopes will be feasible along the majority of the excavation edges. However, excavation within the existing Lower Ground Floor may contribute to undermining of the existing Ground Floor footings. Therefore, underpinning will be required to support the existing structure and appropriate propping works will be required prior to excavation work within the Lower Ground Floor

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

The bedrock has the significant potential for weathered seams and defects as well as detached section/boulders which could impact the stability of the excavation. 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.

The risks associated with the proposed development as well as the existing site conditions can be maintained within an ‘Acceptable’ Risk Management Criteria provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared By:



Prince Shrestha
Geotechnical Engineer
B.E. (Hons.) Civil

Reviewed By:

Troy Crozier
Principal Engineering Geologist
MIEAust., MAIG, RPGeo
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. V. Gardiner & R. Dackombe 1983, “Geomorphological Field Manual” by George Allen & Unwin
5. V. Gardiner & R. Dackombe 1983, “Geomorphological Field Manual” by George Allen & Unwin.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

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

Site Anomalies

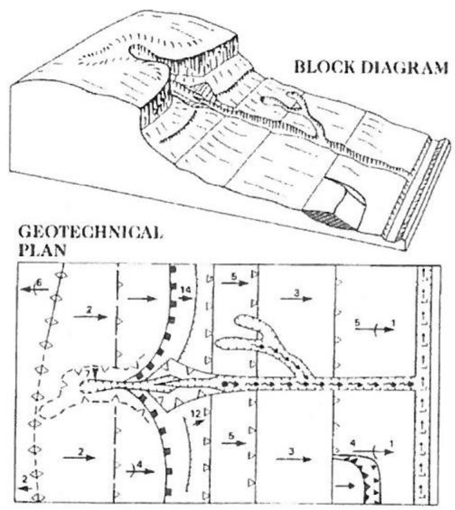
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

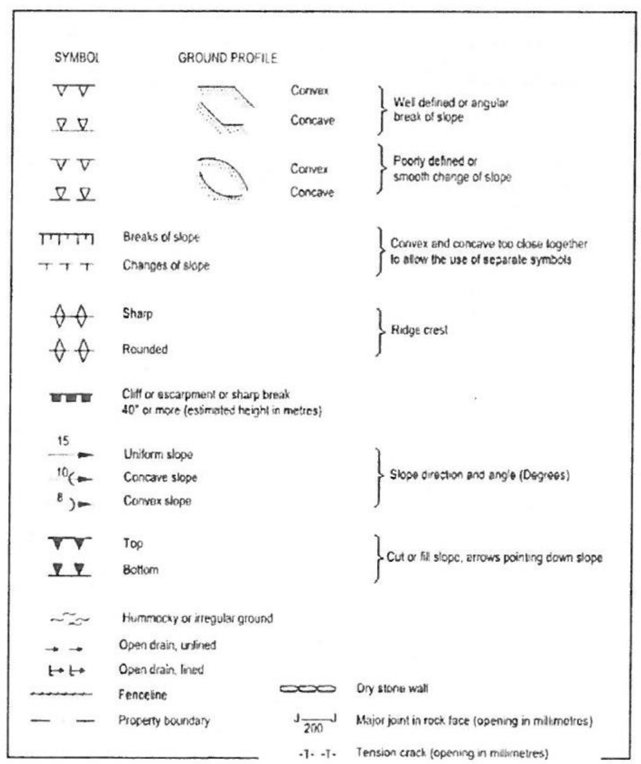
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

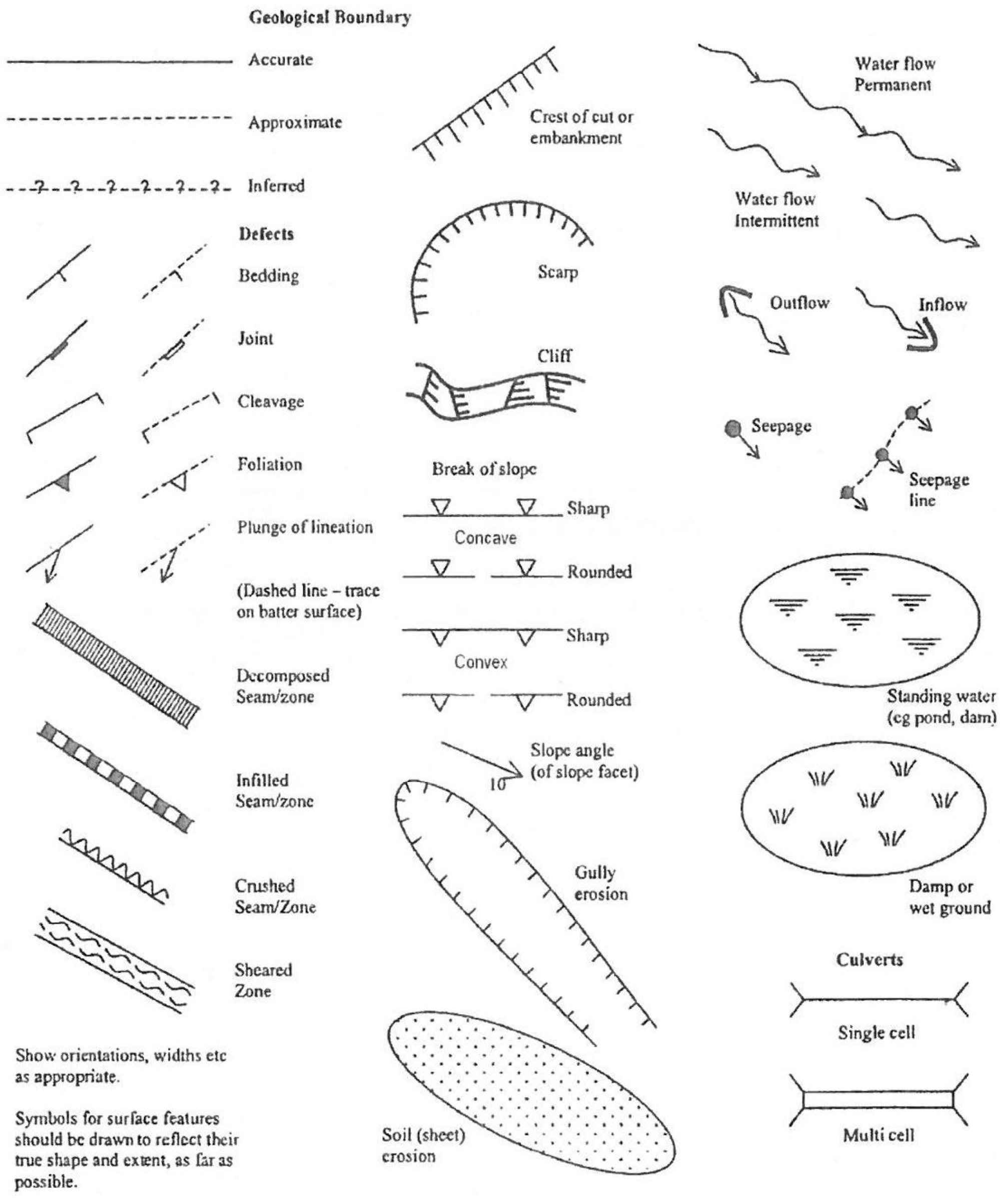
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





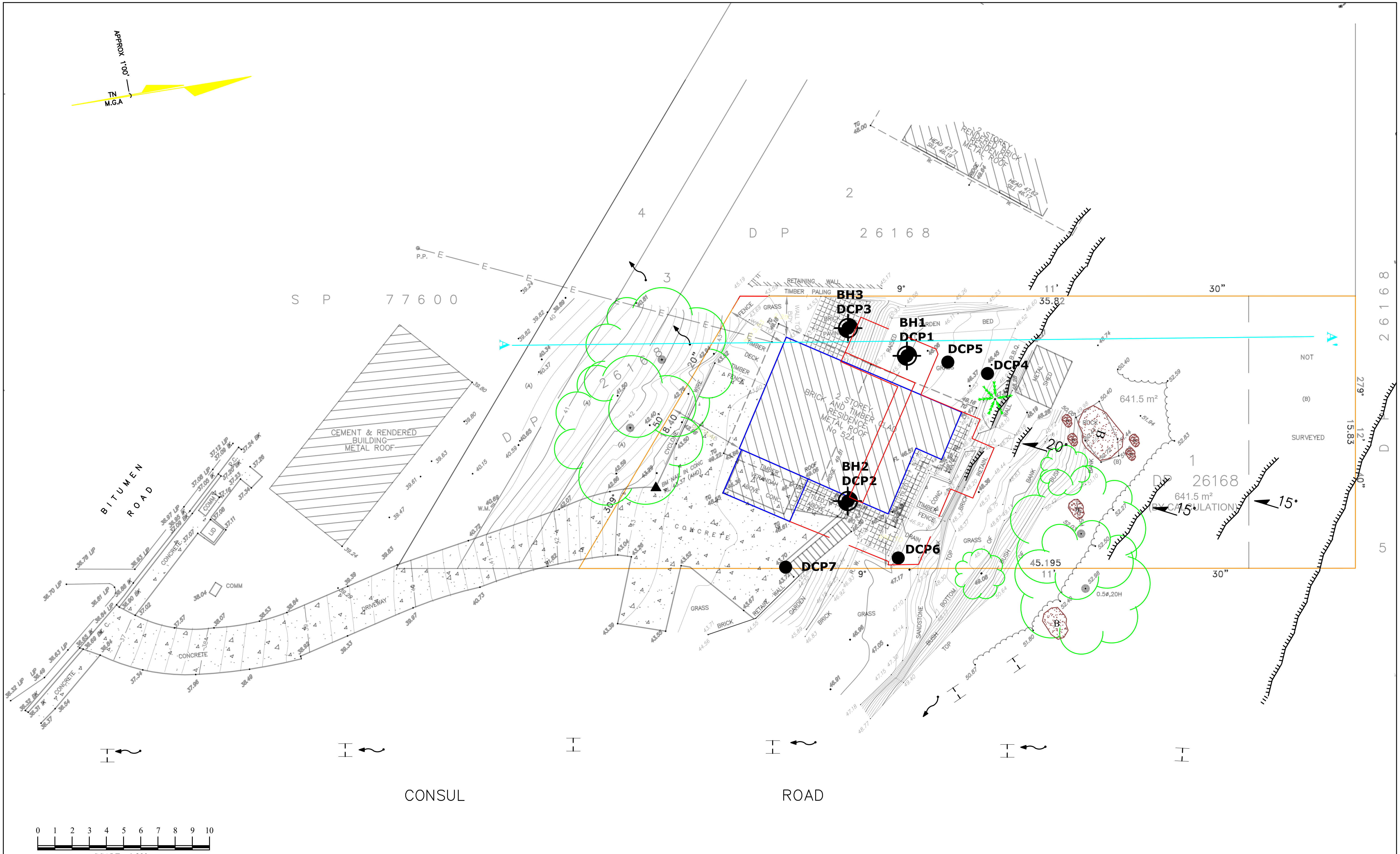
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN & TEST LOCATIONS

FIGURE 1.

CROZIER GEOTECHNICAL CONSULTANTS

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

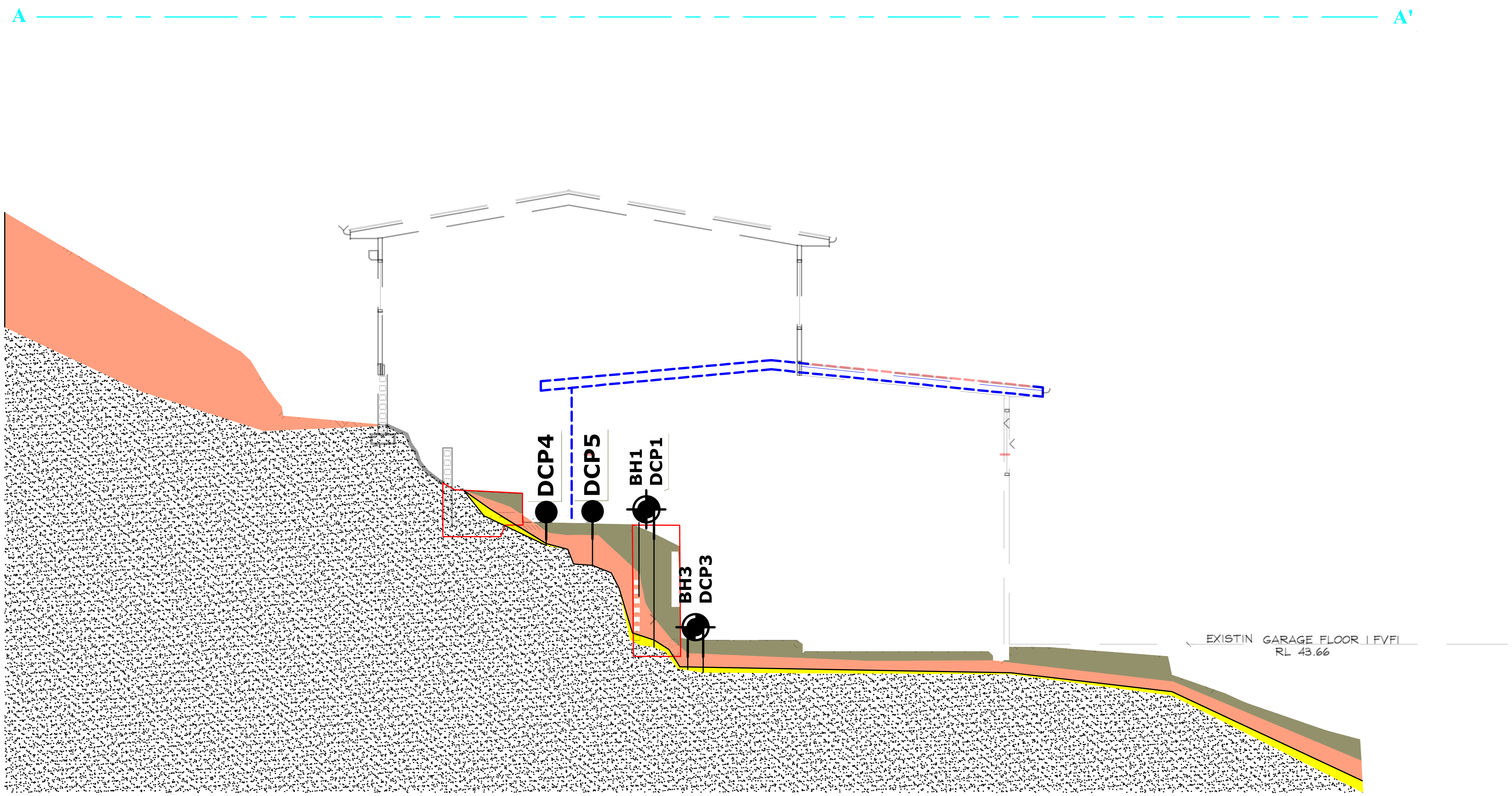
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP	DYNAMIC CONE PENETROMETER
	CLIFF LINE
	GULLY
	SURFACE WATER FLOW
x°	SLOPE ANGLE
B	SANDSTONE BOULDER
	PROPOSED EXCAVATION
	EXISTING STRUCTURES
A — A'	SECTION LINE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	09/2023
APPROVED BY:	TMC
DRAWN BY:	PS
PROJECT:	2023-183

PREPARED FOR:
Jacqueline and Michael Andreson

ADDRESS:
52A Consul Road, Brookvale



0 1 2 3 4 5

SCALE: 1:100

SITE SECTION

FIGURE 2.

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
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Phone: (02) 9939 1882
Fax: (02) 9939 1883

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

LEGEND

- BH / DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
- DCP — DYNAMIC CONE PENETROMETER
- EXISTING STRUCTURES
- PROPOSED EXCAVATION
- TOPSOIL/FILL
- EXTREMELY WEATHERED SANDSTONE
- SANDSTONE BEDROCK
- RESIDUAL SOIL
- A —— A' SECTION LINE

SCALE:	1:100 @ A3
DRAWING:	FIGURE 2
DATE:	09/2023
APPROVED BY:	TMC
DRAWN BY:	PS
PROJECT:	2023-183

PREPARED FOR:
Jacqueline and Michael Andreson

ADDRESS:
52A Consul Road, Brookvale

BOREHOLE LOG

CLIENT: Mike Anderson **DATE:** 12/09/2023 **BORE No.:** 1

PROJECT: Alterations and Additions **PROJECT No.:** 2023-183 **SHEET:** 1 of 1

LOCATION: 52A Cousul Road, Brookvale, NSW **SURFACE LEVEL:**

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		FILL: Dark grey, medium grained, moist silty sand fill with fine gravels				
0.20		… becoming brown to reddish brown with weathered sandstone and ironstone gravels				
1.00		… becoming dark grey trace silt				
1.10		Clayey SAND: Very loose, dark grey, medium grained, moist to wet				
1.20		… becoming medium dense		1.20		
			D			
1.30		… becoming brown		1.30		
1.40		… becoming pale brown to reddish brown coarse grained (interpreted extremely weathered sandstone)		1.40		
			D	1.50		
1.60		AUGER REFUSAL at 1.60m on interpreted extremely weathered sandstone				

RIG: Not Applicable DRILLER: J.H

METHOD: Hand Auger LOGGED: P.S

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mike Anderson **DATE:** 12/09/2023 **BORE No.:** 2

PROJECT: Alterations and Additions **PROJECT No.:** 2023-183 **SHEET:** 1 of 1

LOCATION: 52A Cousul Road, Brookvale, NSW **SURFACE LEVEL:**

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 grey, fine to medium grained, moist silty sand fill with gravels				
0.10		Clayey SAND: Very dense, pale grey with pale brown bands, medium grained, moist (interpreted extremely weathered sandstone)				
0.30		AUGER REFUSAL at 0.30m depth on intrepreted extremely weathered sandstone bedrock				

RIG: Not Applicable DRILLER: J.H

METHOD: Hand Auger LOGGED: P.S

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mike Anderson **DATE:** 12/09/2023 **BORE No.:** 3

PROJECT: Alterations and Additions **PROJECT No.:** 2023-183 **SHEET:** 1 of 1

LOCATION: 52A Cousul Road, Brookvale, NSW **SURFACE LEVEL:** R.L 43.65

Depth (m) 0.00	Classification	**Description of Strata** **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	**Tests**	**Type**	**Results**
0.05		BRICK (0.05m)				
0.30		FILL: Dark grey, medium grained, moist silty sandy fill with fine gravels				
0.40		Clayey SAND: Very loose, pale brown to reddish brown, medium to coarse grained, moist clayey sand				
0.65		… becoming medium dense				
		AUGER REFUSAL at 0.65m depth on interpreted extremely weathered sandstone bedrock				

RIG: Not Applicable DRILLER: J.H

METHOD: Hand Auger LOGGED: P.S

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Mike Anderson **DATE:** 12/09/2023

PROJECT: Alterations and Additions **PROJECT No.:** 2023-183

LOCATION: 52A Cousul Road, Brookvale, NSW **SHEET:** 1 of 1

	Test Location									
Depth (m)	1	2	3	4	4a	5	6	7		
0.00 - 0.10	1	6	1	2	2	2	2	3		
0.10 - 0.20	0	14	5	5	20 (B) at 0.20m	4	4	12		
0.20 - 0.30	1	17	2	10		2	3	10 (B) at 0.20m		
0.30 - 0.40	1	20	1	14		3	7			
0.40 - 0.50	2	17	4	18		3	4			
0.50 - 0.60	1	19	5			3	7			
0.60 - 0.70	1		1 (B) at 0.70m			17	9			
0.70 - 0.80	1					21	5 (B) at 0.72m			
0.80 - 0.90	1					15 (B) at 0.95m				
0.90 - 1.00	1									
1.00 - 1.10	1									
1.10 - 1.20	1									
1.20 - 1.30	6									
1.30 - 1.40	4									
1.40 - 1.50	2									
1.50 - 1.60	4									
1.60 - 1.70	4									
1.70 - 1.80	3									
1.80 - 1.90	3									
1.90 - 2.00	3									
2.00 - 2.10	3									
2.10 - 2.20	5									
2.20 - 2.30	6									
2.30 - 2.40	8									
2.40 - 2.50	7									
2.50 - 2.60	27									
2.60 - 2.70	27									
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

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

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

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

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

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (Earth slide $<1.5m^3$) of surficial soils excavation works		Soil excavation down to ≤1.50m depth	a) 1.20m depth soil excavation 1.40m away from the dwelling		a) Person in house 18hrs/day	a) Unlikely to not evacuate	a) Person in house, unlikely to be buried, structure impact only	
			Likely	**Prob. of Impact**	**Impacted**				
		a) No. 52A Counsul Road (Site Dwelling)	0.01	0.60	0.01	0.75	0.25	0.05	**5.63E-07**
B	Collapse (rock slide $<2m^3$) in excavations		a) Bedrock excavation down to ≤3.0m depth	a) 3.0m depth rock eacavation 1.40m away from the dwelling		a) Person in house 18hrs/day	a) Possible to not evacuate	a) Person in house, unlikely to be buried, structure impact only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) No. 52A Counsul Road (Site Dwelling)	0.001	0.90	0.01	0.75	0.50	0.05	**1.69E-07**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probability of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)
* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (Earth slide <1.5m3) of surficial soils excavation works	a) No. 52A Counsul Road (Site Dwelling)	Possible	The event could occur under adverse conditions over the design life.	insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Collapse (rock slide <2m3) in excavations	a) No. 52A Counsul Road (Site Dwelling)	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

* 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%.

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event.
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with Councils Risk Management Policy.

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

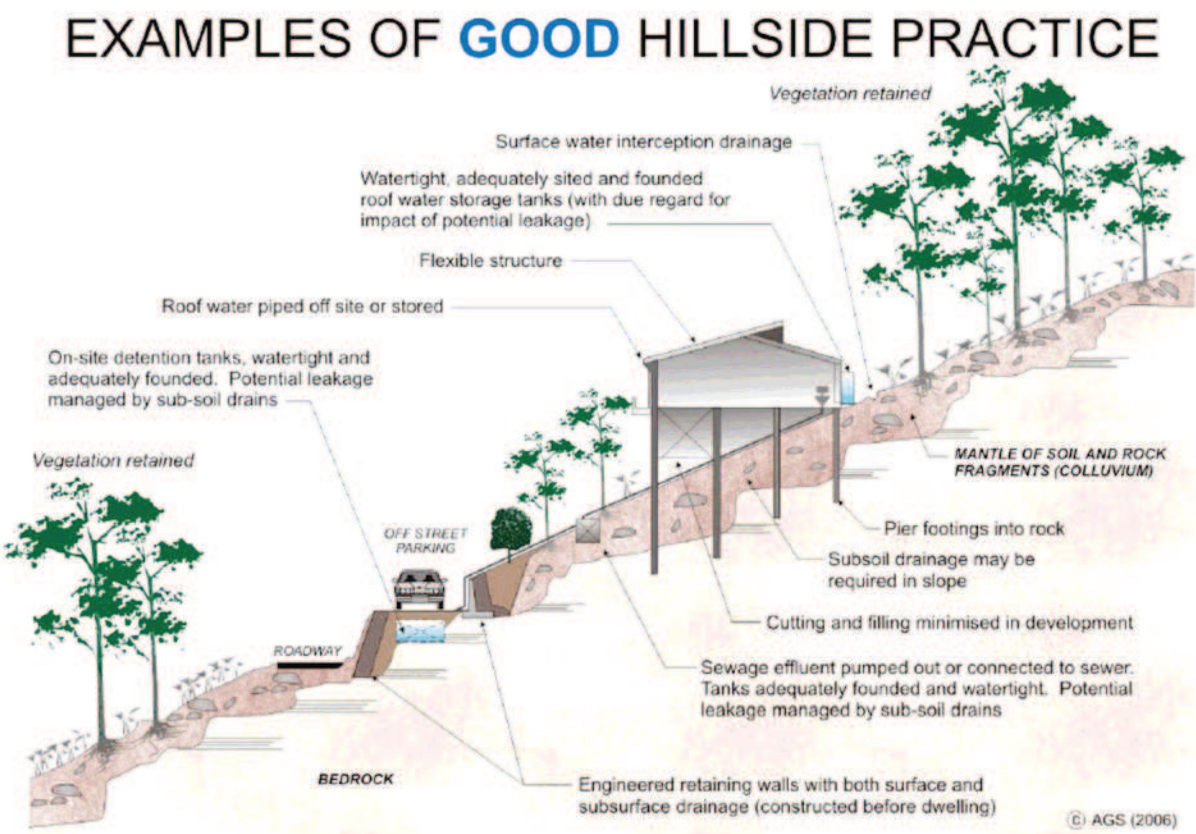
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

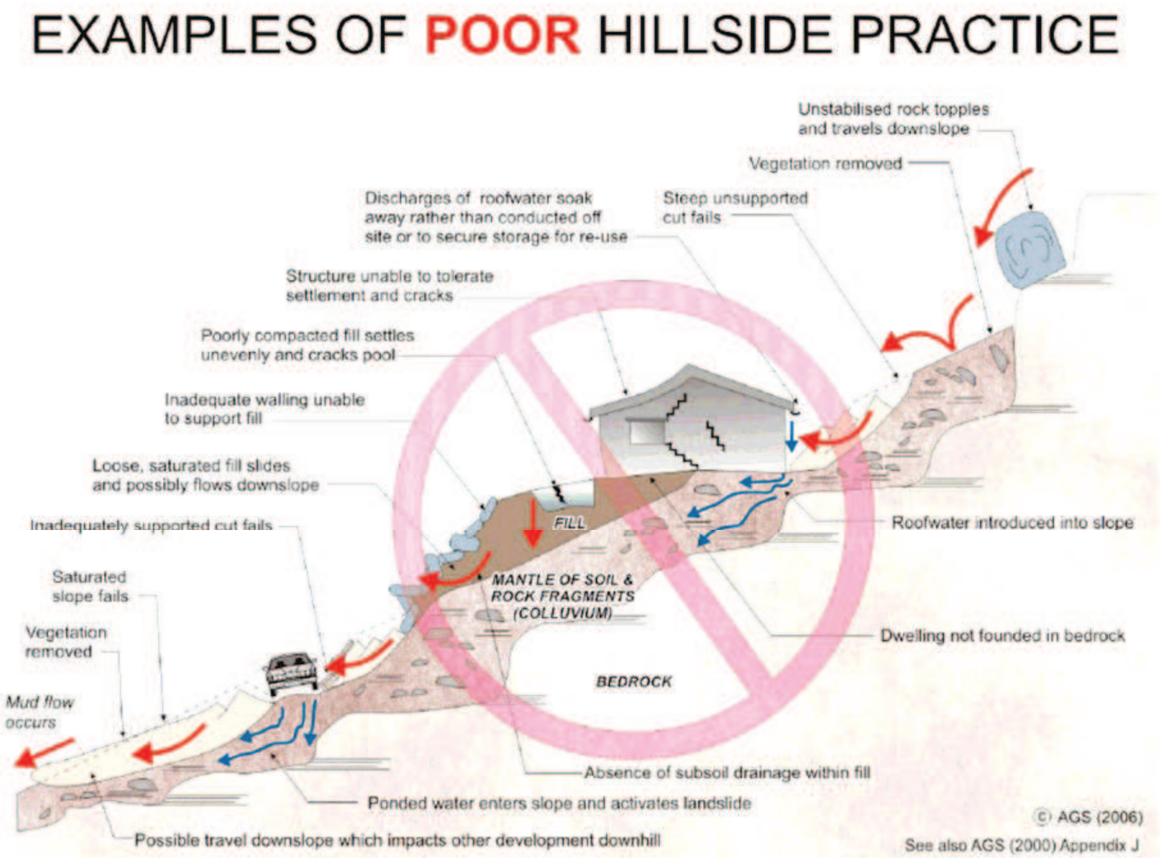
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

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)
© AGS (2006)


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