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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW DWELLINGS WITH CARPARK

at

52 LAUDERDALE AVENUE, FAIRLIGHT

Prepared For

David Allen and Jim Casey

Project: 2016-013.1

February, 2019

Document Revision Record

Issue No	Date	Details of Revisions
0	10th March, 2016	Original Issue
1	25th February 2019	Revised Issue as per latest Pre DA Plans

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Date: 25th February 2019

Project No.: 2016-013.1

GEOTECHNICAL REPORT FOR PROPOSED NEW DWELLINGS WITH CARPARK 52 LAUDERDALE AVENUE, FAIRLIGHT, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for the proposed two semi-detached dwellings with basement car park at 52 Lauderdale Avenue, Fairlight, NSW. The investigation was undertaken by Crozier Geotechnical consultants (CGC) at the request of Platform Architects on behalf of the client David Allen and Jim Casey.

The site is situated on the high, northern side of Lauderdale Avenue within steeply south sloping topography. It is currently occupied by a two storey brick clad dwelling located to the rear of the site. At the front of the house is a steep slope which extends down towards the road reserve where a 3 ï 4m high cliff exists.

The site is located within Area G2 of Manly Councils 2013 DCP ï Potential Geotechnical Landslip Hazard Map. Therefore as part of the Development Application, Manly Council requires a Site Stability Report.

A walkover inspection/assessment was carried out to prepare and provide this report for DA purposes. It also involved review of the latest site plans and documents along with a review of information from our database of investigations within nearby properties. The report includes a full risk assessment of the site for both property and life as per the AGS March 2007 publication. The report also includes a description of site and sub-surface conditions, a geotechnical assessment of the development and landslip risk assessment, site mapping/plan, geological section and provides recommendations for construction and stormwater disposal.

The investigation and reporting were undertaken as per the Tender P15-482, Dated: 10th November 2015.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer and Principal Engineering Geologist.
b) A photographic record of the site conditions.

The following plans and diagrams were supplied by the architect for the work;

- Architectural drawings (Pre DA Issue) by Platform Architects, Project: LFA, Drawing No.:
  - A0.00 and A0.02, Revision: Pre DA, Dated : 22nd January 2019;
  - A0.04, Revision: PR4, Dated: 15th January 2019;
  - A1.00, Revision: PR5, Dated: 22nd January 2019;
  - A1.01, A1.02, A1.04, A1.06 and A2.03, Revision: PR5, Dated: 4th February 2019;
  - A1.03, Revision: PR7, Dated: 15th February 2019;
  - A1.05, A1.06, A2.01 – A2.04, A3.01 and A5.01 – A5.03, Revision: PR7, Dated: 18th February 2019.
- Site survey plan by Geosurv, Plan Reference: 150834_A, Date of Survey: 18th August 2015.

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve demolition of existing site structures and construction of two, 5 storey high, semi-detached dwellings with a basement car park. The basement car park will be accessed from the roadway level via a driveway.

Based on the provided architectural drawings, the basement is designed with a finished floor level (FFL) to RL 33.35m. Therefore a bulk excavation will be required approximately to RL 33.10m to allow for construction of a basement concrete slab. As such the proposed construction will require a bulk excavation to approximately 12.00m depth which based on initial observations will extend entirely through sandstone bedrock. The proposed excavation will extend below the Council's footpath beyond the front boundary and will extend to both of the side boundaries. The lift overrun pits and service trenches may require deeper excavations locally.

3. SITE FEATURES:

3.1. Description:

The site is a skewed rectangular shaped block with angled front boundary located on the high north side of Lauderdale Avenue, within steeply south sloping topography. It has a front south boundary of 20.00m, rear north boundary of 15.24m, east side boundary of 41.44m and west side boundary of 28.50m as referenced from the provided survey plan.

The site is currently occupied by a two storey brick clad dwelling located within the rear portion of the property with dense vegetation and paved areas within the front portion of the property and at the rear of the existing house. A photograph of the site taken from Lauderdale Avenue is shown below:

Photograph 1: Front view of the site, looking north

3.2. Geology:

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

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defects orientations being south-east and north-east. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders.

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 3rd February 2016 by a Geotechnical Engineer and Principal Engineering Geologist which included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land. Due to the presence of boulders, outcrop, paved areas and dense vegetation throughout the site, it was not possible to successfully drill boreholes.

Explanatory notes are included in Appendix: 1. Mapping information is shown on Figure: 1 in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix 2.

4.2. Field Observations:

The project site is located on the high north side of Lauderdale Avenue within steeply south sloping topography. Along the road reserve, outside of the property boundaries, there is a sandstone rock cliff (3.00 ï 4.00m high, extending ~2.00m onto the concrete kerb) which extends beyond the width of the front boundary and appears to have been formed by excavation for construction of Lauderdale Avenue. The sandstone outcrop is of medium to high strength, highly weathered containing sub horizontal bedding defects with a dip/dip direction of 10°/260°. The bedrock within the immediate vicinity of the bedding defects is extremely weathered and of extremely low strength. Residual sandy/silty soil infill of up to 0.25m width was observed along some of the bedding defects. The intersecting sub horizontal bedding defects and sub vertical fracture and discontinuous joint (65°/190°) defects create blocks of rock measuring up to approximately 2.00m x 2.50m in size above the road reserve. It was observed that some of those blocks of rock appear to be detached from the main rock mass and are relatively unstable.

Above the cliff outcrop the road reserve contains a gently sloping (-10°) concrete path in good condition adjacent to a fenceless front boundary of the site. The front portion of the site consists of a concrete paved area with low lying rock walls extending across the site. The south east corner and a section of the site along the east side common boundary are covered with dense vegetation. A stairway located at the south west corner of the site leads up to a concrete path connected to stairs leading towards the house. The concrete path is in poor condition with multiple cracks. To the north of the path there is a cemented rock wall in good condition enclosing a steeply sloping (-20°) garden filled with overgrowth. The path leads to a set of concrete stairs which extends along the eastern boundary to the rear of the house and to a timber staircase that leads to a verandah at the front of the house.

The site is currently occupied by a two storey brick and clad dwelling at the rear of the property. The structure appears to be in good condition with no obvious cracks. The front wall of the dwelling is supported by a cemented rock wall (0.70m high), beneath the house floor slab, which is supported directly off moderately weathered, medium to high strength sandstone bedrock outcrop. The rock wall appears to be in good condition with some mortar loss, with the front of the dwelling showing no obvious signs of cracking or settlement. The bedrock extends below the majority of the front wall of the dwelling and extends to the west boundary, protruding 1.00 ï 1.20m from the building.

A steeply sloping (-25°) concrete path grants access to the rear of the house and site where there is highly weathered, medium to high strength sandstone bedrock outcrop as a low cliff adjacent to the rear of dwelling. The cut face is approximately 0.90m ï 1.20m high and gradually reduces to become level with the ground towards the eastern boundary. Near the western boundary, there is a tree growing through the defects within the cliff crest, with soil eroding along the defects. Above the cut face within a narrow gently sloping area, there is a metal shed and trees which extend to the rear boundary. The eastern boundary contains a large boulder or bedrock outcrop extending beneath the fence and onto the neighbouring property (No. 50).

The neighbouring property to the east (No. 50) consists of a two storey brick dwelling of > 25 years of age at the front of the property that steps down the slope and may have involved some excavation into the slope. Upon limited inspection, the structure appears to be in good condition with no obvious cracks or signs of settlement. Directly behind the dwelling, there is a concrete retaining wall approximately 5.00m high and in good condition. This retaining wall extends across the property in the east-west direction and joins with a concrete wall (approximately 1.5m high) extending in north-south direction along the common boundary. The rear portion of the property consists of a garden with sandstone outcrop that extends across the property (west to east). The property is at a similar ground level along the boundary with the building within 1.50m of the common boundary.

The neighbouring property to the west (No. 54) consists of a modern five storey brick rendered dwelling that covers the majority of the block with a basement level car park excavated in through the road reserve cliff line. Upon limited inspection, the structure appears to be in excellent condition with no obvious cracks or signs of settlement. A concrete block wall approximately 1.50 ï 2.00m high and stepping down along the slope is located at the common boundary with the site. The property is at a similar ground level along the boundary with the existing building within 1.00m of the common boundary where visible. The outline of the basement garage is unknown.

The neighbouring property to the rear north (No. 37 Upper Clifford Avenue) consists of a multi storey brick rendered dwelling and swimming pool with gently sloping lawn and garden at the rear. The property is at a higher ground level along the boundary with the pool located within 6.00m of the common boundary.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified outcropping sandstone bedrock across the site area, including outcrops at the surface at the front and rear of the existing house exposing medium to high strength, massive sandstone bedrock. A sandstone cliff extends across the site within the road reserve. This cliff appears previously excavated and contains several defects which could create unstable blocks of rock during excavation.

The slope extending from the front boundary of the site to the bedrock outcrop below the southern edge of the existing house, is covered with a paved area, paths and dense vegetation. This section of the site has a topsoil cover which is expected to be of limited thickness due to the extensive outcrop observed. Similarly a small area extending from the rear north boundary of the site to the top of the bedrock outcrop at the rear north side of the existing house is also covered with topsoil of limited thickness.

The sandstone bedrock exposed at the front and rear of the existing house and at the front boundary of the site is of medium to high strength and dominated by near horizontal bedding defects with few joint defects of a discontinuous nature. The project site contains no existing landslip hazards whilst there were no hazards identified within the neighbouring properties that could impact the site.

The proposed works involve construction of two, 5 storey high, semi-detached dwellings with a basement car park. The basement car park will be accessed from the roadway level via a driveway. The proposed construction will require a bulk excavation to approximately 12.00m depth that will extend below the Councilôs footpath beyond the front boundary and will extend to both of the side boundaries of the site.

Outside of minor topsoil and residual soil, the entire excavation will extend through sandstone bedrock of medium to high strength therefore rock excavation equipment is required. This reduces the probability of instability slightly however this has the potential to create significant ground vibrations which could damage neighbouring houses.

There is a potential for poorly oriented defects within the excavation to result in localized rock slide/topple failure with potential impact to the work site or neighbouring properties or structures. However through selection of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the of installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within óAcceptableô levels. Therefore the proposed development is considered suitable for the site.

The recommendations and conclusions in this report are based on an investigation utilizing only surface observations. This provides limited data across the entire site therefore some minor variation to the interpreted sub-surface conditions is possible, especially where the surface is covered with dense vegetation and pavements. However the bedrock underlying the site is encountered across the majority of the eastern half of Sydney and is well suited to the proposed works with numerous excavations of similar and larger size achieved safely.

Based on the known orientation of defects in the Hawkesbury Sandstone it is considered that core boreholes to below the excavation level will have limited use and that ongoing, regular inspection of the excavation by an experienced geotechnical engineer/engineering geologist along with timely installation of support measures as required will be better suited to ensure stability is maintained.

5.2. Site Specific Risk Assessment:

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

A. Landslip (rock slide/topple <5m^3) from proposed new excavation
B. Ground vibration damage to neigbhouring structures

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 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 vary from 1.67 x 10^{-5} to 2.08 x 10^{-11} to the persons working within the excavation and to the neighbouring houses, whilst the Risk to Property was considered to be óVery Low to Lowô. The hazard was therefore considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

5.3. Design & Construction Recommendations:

5.3.1. New Footings:

The results of the investigation identified the presence of shallow sandstone bedrock across the site. The new basement car park along with the building above will be founded at the base of an excavation into bedrock which is expected to expose medium to high strength sandstone. Therefore the structure may be founded on a slab or strip footings.

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 in-situ 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 slab or strip footings.

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

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

5.3.2. Excavation:

It is understood that the proposed works for the development include excavation up to 12.00m depth, which will extend to both the east and west side boundaries. The block wall located along the common boundary of the property No. 54 and the concrete wall located along the common boundary of the property No. 50 will be at the crest of the proposed excavation and will require assessment of their footings and underpinning to bedrock of at least medium strength if not already supported off. This should be confirmed prior to rock excavation. The upper north side boundary will be more than 5.00m away from the excavation limit therefore the probability of instability impacting this boundary is very low. The excavation will be up to 12.00m depth (RL 33.10m), for the basement car park and will reduce to 2.50 ï 4.50m depth (RL 32.50) for the access driveway below the Councilôs footpath along the front south boundary of the site.

Almost the entire excavation will extend through medium to high strength sandstone bedrock with the possibility of very 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. However if neighbouring structures are heritage listed, it is recommended that a vibration limit be 3mm/s be set.

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. These tests are conducted prior to rock excavation work being carried out to define the equipments characteristics, confirm appropriate buffer distances and site vibration characteristics.

Full time vibration monitoring is recommended, however the details of which will be determined by 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 the specified low level of vibration the below hammer weights and buffer distances are required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.0m
400kg	3.0m
600kg	6.0m
900kg	20.0m

It is expected that larger rock hammers will be preferred due to the depth and scale of the excavation, therefore ground vibration assessment and monitoring will be critical.

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 buildings, 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 15m 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.

5.3.3. Excavation Support:

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. Based on the proximity of the cuts to the western and eastern boundaries any excavation support (i.e. rock bolts) will need to extend across into the neighbouring properties.

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 across property boundaries. Therefore if required bolts should be based on temporary anchoring with full time support implemented as part of the development. Rock bolting across property boundaries will require approval from neighbouring property owners.

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 rock bolted.

Due to the expected presence of sub-vertical joints within the sandstone bedrock, toppling and wedge sliding failure during excavation may need to be addressed. 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 as per below mentioned sequence:

- Upon clearing of all soils from the bedrock surface,
- Following excavation to approximately 1.0m depth,
- Every 2.00m depth interval of the main excavation,
- At completion of the excavation.

5.3.4. Retaining Structures:

Whilst the medium strength bedrock will be self supporting new retaining walls may be required as part of the proposed development.

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.

Medium and better strength sandstone bedrock will be self supporting and will not require retention provided it is defect free and unaffected by the excavation works. Where defects are identified in the geotechnical inspections then rock support may be required. Backfilled retaining walls utilizing the coefficient for low strength sandstone may be implemented throughout as permanent support.

Temporary rock bolts may extend beyond the site boundaries and require written permission from property owners and be de-stressed at completion of the site works.

Table: 1 - Retaining Structures Design Parameters

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

* Ultimate design values   * ELS ï Extremely Low Strength   *LS ï Low Strength

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.

It is considered that the medium to high strength bedrock expected to be encountered within the proposed excavation is suitable for the use of galvanized steel rock bolts as a slope stabilising measure. The medium strength bedrock is suitable for a grout/rock adhesion of 600kPa (FoS å 2) for fully grouted steel rock bolts. It is recommended that Double Corrosion Protection bolts be used if geotechnical inspections determine the need for rock support. The location of individual rock bolts should be assessed by the geotechnical engineer. Rock bolts across property boundaries will require permission from neighbouring property owners, and should be based on temporary design with permanent support via the new development. The permanent retention design will need to be determined/assessed by the structural engineer.

5.3.5. Drainage and Hydrogeology:

The site is situated at mid-slope level within steeply south sloping topography which contains extensive sandstone bedrock outcrops. Minor groundwater seepage will be encountered on geological defects however based on the site location and topography 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 Councilôs stormwater system off site.

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspect the cleared bedrock surface and the proposed excavation equipment prior to its use,
3. Inspect excavation at 2.00m depth intervals
4. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
5. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilisation and drainage measures are in place.

5.5. Design Life of Structure:

We have interpreted the design life requirements to refer to structural elements designed to support the house etc, the adjacent slope, control storm-water and maintain the risk of instability within óAcceptableô limits. Specific structures and features that may affect the maintenance and stability of the site in relation to the proposed and existing development are considered to comprise:

- stormwater and subsoil drainage systems,
- retaining walls and soil slope erosion and instability,
- maintenance of trees/vegetation on this and adjacent properties,

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 ï 2011 (50 years)). It will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally the property owner should adopt and implement a maintenance and inspection program.

If this maintenance and inspection schedule are not maintained the design life of the property may not be attained. A recommended program is given in Table: C in Appendix: 3 and should also include the following guidelines.

- The conditions on the block donôt change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
    - a) CSIRO sheet BTF 18
    - b) Australian Geomechanics ñLandslide Risk Managementò Volume 42, March 2007.
    - c) AS 2870 ï 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owners have any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component.

It is assumed that Manly Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, adjacent Public Reserve, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they donôt leak and increase either the local groundwater level or landslide potential.

Recommendations for construction within hill slopes are also provided in Appendix: 5.

6. CONCLUSION:

The site investigation identified shallow sandstone bedrock across the site area, including outcrops at the surface at the front and rear of the existing house and at the front boundary of the site. These outcrops exposed medium to high strength, massive sandstone bedrock with few sub-vertical defects. There were no obvious landslip hazards identified across the site. However the proposed development has the potential to expose new hazards such as rock slide/topple during excavation whilst the use of unsuitable excavation equipment also has the potential to create damage to neighbouring house structures through excessive ground vibrations.

The proposed works involve construction of two, 5 storey high, semi-detached dwellings with a basement car park. The proposed excavation will be up to 12.00m depth (to RL 33.10m), for the basement car park and will reduce to 2.50 ï 4.50m depth (to RL 32.50) for the access driveway below the Councilôs footpath along the front south boundary of the site. The excavation will extend to the east and the west boundaries of the site and the concrete and block walls at the common boundaries. The excavation will extend almost entirely through sandstone bedrock of medium to high strength therefore the risk of instability is significantly reduced. However ongoing mapping by a geotechnical engineer is necessary to ensure the stability of the rock excavation.

It is 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 to maintain comfort levels and provide a very low probability of structure damage.

It is considered that the site and proposed works can maintain an óAcceptableô risk criterion for the design life of the development, taken as 50 years, provided proper engineering design and construction methods are implemented, including but not limited to the recommendations of this report.

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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: -

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

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

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

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

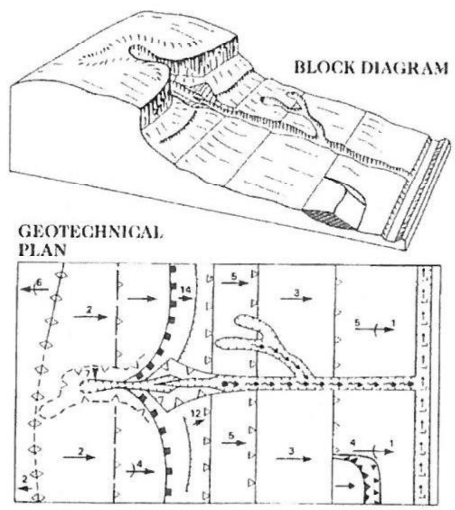
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

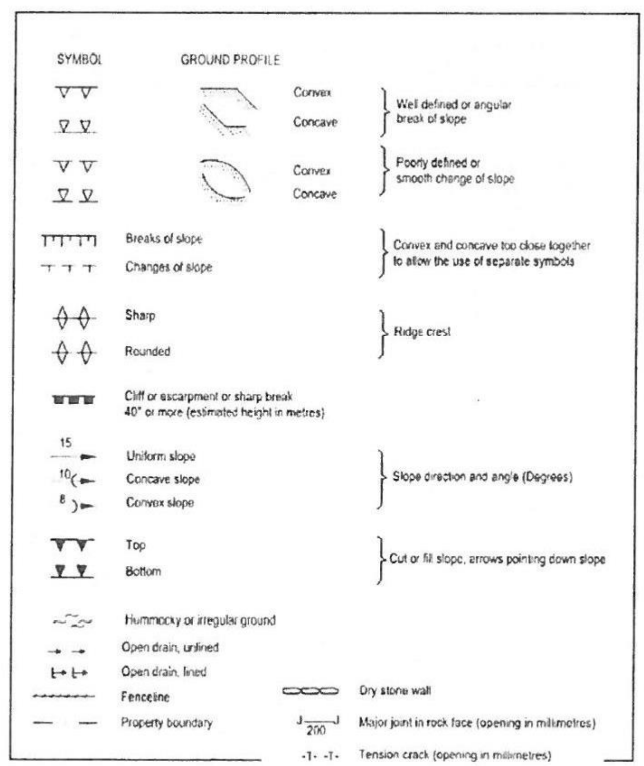
Reproduction of Information for Contractual Purposes

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

Site Inspection

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

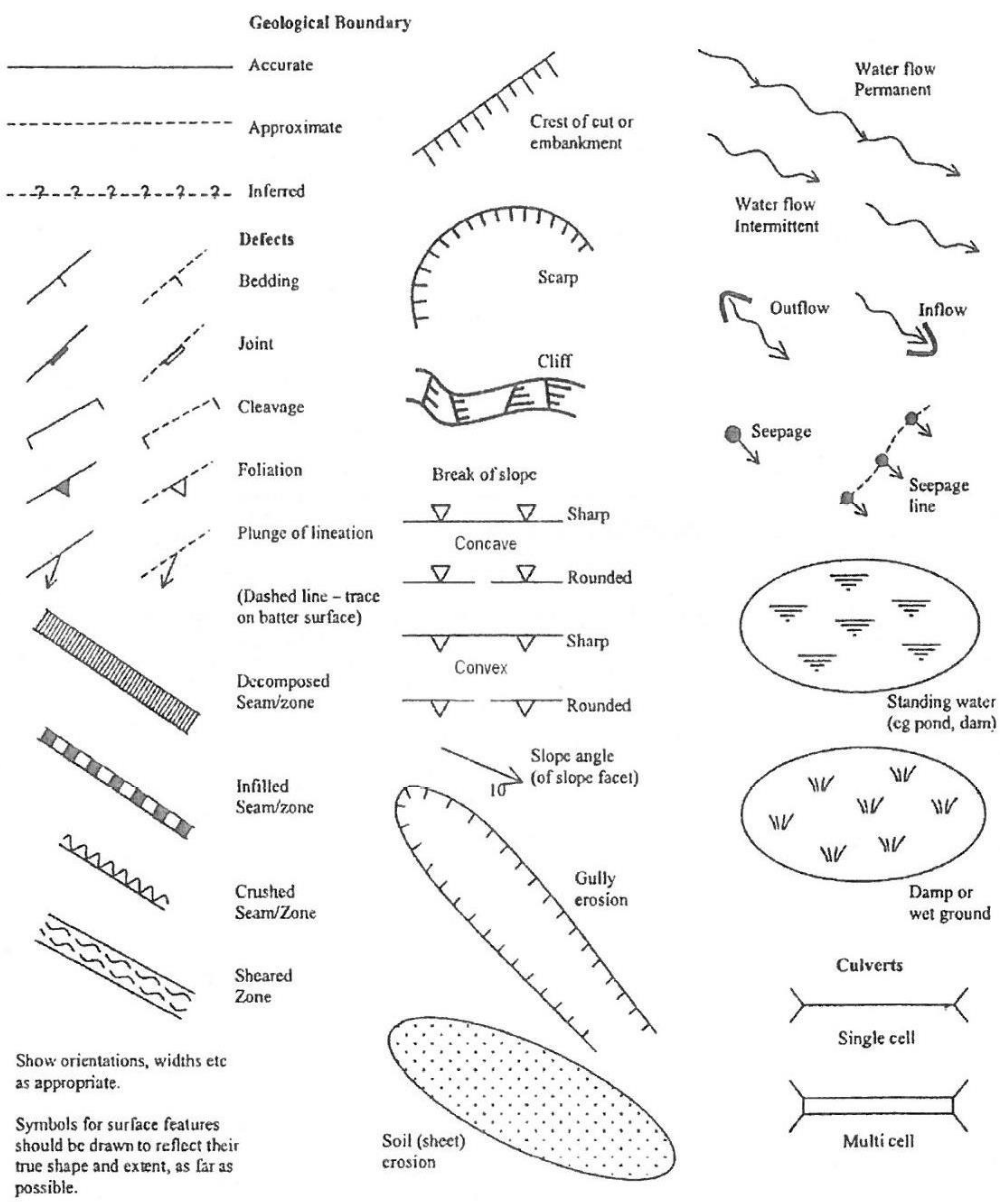




Example of Mapping Symbols

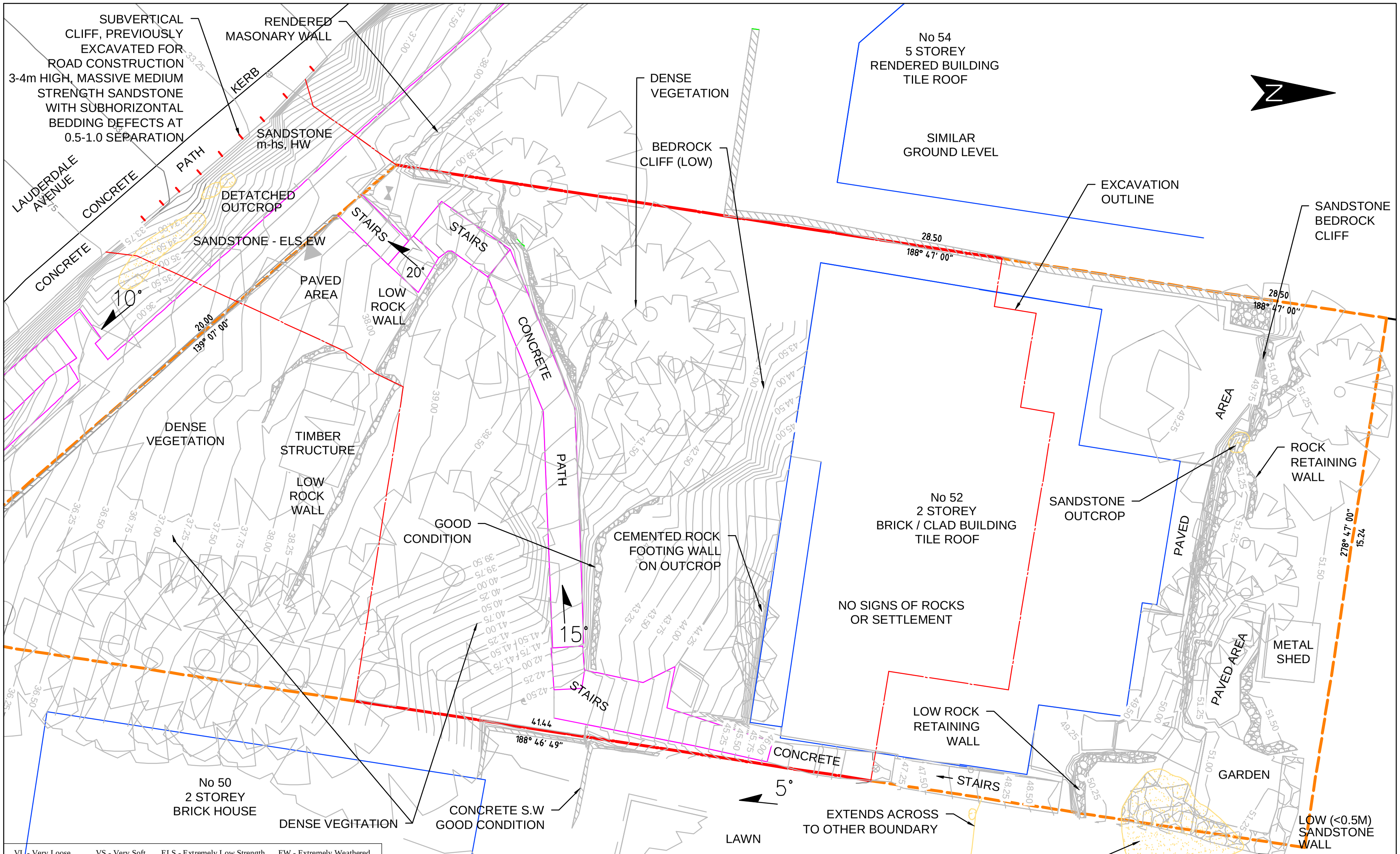
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered
SS - Sandstone	H - Hard	VHS - Very High Strength	FR - Fresh

LEGEND

TOP OF SLOPE
BASE OF SLOPE
SURFACE WATER FLOW
AUGER LOCATIONS
DYNAMIC PENETROMETER TEST
SANDSTONE BEDROCK
SLOPE ANGLE
SANDSTONE RETAINING WALL
BOULDER
CLIFF LINE

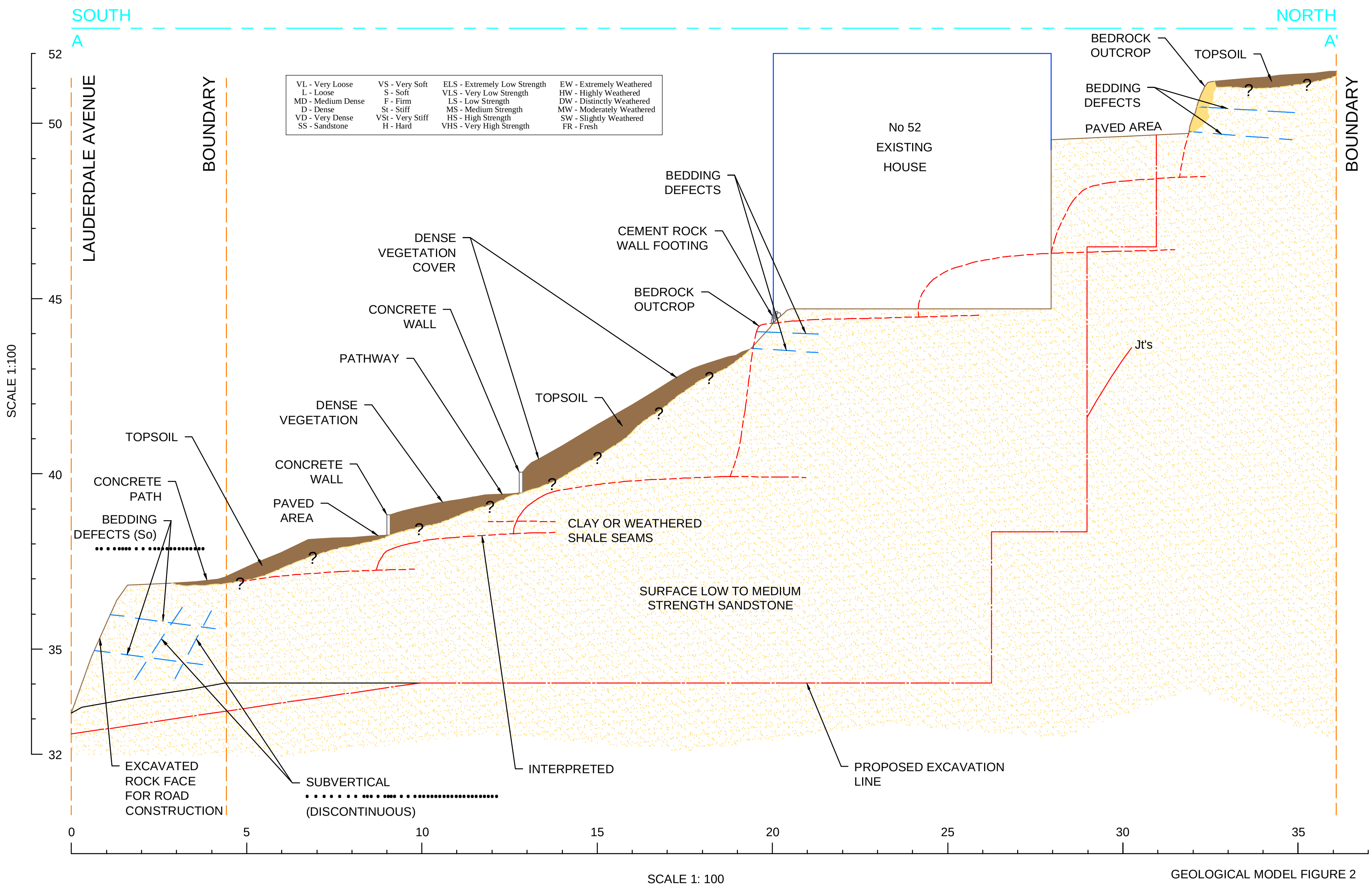
CROZIER GEOTECHNICAL CONSULTANTS

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

SCALE:	1:100
DRAWING:	FIGURE 1
DRAWN:	SC
DATE:	22-02-2019
APPROVED BY:	
PROJECT:	2016-013.1

PREPARED FOR:

ADDRESS:
52 LAUDERDALE AVENUE
FAIRLIGHT



GEOLOGICAL MODEL FIGURE 2

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
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LEGEND

AUGER LOCATIONS
DYNAMIC PENETROMETER TEST
TOP SOIL
SANDSTONE BEDROCK

SCALE:	1:100
DRAWING:	FIGURE 2
DRAWN:	SC
DATE:	22-02-2019
APPROVED BY:	
PROJECT:	2016-013.1

PREPARED FOR:

ADDRESS:
52 LAUDERDALE AVENUE
FAIRLIGHT

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 slide/topple <5m³) due to proposed excavation		Minimal soil cover over massive bedrock, minimal defects observed in outcrop, deep excavation likley to intersect some unfavourable jointing	a) work within excavation, may impact up to 10% of work area b) excavation 2.5m from edge of house and 1m from the concrete wall at common boundary, only impact small portion of structure c) excavation within 2m of house and 1m from the brick wall at common boundary, only impact small portion, d) excavation more than 10m away from pool, no impact		a) Person on site 8 hrs/day, b)Persons house 20 hrs/day, c) Persons house 20 hrs/day, d) Persons use pool 2 hrs/day	a) Possible to not evacuate b)Possible to not evacuate, c) Possible to not evacuate, d) Unlikely to not evacuate	a) Person buried b)Person in building minimal impact, c) Person in building minimal impact d)Person using pool, no impact	
			Possible	**Prob. of Impact**	**Impacted**				
		a) site works	0.001	1.00	0.10	0.3333	0.50	1.00	**1.67E-05**
		b) neighbouring house (No. 50)	0.001	0.20	0.05	0.833	0.50	0.05	**2.08E-07**
		c) neighbouring house (No. 54)	0.001	0.20	0.05	0.833	0.50	0.05	**2.08E-07**
		d) neighbouring house with pool (Rear North)	0.001	0.01	0.01	0.083	0.25	0.01	**2.08E-11**

* hazards considered in current condition and/or without remedial/stabilisation measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing 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 slide/topple <5m³) due to proposed excavation	a) site works	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) neighbouring house (No. 50)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		c) neighbouring house (No. 54)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		d) neighbouring house with pool (Rear North)	Rare	The event is conceivable but only under exceptional 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: C

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event.
Retaining Walls. or remedial measures	Owner to inspect walls for deviation from as constructed condition.	Every two years or following major rainfall event.
Large Trees on or adjacent to site	Arbourist to check condition of trees and remove branches as required.	Every five years
Slope Stability	Hydraulics (stormwater) & Geotechnical Consultants to check on site stability at same time and provide report.	One year after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with AS2870.

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

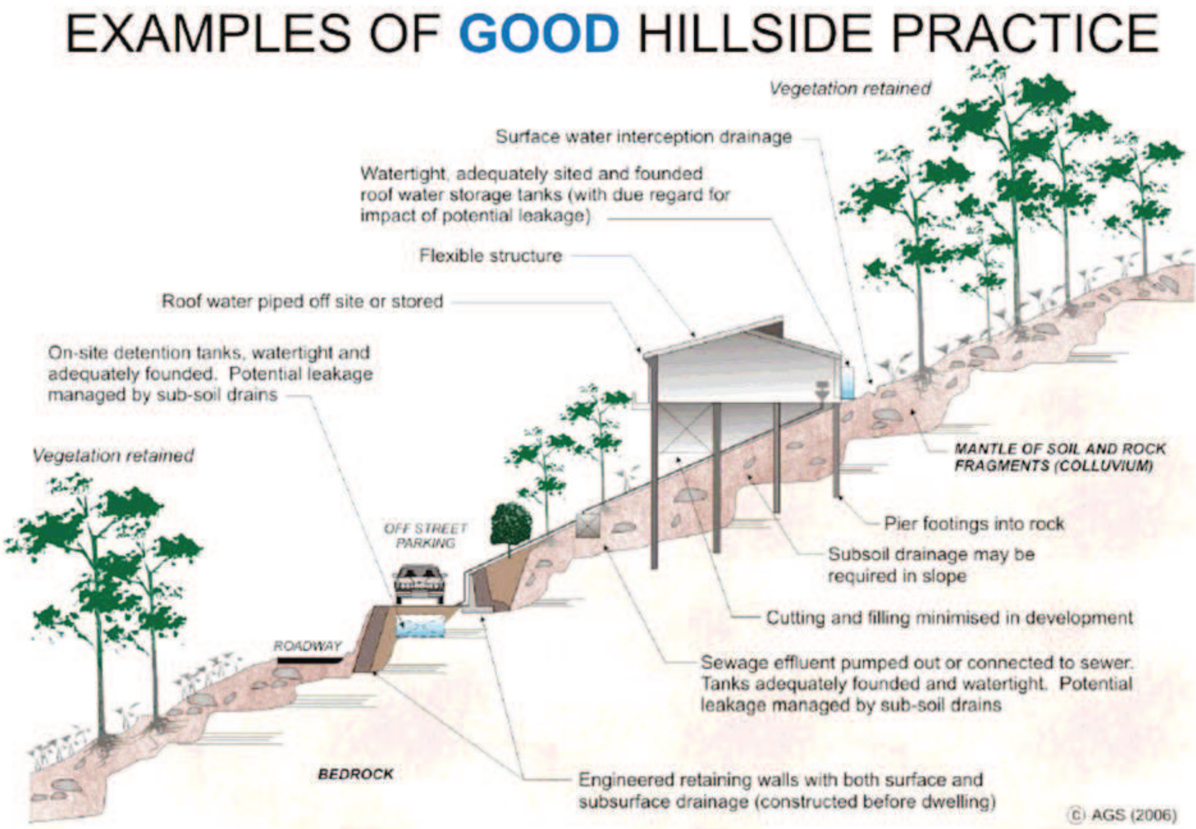
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
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.

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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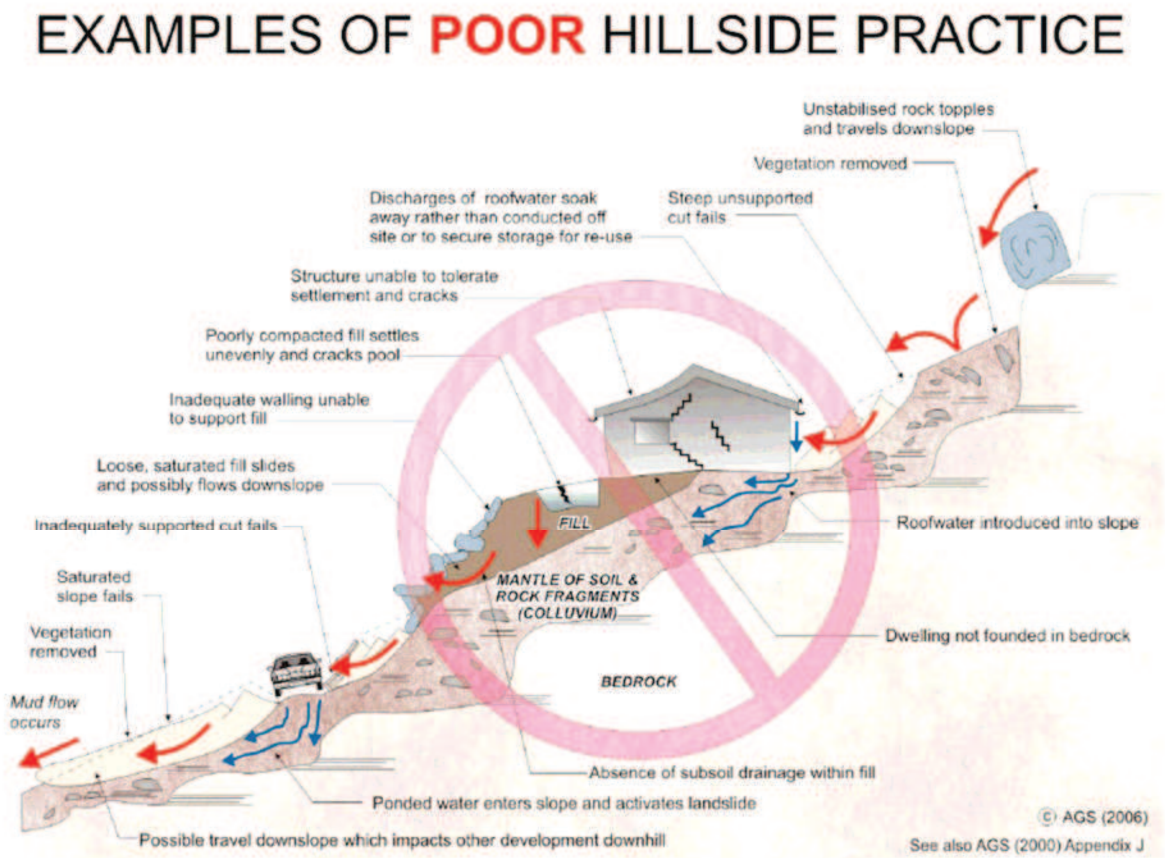
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles 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