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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW THREE STOREY HOUSE WITH BASEMENT

at

25 LOCH STREET, FRESHWATER

Prepared For

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GEOTECHNICAL REPORT FOR PROPOSED NEW THREE STOREY HOUSE WITH BASEMENT
25 LOCH STREET, FRESHWATER, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a new three storey house with basement at 25 Loch Street, Freshwater, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Rolf Ockert Architect on behalf of Andrew and Johanna Smith.

The proposed works involve demolition of existing site structures and construction of a new residential dwelling with a basement level. The proposed development will require excavation of up to 4.0m depth below the existing ground level to enable the construction of the garden level at FFL44.40m. Required excavation will be the greatest at the front of the property, decreasing towards the rear of the proposed structure to approximately 1.0m. The excavation will be approximately 1.0m the north and south boundaries and will be approximately 12.50m and 19.0m from the east and west boundaries respectively.

Northern Beaches (Warringah) Councils 2011 Land Environment Plan (LEP) and Development Control Plan (DCP) states that all building development applications must be accompanied by a geotechnical landslip assessment. Developments within Class óAô, óBô and óDô landslip risk zone may require a preliminary assessment only where excavation/fill is <2.0m depth, however Class óCô and óEô will require a geotechnical report, see details in Section E10 Landslip Risk.

This site is located within landslip risk Class óBô within the Geotechnical Risk Management Map. 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 geotechnical investigation is required.

To fulfil this requirement, an investigation into ground conditions is required along with detailed mapping and a landslide risk assessment of the site for both property and life as per the Council requirements and the Australian Geomechanics Society Guidelines (2007). This report therefore includes a description of the field work, plans showing test locations, a geological section, and site specific risk assessment and provides recommendations for construction and preliminary design to ensure geotechnical stability can be maintained.

The investigation and reporting were undertaken as per the Tender P18-253, Dated: 13th August 2018.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Senior Engineering Geologist.
b) Drilling of two boreholes using hand tools along with six Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface conditions.

The following plans and drawings were supplied for the work:

- Site survey plan by Stutchbury Jaques Pty Ltd, Plan Reference: 8369/13, Dated: 17/04/2018.
- Architectural drawings by Rolf Ockert Architect, Drawing Nos.: 25L 1.00 to 1.04, Rev. p6 and 25L 2.01 to 2.05 Rev. p6 Dated: October 2018.

2. SITE FEATURES:

2.1. Description:

The site is a broadly rectangular shaped block located on the low east side of Loch Street. It has front west and rear east boundaries of approximately 12.5m and side north and south boundaries of approximately 50m and covers an area of approximately 630m^2 in plan as determined from provided survey plan.

Ground surface levels within the site vary from a high of approximately RL48.0m within the west of the site to a low of approximately RL38.0m near the eastern site boundary at the base of a steeply east dipping rear garden.

The site is situated within gently then steeply east dipping topography and contains exposures of bedrock within the rear garden. The site is currently occupied by a one and two storey residential house with a partial basement garage. The front of the site contains a near level lawn and driveway ramp which provides access to the garage underneath the dwelling.

A view of the site is provided in Photograph 1.

Photograph 1: View of the front of the site and immediate surrounds looking broadly east

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. Units of this rock were identified within the local area of the site.

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 20th September 2018 by a Senior Engineering Geologist. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of outcrops, slopes and existing structures. It also included the drilling of two auger boreholes using a hand auger (BH1 and BH2) to determine sub-surface geology.

Dynamic Cone Penetrometer (DCP) testing was carried out from ground surface adjacent to the boreholes and at five other select locations in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò to estimate near surface soil conditions and assess depths to bedrock.

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

3.2. Field Observations:

The site contains a one and two storey brick house within gently then steeply east dipping topography. Retaining walls up to approximately 2.50m in height are present within the rear (east) of the property and displayed some signs of deterioration however it is considered likely a result of lack of maintenance rather than a deep seated stability issue. Bedrock outcrops were observed within the front and rear of the property and comprised medium to high strength sandstone. Within the rear of the property, an exposure of sandstone was observed which forms part of a larger, north-south trending cliff feature. Signs of potential instability where not evident within the rock mass and loosened, detached boulders were not observed within or adjacent to the site.

Near the base of the access ramp within the property, bedrock was also observed underlying an exterior wall as shown in Photograph 4. It is considered likely that the remainder of the dwelling is founded on bedrock given the absence of any cracking observed in the exterior brickwork.

Photograph 4: Existing house founded on bedrock

To the north of the site the access driveway for No.2 Wyadra Avenue is located which is a recent subdivision and the main part of the lot lies directly to the east of the site. The subdivision is currently vacant is but it is understood a residential development is proposed however details are not known. No.23 Loch Street lies to the south of the site and contains a two storey residential dwelling.

Ground levels within the surrounding properties are similar to site immediately adjacent to all shared boundaries.

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

3.3. Ground Conditions:

For a description of the subsurface conditions encountered at the borehole locations, the Test Bore Report sheets and Dynamic Penetrometer Test Sheet should be consulted. However, a broad summary of the subsurface conditions encountered at the test locations is given below.

- **FILL** ï this layer was encountered at both borehole locations and comprised sandy gravel which was proven to depths of 0.30m (BH1) and 0.40m (BH2). The boreholes refused within the fill in BH2 and base the base of the fill was not determined. Fill appeared to extend to a maximum depth of 2.25m (DCP5) within the rear of the site. Elsewhere within the rear of the site (e.g. DCP4 and DCP6) the fill extended to depths of less than 1.0m and was underlain by bedrock.
- **SANDSTONE BEDROCK** ï Based on the results of DCP testing, the depth to the sandstone bedrock of a minimum low strength was interpreted to vary from 0.30m (DCP1 and DCP2) at the front of the site to 2.25m (DCP5) at the rear of the site adjacent to the top of a retaining wall.

A free standing ground water table or significant water seepage were not identified within any of the boreholes. No signs of ground water were observed after the retrieval of the DCP rods.

4. COMMENTS:

4.1. Geotechnical Assessment

The site investigation identified the presence of sandy gravel fill of generally shallow (<1.0m) thickness over the majority of the site underlain by sandstone bedrock. Based on the results of the DCP testing, the depth to sandstone bedrock of a minimum low strength is inferred to be varying from 0.30m (DCP1) within the front of the site, to 2.25m (DCP5) at the rear of the site. The depth to bedrock is variable depending on location with respect to existing retaining walls. This low strength bedrock is expected to grade very quickly to medium to high strength based on previous local experience and assessment of bedrock exposures within and surrounding the site.

The proposed works involve demolition of existing site structures and construction of a new residential dwelling with a basement level. The proposed development will require excavation of up to 4.0m depth below the existing ground level to enable the construction of the garden level at FFL44.40m. Required excavation will be the greatest within the front of the property, decreasing to approximately 1.0m towards the rear of the proposed structure. The excavation will be approximately 1.0m from the north and south boundaries and will be approximately 12.50m and 7.0m from the east and west boundaries respectively.

The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser or higher strength than interpreted. For confirmation of bedrock strength to below proposed excavation/footing level an investigation utilizing cored boreholes in the actual excavation/footing location would be required. However access for such equipment is very limited by existing site conditions. It is anticipated that assessment of rock strength can be undertaken during construction to allow confirmation of preliminary recommendations provided in this report.

The excavation of medium to high strength bedrock will require the use of rock excavation equipment, which can produce damaging ground vibrations. Since the excavation work is to be carried out adjacent to neighbouring residential buildings within adjacent properties, it is recommended that the geotechnical engineer or engineering geologist assess the excavation methodology and equipment prior to commencement to determine the need for full time vibration monitoring of neighbouring structures (principally No.23).

Whilst there were no stability hazards identified in the investigation, there is a potential for poorly oriented defects within the excavated bedrock to result in localized rock slide/topple failure with potential impact to the work site or the adjacent properties. Based on the separation distances, any potential instability has a low probability of impacting neighbouring building structures. Through selection of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within ôAcceptableö levels for all situations.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand drilling tools due to access limitations. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into bedrock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. The results of the investigation provide a reasonable basis for the DA analysis and preliminary design.

4.2. Site Specific Risk Assessment:

Based on our site investigation we have identified the following geological/geotechnical landslip hazard which needs to be considered in relation to the existing site and the proposed works. The hazard is:

A. Rockslide/topple failure in bedrock due to defects

A qualitative assessment of risk to life and property related to these hazards 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 3.13×10^{-6} for occupants of the basement excavation and varies from 1.04×10^{-7} to 7.50×10^{-8} for the neighbouring properties, whilst the Risk to Property was considered to be 'Very Low'** in all situations. The works are considered to have an ôAcceptableö risk level when assessed against the criteria of the AGS 2007.

4.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class ôAö for footings on bedrock at base of excavation
Type of Footing	Strip/Pad or Slab at base of excavation
Sub-grade material and Maximum Allowable Bearing	- Low Strength Sandstone: 1,000kPa

Capacity	- Medium Strength Sandstone: 2,000kPa (unconfirmed) - High Strength Sandstone: 3,000kPa (unconfirmed)
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: Higher pressures based on the Pells et al (1978) system are possible however would require further assessment. All footings should be founded off bedrock of similar strength to reduce the potential for differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.	

4.3.2. Excavation:		
Depth of Excavation	Basementï up to 4.00m	
Distance to Neighbouring Properties/Structures	No.2 Wyadra Avenue (driveway) ï 1.0m to the common boundary, No.2 Wyadra Avenue (main property lot) ï 12.50m to the common boundary, lot currently vacant, No.23 Loch Street ï 1.0m to the common boundary, site building a further 2.00m away Road Reserve ï 7.0m to Loch Street road reserve.	
Type of Material to be Excavated	0.00 up to 2.25m (Localised only)	Sandy gravel fill
	Below 2.25m (although typically below 1.0m)	LS and likely MS/HS sandstone bedrock
Guidelines for batter slopes for this site are tabulated below:		

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill soils	1:1.5	2:1
Low strength bedrock	1:1	1.25:1
Medium strength (MS), defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by engineering geologist.

Remarks:
Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Topsoil/Fill	Excavator with bucket
	LS ï MS/HS bedrock	Rock hammer and saw

LS ï low strength, MS ï medium strength, HS ï High Strength

Remarks:
Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to achieve a low level of vibration as recommended by the Australian Standards then the below hammer weights to approximate buffer distances are required:

Buffer Distance from Structure	Maximum Hammer Weight
2.0m	200kg
4.0m	500kg
5.0m	800kg
8.0m	1000kg

Onsite calibration will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Calibration of rock excavation machinery will need to be carried out prior to commencement of rock excavation works and will determine the need for full time monitoring.

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. In addition, intact detached sections of rock near

the surface can be split where they extend across neighbouring and below structures.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Buildings and services in surrounding properties = 5mm/s
Vibration Calibration Tests Required	Yes
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: • Following cleaning of soils from bedrock surface • At 1.50m depth interval of excavation where unsupported • At completion of the excavation
Dilapidation Surveys Requirement	On neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

4.3.3. Retaining Structures:	
Required	MS-HS sandstone bedrock is self supporting and does not require retaining walls, provided it is un-fractured by excavation works and contains no poorly orientated defects. Where identified by geotechnical inspection, the detached/potentially unstable sections can be supported by either bolt/shotcrete on either temporary or permanent basis with backfilled retaining walls near surface in areas of poor quality bedrock. Where support is required, with bolts extending across the boundary then the bolts should be on temporary basis with lateral support implemented by the competent building development/structure.
Types	Steel reinforced concrete/concrete block wall designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures..
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill and Natural soils	20	ϕ' = 32°	0.30	0.50	N/A
LS or fractured bedrock	23	ϕ' = 40°	0.10	0.15	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 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).

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (Ò0.50 L/min), on defects and at soil/rock interface
Site Location and Topography		Low east side of the road within moderate to steeply east sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Disposal via absorption methods is not feasible, however dispersion may be suitable.

Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Councilÿs stormwater system off site.

It is understood on site detention is proposed to manage water from the rear section of the new structure. Adequate thicknesses of soil potentially suitable for a detention system were only encountered in DCP5 (2.25m) at the rear of a poor quality rock retaining wall. Care will have to be taken to ensure that if a

detention system is proposed in this area is does not result in increased water pressure behind the existing retaining wall or increased detachment of this structure. If this is proposed then a support system or new retaining wall is recommended.
It should be noted that the depth of soil at all locations accept DCP5 was less than 1.0m which will need to be considered in hydraulic design.

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural drawings and stormwater system plans for compliance with the recommendations of this report,
2. Review excavation Methodology and Equipment prior to hard rock excavation,
3. Inspect excavation at 1.50m depth intervals where unsupported,
4. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect the completed development to ensure all 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 Councils Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot complete the certification if it has not been called to site to undertake the required inspections.

5. CONCLUSION:

The site investigation identified the presence of gravelly fill of shallow thickness (generally <1.0m increasing behind existing retaining walls to 2.25m) over the project site overlying low strength sandstone bedrock. This low strength bedrock is expected to grade very quickly to medium strength and likely stronger.

The proposed works involve demolition of existing site structures and construction of a new dwelling with basement. The proposed development will require an excavation of up to 4.00m depth below the existing ground level within the front of the site. It is expected that most of the excavation will extend through sandstone bedrock of low to medium and potentially high strength.

The excavation of medium to high strength bedrock will require the use of rock excavation equipment, which can produce damaging ground vibrations. Since the excavation work is to be carried out within 5.00m of surrounding structures, it is recommended that the geotechnical engineer or engineering geologist assess the excavation methodology and equipment prior to commencement to determine the need for vibration monitoring of neighbouring structures.

The risks associated with the proposed development were determined as being and can be maintained within óAcceptableô levels with negligible impact to neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared by:	Reviewed by:
Kieron Nicholson	Troy Crozier
Senior Engineering Geologist	Principal Engineering Geologist
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6. REFERENCES:

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2. Geological Society Engineering Group Working Party 1972, ñThe preparation of maps and plans in terms of engineering geologyò Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
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4. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
5. Pells et. al. Design loadings for foundations on shale and sandstone in the Sydney region. Australian Geomechanics Society Journal, 1978.
6. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings.

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Appendix 1

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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 ï 50
Stiff	50 ï 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 ï 10	2 ï 5
Medium dense	10 ï 30	5 -15
Dense	30 ï 50	15 ï 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 ï 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) ï 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 ï 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 ï 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 SPTõs 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 ôfeelö and rate of penetration.

Rotary Mud Drilling ï 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 ï 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, ñMethods of Testing Soils for Engineering Purposesò ï 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 ôNö 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.

ð 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

ð 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 ï 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: -

ð Cone resistance ï the actual end bearing force divided by the cross-sectional area of the cone ï expressed in MPa.

ð Sleeve friction ï the frictional force on the sleeve divided by the surface area ï expressed in kPa.

ð 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 ï 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 ï 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: -

Qc (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: -

Qc = (12 to 18) Cu

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.

- ŏ Perth sand penetrometer ï a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.2). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- ŏ Cone penetrometer (sometimes known as Scala Penetrometer) ï 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 ñMethods of Testing Soil for Engineering Purposesò. 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 óstraight lineô 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
B	Bulk Sample	PP	Pocket Penetrometer Test
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test
U63	63mm ñ ñ ñ ñ ř		

Ground Water

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

- ŏ 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.
- ŏ A localised perched water table may lead to an erroneous indication of the true water table.
- ŏ 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.
- ŏ 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:

ŏ unexpected variations in ground conditions ï the potential for this will depend partly on bore spacing and sampling frequency,

ŏ changes in policy or interpretation of policy by statutory authorities,

ŏ 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.

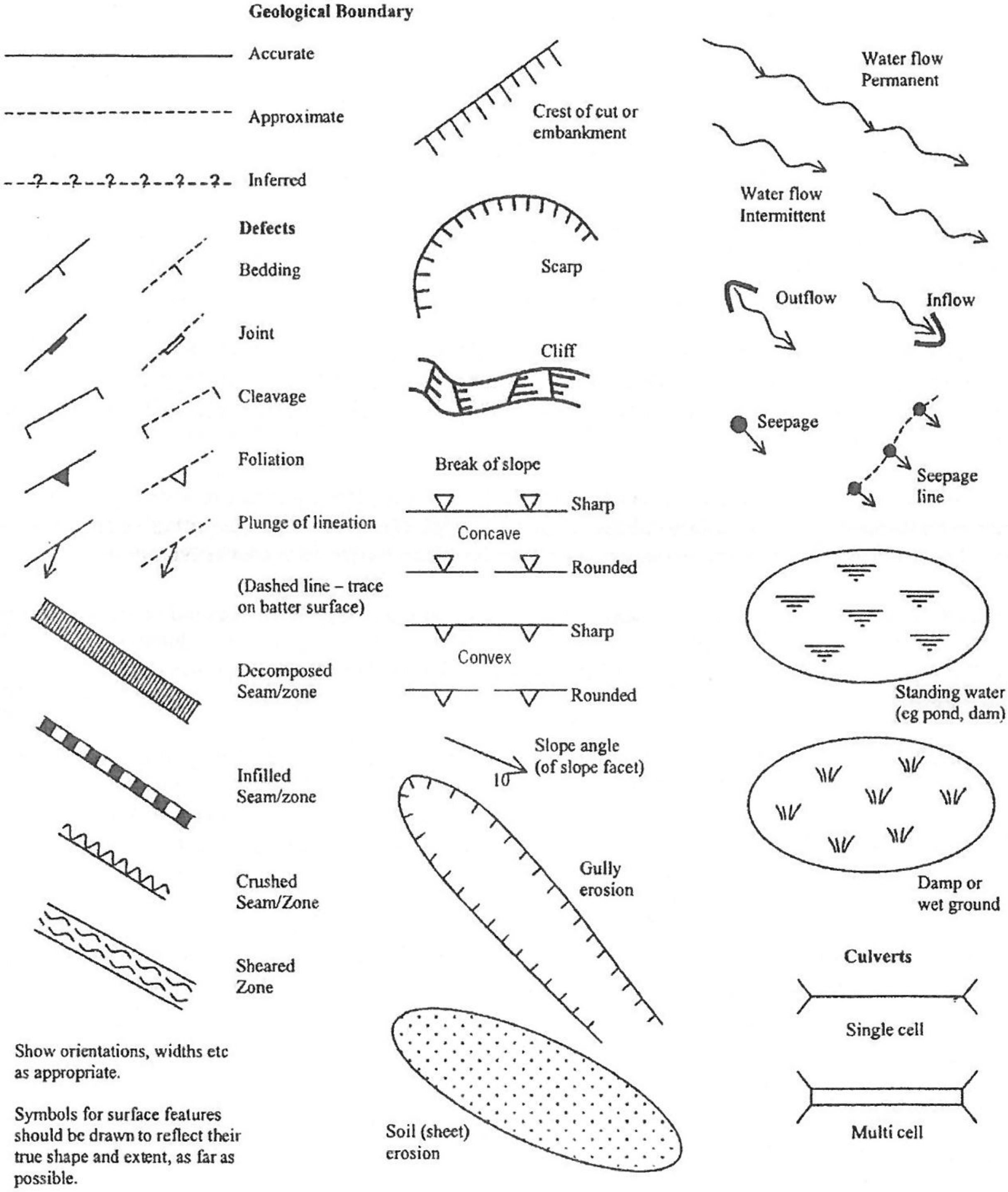
Reproduction of Information for Contractual Purposes

Attention is drawn to the document ñGuidelines for the Provision of Geotechnical Information in Tender Documentsò, 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.

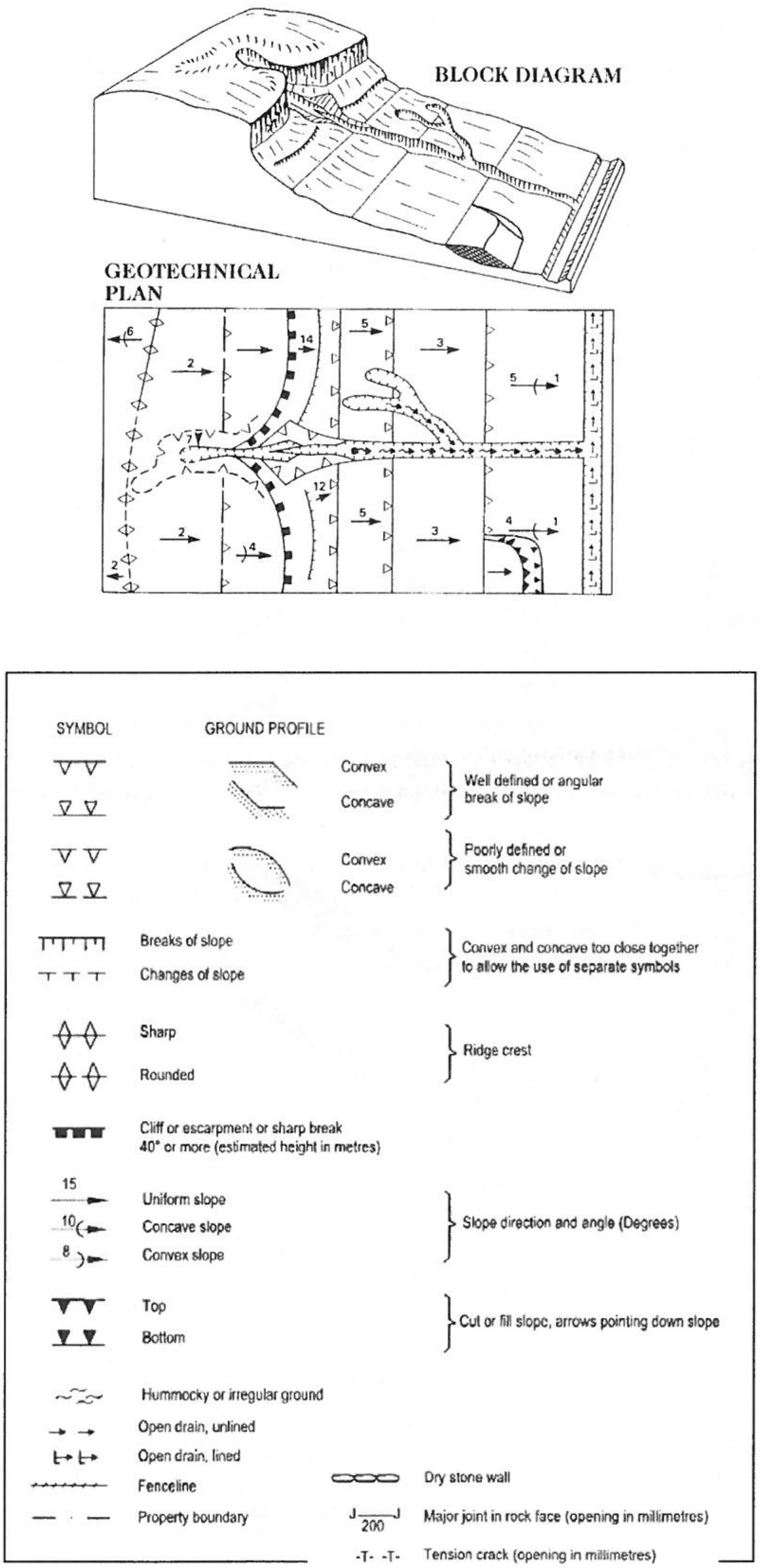
Site Inspection

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.

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).



Example of Mapping Symbols

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

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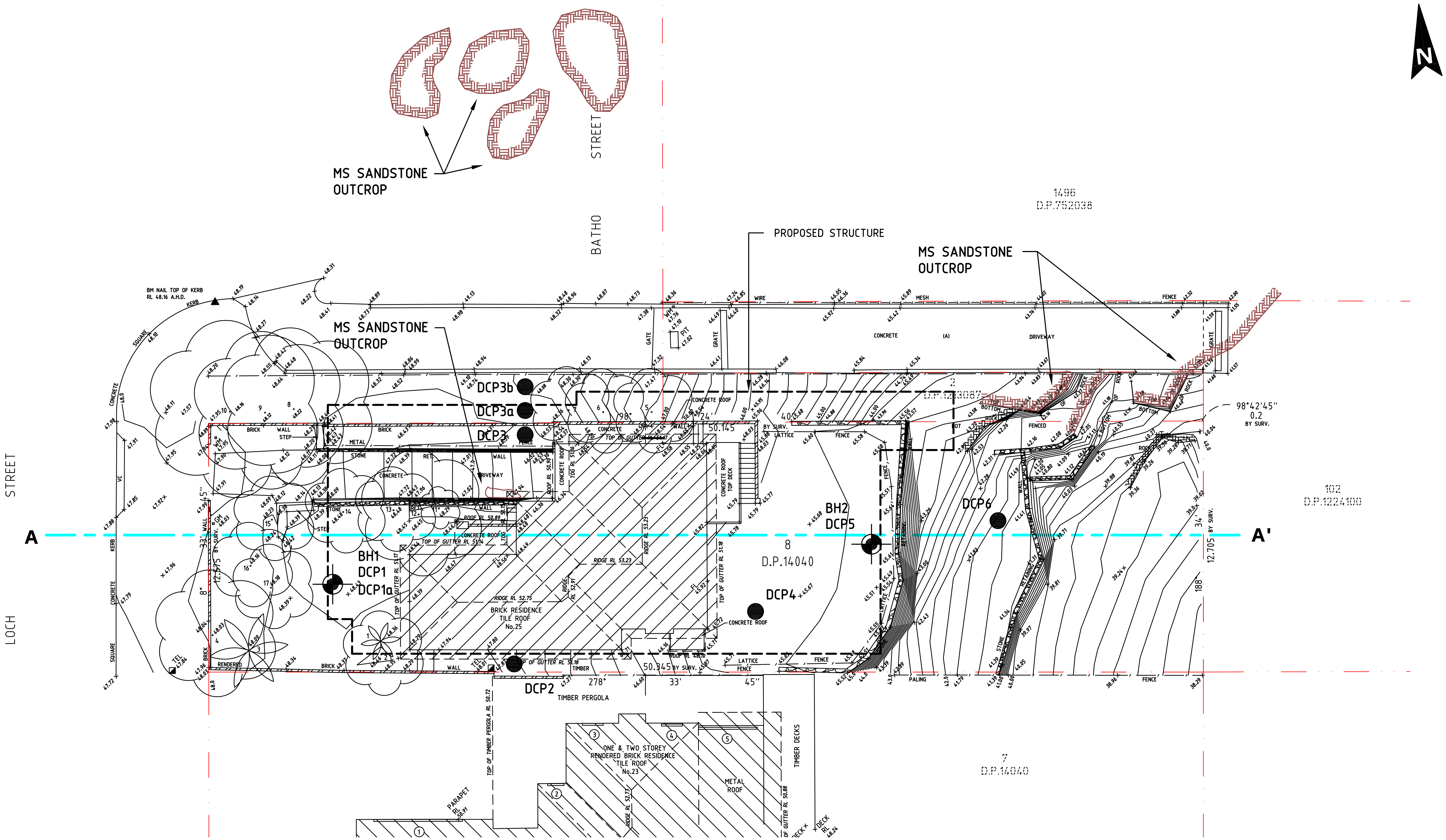
Appendix 2



0 2 4 6 8 10m

4 3 2 1

SCALE 1:200

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

SITE PLAN AND 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
Email: info@croziergeotech.com.au
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

LEGEND

BH- AUGER LOCATION

BH- DCP- AUGER / DYNAMIC PENETRATION TEST

DCP- DYNAMIC PENETRATION TEST

A—A' CROSS SECTION REFERENCE LINE

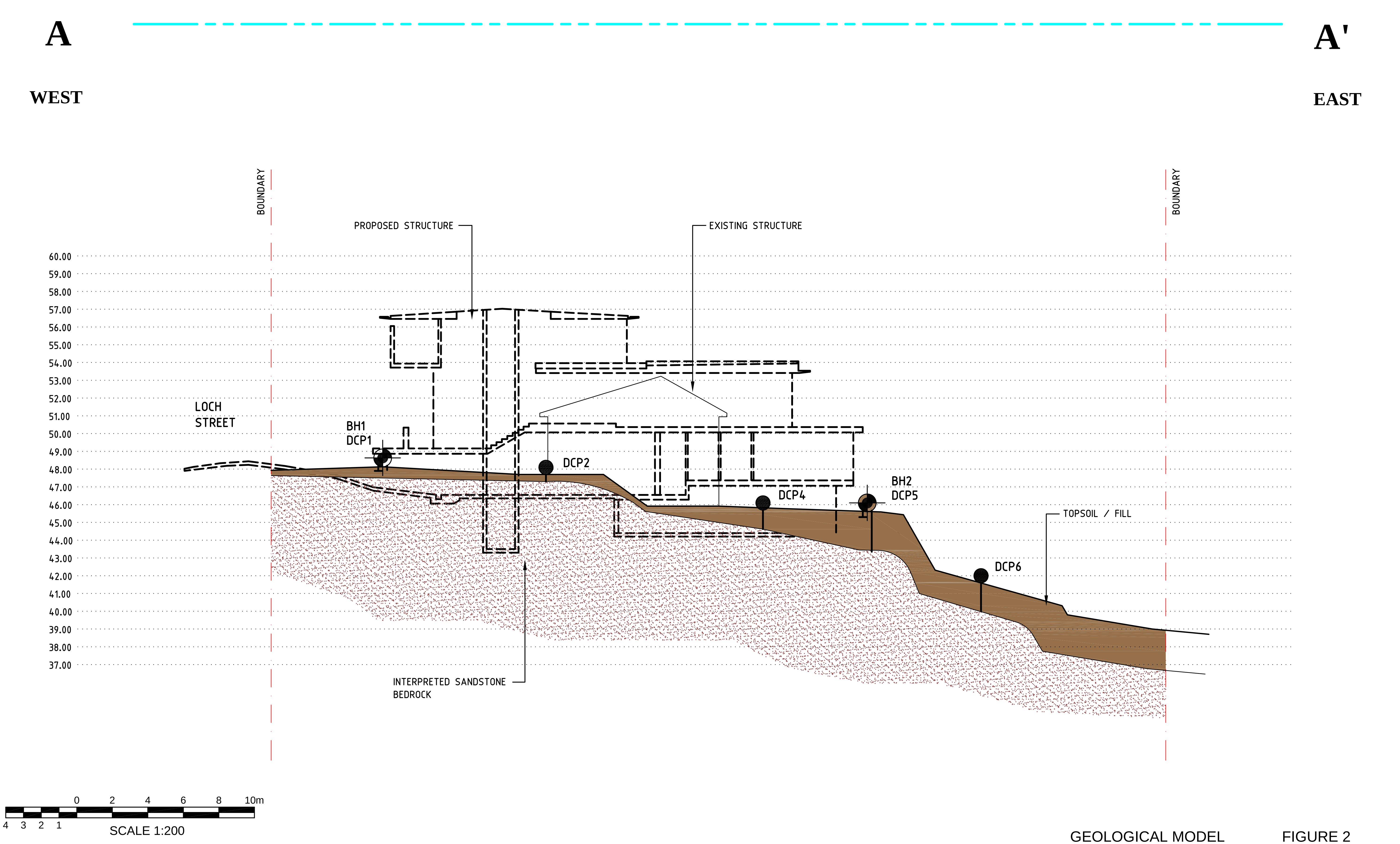
BOUNDARY LINE

SCALE : 1:200
DRAWING : FIGURE 1
DATE : 17.10.18

APPROVED BY : K.N.
DRAWN BY : A.C.W.
PROJECT : 2018-147

PREPARED FOR :
ANDREW & JOE SMITH

ADDRESS :
25 LOCH STREET
FRESHWATER, N.S.W.



GEOLOGICAL MODEL

FIGURE 2

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100

ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

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

LEGEND

- AUGER LOCATION
- DYNAMIC PENETRATION TEST
- TOPSOIL / FILL
- SANDSTONE BEDROCK
- AUGER / DYNAMIC PENETRATION TEST
- BOUNDARY LINE

SCALE :	1:200
DRAWING :	FIGURE 2
DATE :	17.10.18
APPROVED BY :	K.N.
DRAWN BY :	A.C.W.
PROJECT :	2018-147

PREPARED FOR :
ANDREW & JOE SMITH

ADDRESS :
25 LOCH STREET
FRESHWATER, N.S.W.

BOREHOLE LOG

CLIENT: Andrew and Johanna smith

DATE: 20th September 2018 **BORE No.:** 1

PROJECT: New Residential dwelling with basement

PROJECT No.: 2018-147 **SHEET:** 1 of 1

LOCATION: 25 Loch Street, Freshwater

SURFACE LEVEL: RL48.4m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00						
	FILL: Brown grey sandy medium to coarse gravel.					
	Hand auger refusal @ 0.30m depth on at least low strength sandstone.					
1.00						
2.00						

RIG: N/A DRILLER: AR LOGGED: KN

METHOD: Hand auger

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED:

Crozier Geotechnical Consultants

BOREHOLE LOG

CLIENT: Andrew and Johanna smith

DATE: 20th September 2018 **BORE No.:** 2

PROJECT: New Residential dwelling with basement

PROJECT No.: 2018-147 **SHEET:** 1 of 1

LOCATION: 25 Loch Street, Freshwater

SURFACE LEVEL: RL45.6m

Depth (m)	Description of Strata PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks	Sampling		In Situ Testing		
		Type	Depth (m)	Type	Results	
0.00						
	FILL: Brown grey sandy medium to coarse gravel.					
	Hand auger refusal @ 0.40m depth on gravel fill.					
1.00						
2.00						

RIG: N/A

DRILLER: AR LOGGED: KN

METHOD: Hand auger

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Andrew and Johanna smith **DATE:** 20th September 2018

PROJECT: New Residential dwelling with b **PROJECT No.:** 2018-147

LOCATION: 25 Loch Street, Freshwater **SHEET:**

	Test Location							
Depth (m)	DCP1	DCP2	DCP3	DCP3a	DCP3b	DCP4	DCP5	DCP6
0.00 - 0.15	3	1	2	2	1	1	--	--
0.15 - 0.30	4	5	8	4	3	3	--	1
0.30 - 0.45	HB @ 0.30m	HB @ 0.30m	2	5	4	2	1	4
0.45 - 0.60			5	HB @ 0.38m	HB @ 0.38m	4	4	3
0.60 - 0.75			6			3	2	4
0.75 - 0.90			5			19	4	10
0.90 - 1.05			9			36	7	20
1.05 - 1.20			HB @ 0.90m			HB @ 1.20m	3	HB @ 0.95m
1.20 - 1.35							6	
1.35 - 1.50							4	
1.50 - 1.65							5	
1.65 - 1.80							4	
1.80 - 1.95							8	
1.95 - 2.10							5	
2.10 - 2.25							7	
2.25 - 2.40							25	
2.40 - 2.55							HB @ 2.25m	
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								
3.00 - 3.15								
3.15 - 3.30								
3.30 - 3.45								
3.45 - 3.60								
3.60 - 3.75								
3.75 - 3.90								
3.90 - 4.05								

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER
AS 1289. F3.3, PERTH SAND PENETROMETER

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

Appendix 3

TABLE : A

Landslide / risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood	Spatial Impact	Occupancy	Evacuation	Vulnerability	Risk to Life
A	Collapse (Rockslide <10m^3) of bedrock around excavation perimeter due to poorly oriented defects		Excavation up to approximately 4.00m depth within approximately 1.00m of northern and southern boundaries, shallow (≤0.50m) granular fill underlain by VLS and then probable MS - HS sandstone. Some potential for adverse defects to intersect excavation	a) Excavation failure is likely to be localised to small portion of excavation area, b) Excavation 1.0m from property boundary, building structure a further 2.00m, limited impact to pergola area only c) Driveway approximately 1.00m from excavation, expect impact to edge of driveway only	a) Person in basement gym 1 hrs/day, b) Person in pergola 2 hrs/day, c) Persons in access driveway, 10 mins per day	a) Likely to not evacuate, b) Possible to not evacuate , c) Possible to not evacuate.	a) Rare buried/failure into excavation, negigible impact b) Rare buried/failure into excavation, negigible impact c) Person in car, neglible impactunlkiburied/crushed,	
		a) Basement excavation	Possible 0.001	0.10	0.04	0.75	1.00	**3.13E-06**
		b) No.23 Loch Street	0.001	0.05	0.08	0.50	0.05	**1.04E-07**
		c) Access driveway to No.2 Wyandra Avenue	0.001	0.05	0.06	0.50	0.05	**7.50E-08**

* hazards considered in current condition and/or without suitable remedial/stabilisation measures
* likelihood of occurrence for design life of new development (considered 100 years) except for (A)
* considered for person most at risk
* 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)
* 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	Collapse (Rockslide <10m3) of bedrock around excavation perimeter due to poorly oriented defects	a) Basement excavation	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Very Low
		b) No.23 Loch Street	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required, no impact to neighbouring properties.	Very Low
		c) Access driveway to No.2 Wyandra Avenue	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required, 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%.

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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

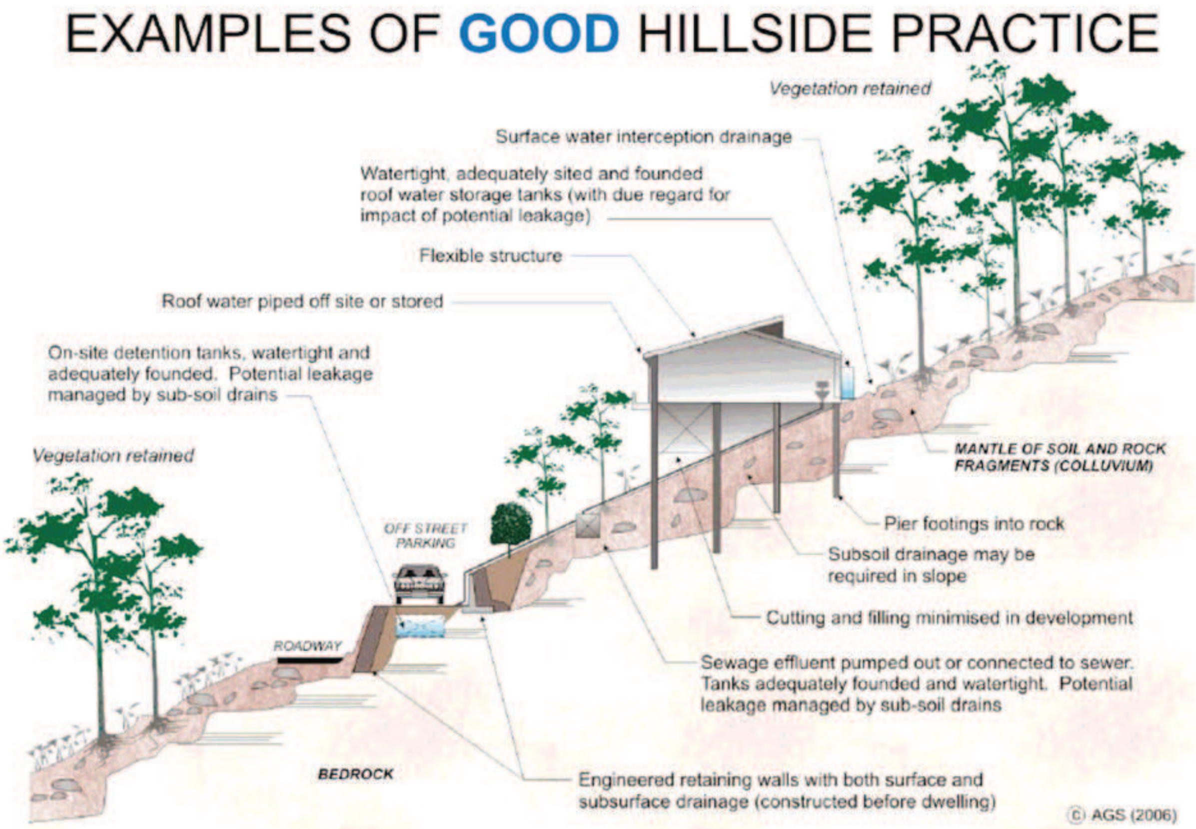
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
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
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
OFF STREET PARKING
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
ROADWAY
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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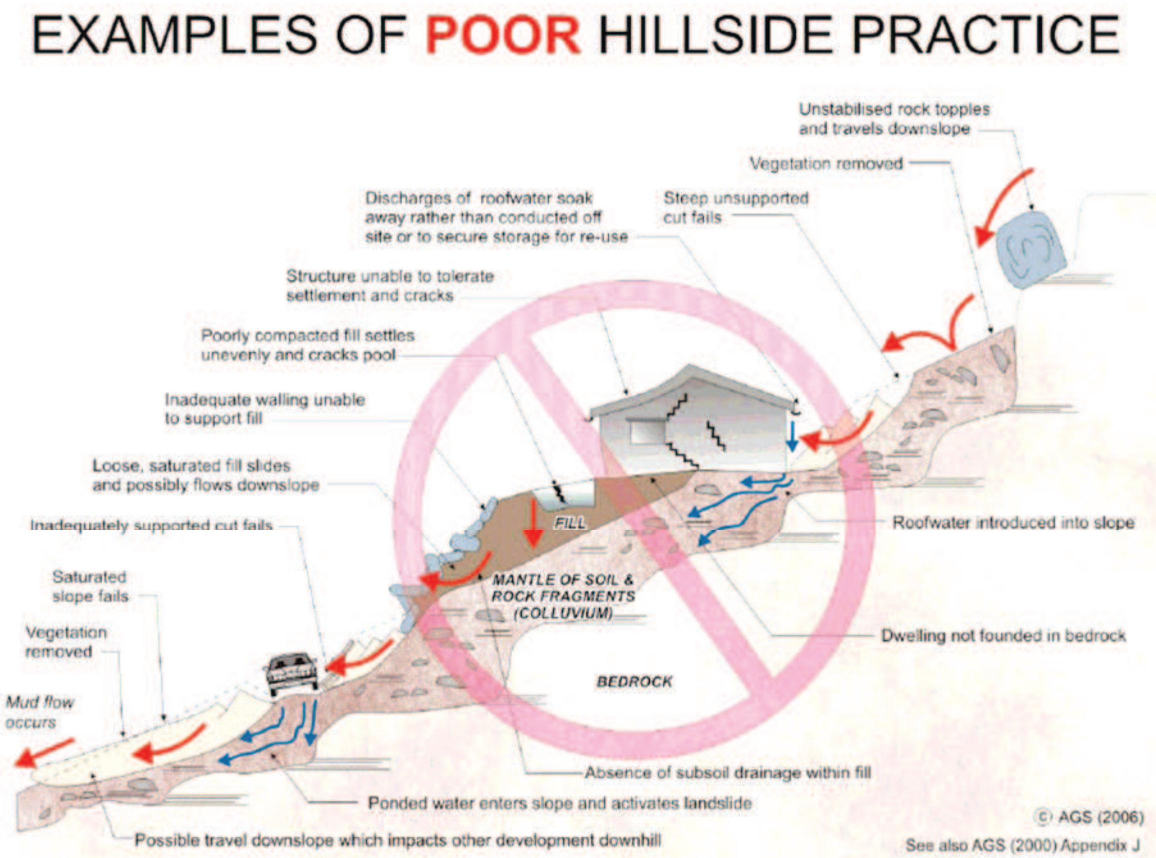
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles 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)
Dwelling not founded in bedrock
Vegetation removed
BEDROCK
Mud flow occurs
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
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Possible travel downslope which impacts other development downhill
See also AGS (2000) Appendix J