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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED NEW DEVELOPMENT

at

1010-1014 PITTWATER ROAD, COLLAROY, NSW

Prepared For

Collaroy Projects Pty Ltd ATF Collaroy Projects Unit Trust

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TABLE OF CONTENTS

APPENDICES

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GEOTECHNICAL REPORT FOR PROPOSED NEW DEVELOPMENT
1010-1014 PITTWATER ROAD, COLLAROY, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation and assessment carried out for a proposed new development at No. 1010-1014 Pittwater Road, Collaroy, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of the Architect Gartner Trovato on behalf of the clients Collaroy Projects Pty Ltd ATF Collaroy Projects Unit Trust.

The proposed works involve the demolition and clearing of the existing site structures and the subsequent construction of a new four and five storey residential/commercial development. The proposed structure will feature a basement level excavated up to 4.00m below existing ground levels which will extend to all site boundaries except the eastern boundary where it will be setback approximately 5.50m.

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

This site is located within landslip risk Class 'A' within the Landslip Risk Map _ Sheet LSR_009. A review of the preliminary checklist and the proposed works identified that the Development Application (DA) involves works which exceed the preliminary assessment guidelines.

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

The geotechnical investigation included:

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

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

- Architectural Drawings – Gartner Trovato, Project No.: 2101, Drawing No.: DA-03-18, Dated: 17/07/2023

2. PROPOSED DEVELOPMENT:

The proposed works involve the demolition and clearing of the existing site structures and the subsequent construction of a new four and five storey residential/commercial structure. The structure will feature a basement level set at FFL 10.230m which will require bulk excavation below existing ground levels to a maximum depth of approximately 4.00m. The basement excavation is proposed to abut all site boundaries except the eastern boundary where it will be setback 5.47m.

3. SITE FEATURES:

3.1. Description:

The site is an irregular shaped block that comprises the three separate properties of No. 1010, 1012 and 1014 Pittwater Road. The site is situated on the eastern side of the road within relatively flat to gentle east dipping topography. The ground levels within the site extend from a high of approximately RL14.20m along the front boundary adjacent to Pittwater Road to a low of approximately RL12.85m along the rear eastern boundary. An aerial photograph of the site location is shown below in Photograph 1, as sourced from NSW Government Six Map spatial data.

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

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet indicates that the site is located near the boundary of quaternary sands (Qha, Qhd) as well as the Newport formation (Rnn) which is of middle Triassic age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones that tend to deep weathering and produce a weak rock mass containing clay bands and fracturing. The quaternary sands Qhd comprise a medium to fine grained "marine" sand with podsols whilst Qha comprises a silty to peaty quartz sand, silt and clay featuring ferruginous and humic cementation in places as well as common shell layers.

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

4. FIELD WORK:

4.1. Investigation Methods:

The field investigation comprised a geotechnical inspection of the site and adjacent land on 26th July 2023 by a Geotechnical Engineer. It involved a photographic record of site conditions as well as geotechnical assessment of the site and adjacent land with examination of existing site structures and inspection of neighbouring structures.

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

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

Dynamic Cone Penetrometer (DCP) testing was carried out from the ground surface adjacent to boreholes and at one additional location. DCP tests were undertaken in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer” to estimate near surface soil conditions and indicate depths to bedrock/boulders.

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

4.2. Field Observations:

The site is situated on the eastern side of Pittwater Road within relatively flat to gentle east dipping topography. Pittwater Road comprises a concrete pavement which is separated from the site via a concrete kerb, gutter and pedestrian pathway. There were no signs of excessive settlement or cracking observed within any of the structures adjacent to the front of the site to suggest any underlying geotechnical concern.

The front of the properties that comprise the site each contain masonry structures abutting the front boundary with commercial retail outlets at ground floor level and the structures within No. 1010 and No. 1014 additionally featuring a second storey residential level. The structures appeared to be of >50 years construction age and the visible aspects of the structures did not exhibit any signs of significant cracking or excessive settlement to indicate any impending geotechnical concern.

Access to the rear of No. 1010 and 1012 was gained via a concrete pathway that extends perpendicular from the roadway, between the respective structures. The rear of the properties both comprised gently east dipping lawn areas interspersed with minor shed structures. The rear of No. 1014 was accessed via a pathway that bisects the main structure with the rear portion of the property similarly comprising an open lawn area with concrete floor slabs as well as an ancillary single storey masonry storage structure.

The neighbouring property to the south (No. 1000-1008 Pittwater Road) comprises a two and three storey masonry residential and commercial structure that abuts the shared boundary and appears to be of relatively recent (<15 years) construction age. It is understood that the structure features a basement level garage however the extent and depth of the structure is unconfirmed. The visible aspects of the structure appeared to be in good condition with no signs of excessive settlement or cracking to indicate any underlying geotechnical concern.

Two separate properties shared a common boundary with the site to the east (No. 12 & No. 14 Cliff Road). Both of which comprise single storey masonry dwellings setback from the shared boundary by a minimum of 10m with the rear gardens comprising relatively open lawn areas. Inspection was limited by boundary structures and vegetation however the neighbouring structures appeared to be in good condition with no signs of excessive settlement or cracking to indicate an impending geotechnical concern.

Two separate properties share a common boundary with the site to the north (No. 1016 Pittwater Road & No. 26 Ocean Grove). No. 26 Ocean Grove contains a two storey masonry residential/commercial structure that is understood (although unconfirmed) to feature a basement level garage. No. 1016 Pittwater Road contains a two and three storey residential and commercial structure. Both properties feature structures that abut the shared site boundary however neither exhibited signs of significant cracking or excessive settlement to indicate any underlying geotechnical concern.

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

4.3. Ground Conditions:

The boreholes (BH1-BH3) were drilled within the open rear portion of the site within the footprint of proposed works. All encountered a relatively shallow layer of topsoil/fill material from ground surface to a maximum depth of 0.50m. Underlying the topsoil fill natural sandy soils were encountered which extended to the maximum investigation depth of 4.00m.

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

- **TOPSOIL/FILL** – This layer was encountered in the test locations and extended to a maximum depth of 0.50m. It generally comprised a loose, dark brown, moist silty sand with building refuse and roots.
- **NATURAL SANDY SOILS –** Natural sandy soils were intersected underlying the topsoil/fill in all boreholes and extended to the maximum investigation depth of 4.00m. These soils varied in composition both with depth and between points with zones of increased clay, silt, quartz gravels and iron content encountered with negligible correlation between test points. Despite the varied composition, the soils graded fairly uniformly in density/stiffness with depth. With effective DCP refusal in very dense/hard natural soils intersected at depths varying between 1.90m (DCP3 & DCP4) and 2.70 (DCP1).

Significant seepage/standing water was encountered in all boreholes at varying depths. Saturated soils were intersected in BH1, BH2 and BH3 at depths of 1.00m, 2.80m and 2.00m respectively.

5. GEOTECHNICAL ASSESSMENT:

5.1. Comments:

The site investigation identified the presence of a relatively shallow layer of topsoil/fill underlain by natural sandy soils of varied composition that graded in density with depth and extended to the maximum investigation depth of 4.00m without bedrock intersection. Significant seepage levels were identified in all test locations with all boreholes encountering refusal within saturated soils at depths ranging from 1.95m to 4.00m however minimal correlation was observed between test points, indicating likely flow along sandy units as opposed to a static groundwater table, though this is unconfirmed. Investigation was only limited to hand tools within the rear of the site due to the existing structures.

The proposed works involve the demolition and clearing of the existing site structures and the subsequent construction of a new four and five storey residential/commercial development which will abut the majority of site boundaries. The proposed structure will feature a basement carpark level which will require bulk excavation to a maximum anticipated depth of 4.00m and will extend to the north, south and west site boundaries with setbacks to the eastern site boundary >5.00m.

The excavation is anticipated to extend through the shallow layer of topsoil/fill prior to intersection of natural sandy soils which will continue to the base excavation level with no hard rock intersection expected though conditions as the front of the site could vary. The excavation should be anticipated to encounter significant seepage as it progresses which will likely vary with localised rainfall levels.

Based on the negligible setbacks to neighbouring boundaries, safe batter slopes will be unachievable within the confines of the site with respect to the neighbouring properties to the north, south and west. Therefore, pre-excavation support which should consist of pile (or similar) shoring walls will need to be implemented prior to bulk excavation to ensure stability external to the site where safe batter slopes cannot be achieved however it would be prudent to extend any support along all excavation edges.

The founding conditions if neighbouring structures are unknown and should be considered as founded at shallow depth unless proven otherwise. As such, they will surcharge excavation support walls and will be highly susceptible to settlement and erosion from the adjacent excavation.

Due to the generally granular nature of the natural soils as well as the limited setbacks, a contiguous system will be required to avoid erosion of soils between piles. Where excavation is adjacent to boundaries or structures then careful control of pile drilling/support installation is required to avoid over excavation whilst all gaps in the wall must be sealed during excavation to prevent erosion between piles. Due to the relatively high seepage levels encountered across site, it is envisaged that open bored methods will not be feasible, therefore a CFA rig will likely be required to ensure successful installation of piles. Also, secant piling will be required to maintain low inflows during excavation and reduce pumping.

Driven style support systems (i.e. sheet piling, concrete/timber piles) are not suitable for use on this site due to ground vibration compaction in the adjacent sands. Also, care will need to be exercised during demolition and large scale breakers should be avoided to prevent damage to neighbouring structures. Where breakers >300kg are used, full time monitoring will be required to ensure ground vibrations are maintained below 8mm/s PPV.

Due to the proximity of the pile walls to neighbouring properties/structures, it is assumed that negligible deflection will be tolerated. As such, provisions should be made to incorporate temporary bracing or anchoring until installation of permanent bracing (in the form of the proposed structure) has been successfully implemented. Structural and geotechnical analysis of potential wall deflections will also be required to allow monitoring during excavation activity.

All new footings will need to bear within soils of similar density/strength in order to minimise the risk of differential settlement. Preliminary values for shallow footings are provided in Section 5.3. It is possible/likely that footings founded to bedrock may be required however this unit was not found in the investigation range, dictating the requirement of further investigation. It is anticipated from previous works in the local area to be weathered siltstone/claystone at >7-8m depth.

It is understood that a Sydney Water (SW) asset underlies the site in the vicinity of the eastern boundary. CGC has not undertaken any investigation into the construction/type/depth etc. of the asset. However, it is indicated as comprising a 150mm vitrified clay pipe which appears to lie at a depth of approximately 1.70m below ground levels. Based on both the asset depth and scope of proposed works, as well as Sydney Water's "Technical Guidelines for Building Over and Adjacent to Pipe Assets, 2015", it is anticipated that a Specialist Engineering Assessment (SEA) will be required for the Construction Certificate (CC) phase of the proposed works.

Due to the geological conditions identified and the limited access to the front of the site, it is recommended that a staged Construction Certificate be applied if required to allow demolition of existing structures and access for mechanical drilling investigation equipment. It will also be required to confirm conditions below excavation level at rear of the site. Investigation should comprise at least four boreholes and it is expected that they will need to extend to >3m below the Base Excavation Level (BEL) to assist engineering design and meet regulation requirements.

The significant seepage observed within the investigation likely dictates the requirement for the basement structure to be fully "tanked" with the excavation potentially requiring a dewatering regime. It is recommended that additional investigation into groundwater/seepage levels be incorporated into the subsequent site investigation.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tools. This test equipment provides limited data from small, isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations and below DCP refusal depths. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

5.2. Site Specific Risk Assessment:

The primary hazard is considered as:

A. Landslip of soils from excavation (<10m^3)

The hazards have been assessed in accordance with the methods of the Australian Geomechanics Society (Landslide Risk Management, AGS Subcommittee, May 2002 and March 2007), see Tables: A and B, Appendix: 3 The Australian Geomechanics Society Qualitative Risk Analysis Matrix is enclosed in Appendix: 4 along with relevant AGS notes and figures. The frequency of failure was interpreted from existing site conditions and previous experience in these geological units.

The **Risk to Life** from **Hazard A** was estimated to be **1.88 x 10^{-5}** for a single person with expected higher levels due to occupancy of the adjacent structures, whilst the **Risk to Property** from the hazards were considered to be **'Very High'**.

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

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'S' due to prevalence of clays/silts in natural soils
Type of Footing	Strip/Pad/Piles
Sub-grade material and Maximum Allowable Bearing Capacity	- Medium dense sand: 150kPa - Dense sand: 250kPa - Very dense sand: 300 kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e – Shallow Soil Site (interpreted)
Remarks: All footings for the proposed structure should be founded off material of similar strength to prevent differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

5.3.2. Excavation:

Basement Excavation

Table 1: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No. 1016 Pittwater Road/No. 26 Ocean Grove	Dwelling	Up to 4.00m	Nil	Nil
South	No. 1000-1008 Pittwater Road	Dwelling			
East	No. 12 & No. 14 Cliff Road	Garden, Dwelling		5.50m	Dwellings a further 10.00m
West	Pittwater Road	Pedestrian Pathway, Roadway		Nil	Pathway immediately adjacent, roadway a further 2.00m

Type of Material to be Excavated	Topsoil/Fill and residual soils to depths ≤ 0.50m. Natural sandy soils, Medium dese to Very Dense/Hard to >4.00m depth

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

Material	Safe Batter Slope (H: V)	
	Short Term/Temporary	Long Term/Permanent
Fill and sandy soils	1:1*	2:1*

* Dependent on assessment by engineering geologist/geotechnical engineer

Remarks:

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

Equipment for Excavation	Fill/natural soils	Excavator with bucket
Full time vibration Monitoring Required	Pending proposed demolition equipment	
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Inspection of any temporary and permanent batter slopes, • During installation of pre-excavation support • Where ground conditions are exposed that differ to those than expected	
Dilapidation Surveys Requirement	Recommended within 10m of excavation perimeter to avoid spurious claims of damage	

Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope. It is recommended that a drainage excavation extend to below floor slab levels to reduce the potential for dampness problems in the completed structure.

5.3.3. Retaining Structures:

Required	New retaining structures are required as part of the proposed development. Pre-excavation support is necessary in most locations
Types	Steel reinforced concrete/concrete block walls post excavation where temporary batters are possible, Bored pier pre-excavation support where temporary batters unachievable. Contiguous for the sandy soils however likely requiring secant to maintain low groundwater inflows. Expected to be braced/anchored to maintain deflection limits. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.

Preliminary 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)	
Sandy Fill and Loose Sand	18	ϕ' = 28°	0.35	0.52	N/A
Medium dense sands	18	ϕ' = 30°	0.33	0.50	3.00
Dense and very dense sands	20	ϕ' = 35°	0.27	0.43	3.69

Remarks: In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

Retaining structures near site boundaries or supporting existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Yes, significant seepage within sandy soils at varying depths
Excavation likely to intersect	Water Table	No
	Seepage	Yes, within natural sands
Site Location and Topography		Eastern side of the road within relatively flat to gentle east dipping topography at base of a slope
Remarks: Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow down slope. This can result in relaxation of excavation faces causing instability prior to installation of permanent retention systems. Therefore, excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional. A stormwater diversion drain should be installed upslope of excavation crests to intercept stormwater runoff and prevent erosion and softening of the excavation faces. An excavation trench should also be installed at the base of excavation cuts to below floor slab levels to reduce the risk of long term dampness and the proposed structure will likely need to be fully "tanked". Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer.		

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Undertake rig boreholes across site post demolition to provide updated geotechnical design and monitoring parameters and recommendations
2. Review the structural drawings, for inclusion the recommendations of this report,
3. Conduct inspections as per the recommendations of Section 5.3 in this report
4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect the completed development to ensure all retention and stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

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

6. CONCLUSION:

The site investigation identified the presence of a relatively shallow layer of topsoil/fill underlain by natural sandy soils featuring varying compositional zones of increased clay, silt, quartz and iron. The soils generally graded in density with depth such that effective DCP refusal was encountered at levels varying between 1.90m and 2.80m. Significant seepage was observed at varying depths across site and resulted in borehole refusal due to continual collapse in saturated soils at depths varying from 1.95m to 4.00m.

The proposed works involve the demolition and clearing of site structures and the subsequent construction of a new four and five storey residential/commercial structure featuring a basement level excavated up to 4.00m below existing ground levels. The excavation will abut the north, south and western site boundaries whilst being setback from the rear eastern boundary by 5.50m.

The excavation will extend through predominantly natural sandy soils with no hard rock excavation equipment anticipated. Safe batter slopes will not be feasible for the basement excavation and pre-excavation support in the form of a braced contiguous pile wall (or similar) will be required. Pile installation will likely require the use of a CFA rig to prevent hole collapse in the saturated soils.

All new footings will need to bear within materials of similar strength/density to minimise the risk of differential settlement.

The Sydney Water asset within the rear garden will likely require a Specialist Engineering Assessment (SEA) prior to site works, it is recommended that Sydney Water be contacted as soon as possible to confirm requirements.

Due to the geological conditions identified and the limited access to the front of the site, it is recommended that a staged Construction Certificate be applied to allow demolition of existing structures and access for mechanical drilling equipment for further detailed geotechnical investigation.

The significant seepage observed within the investigation likely dictates the requirement for the basement structure to be fully “tanked” with the excavation potentially requiring a dewatering regime. It is recommended that additional investigation into groundwater/seepage levels be incorporated into the subsequent site investigation.

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

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

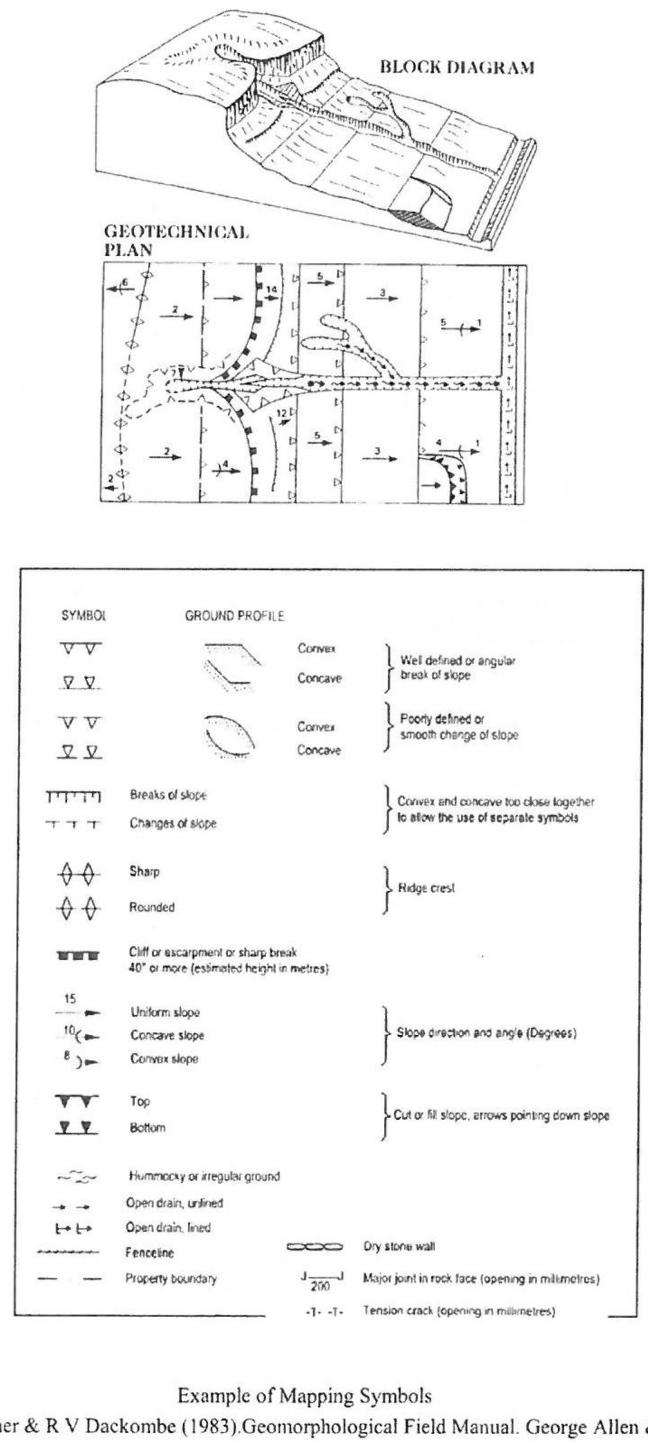
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

Reproduction of Information for Contractual Purposes

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

Site Inspection

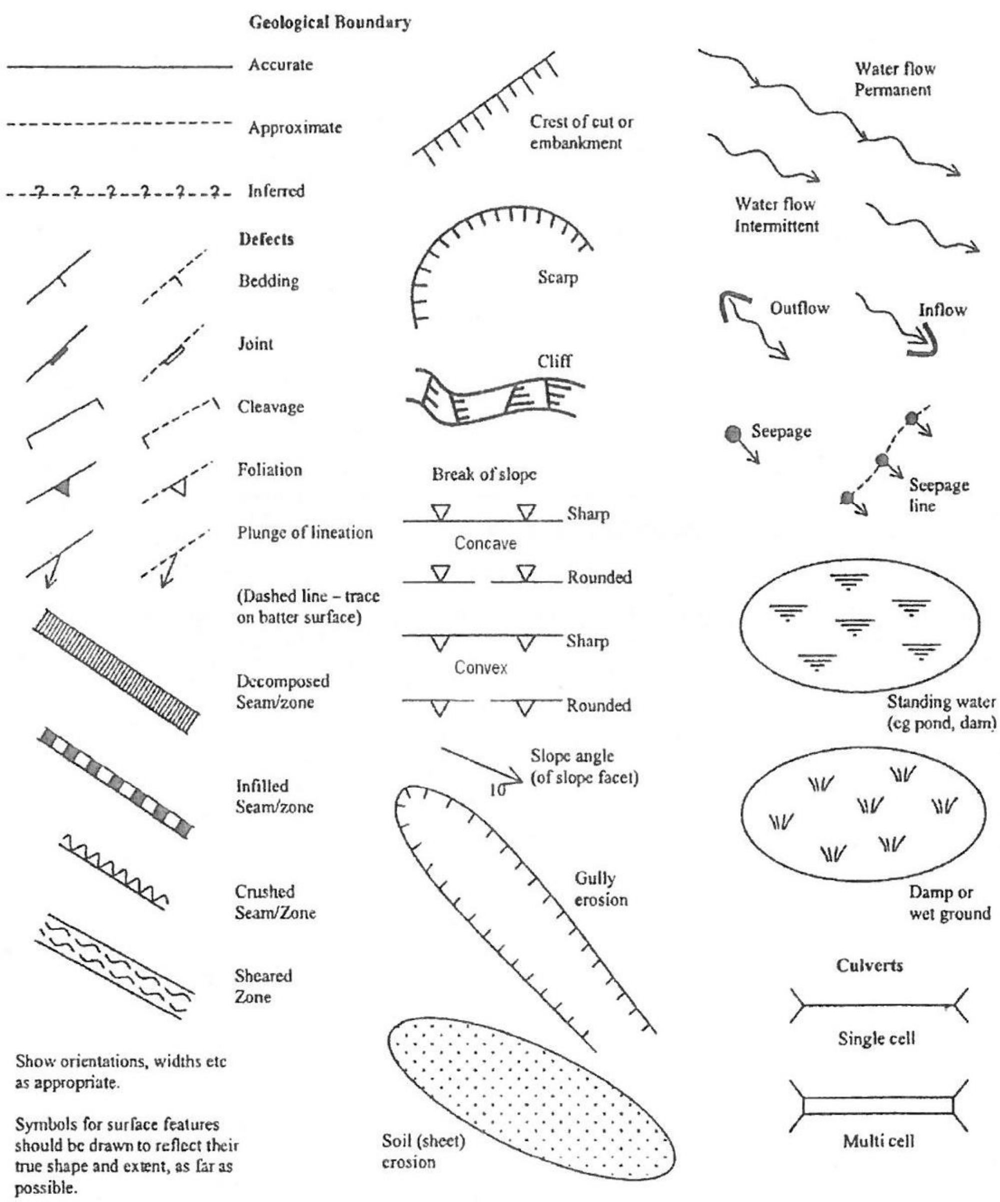
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.



Example of Mapping Symbols

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

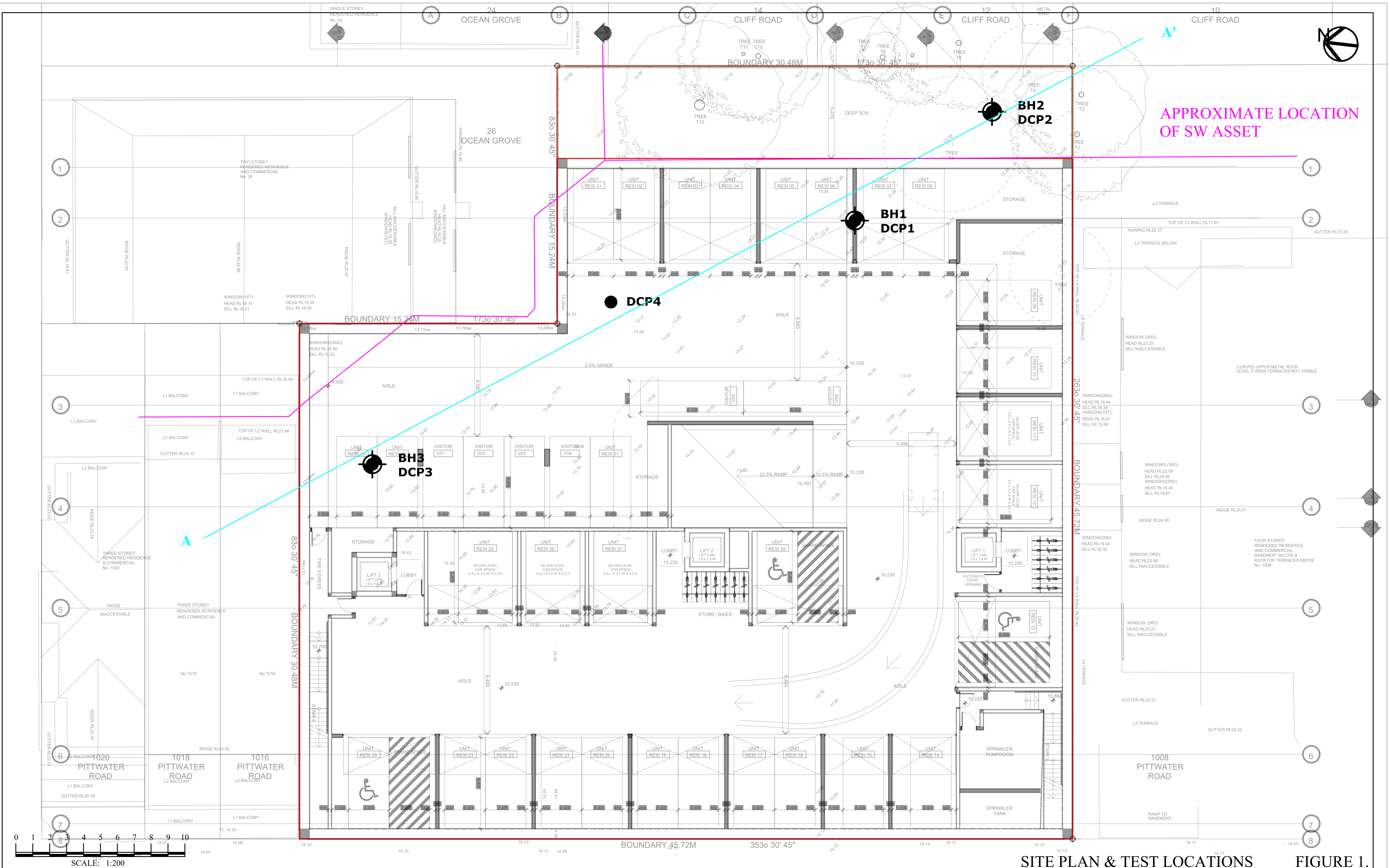
APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 2



SITE PLAN & TEST LOCATIONS

FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

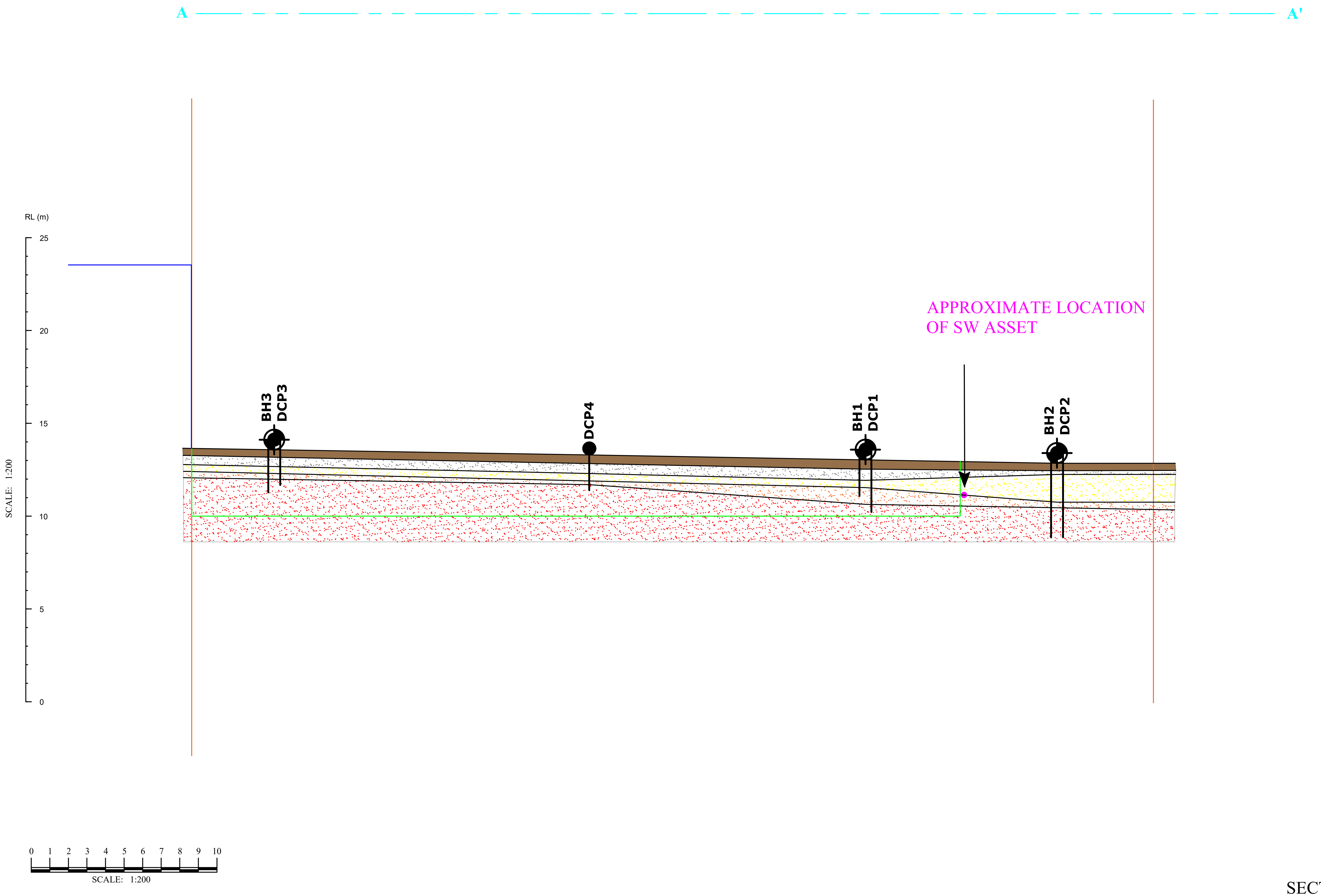
LEGEND

PROPOSED WORKS	red line
EXISTING STRUCTURES	blue line
PROPERTY BOUNDARY	orange dashed line
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
A — A'	CROSS-SECTION REFERENCE LINE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	05 /2023
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2023-084

PREPARED FOR:
Nicole and Steve Coppola

ADDRESS:
32 Wentworth Street, Dover Heights



SECTION A: 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
Fax: (02) 9939 1883

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

LEGEND

- PROPOSED EXCAVATION OUTLINE
- EXISTING STRUCTURES
- SITE BOUNDARIES
- FILL
- VERY LOOSE TO LOOSE NATURAL SANDS
- MEDIUM DENSE NATURAL SANDS
- DENSE NATURAL SANDS
- VERY DENSE NATURAL SANDS
- A —— A' SECTION LINE

SCALE:	1:200 @ A3	PREPARED FOR: Collaroy Projects Pty Ltd
DRAWING:	FIGURE 2	
DATE:	07/2023	
APPROVED BY:	TMC	ADDRESS: 1010-1014 Pittwater Road
DRAWN BY:	JD	
PROJECT:	2023-150	

BOREHOLE LOG

CLIENT: Collaroy Projects Pty Ltd **DATE:** 26/07/2023 **BORE No.:** 1

PROJECT: New residential & Commercial development **PROJECT No.:** 2023-150 **SHEET:** 1 of 1

LOCATION: 1010-1014 Pittwater Road, Collaory **SURFACE LEVEL:** RL 13.40m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
0.00			Type	Tests	Type	Results
		Topsoil/Fill: loose, brown, moist silty sand with building refuse and roots				
0.50	SM	Silty SAND: dark grey, loose, fine to medium grained moist/wet sand with silt and trace clay				
0.80		… pale grey, no clay, wet				
1.00	SM/SC	… saturated with yellow brown clay mottle	D	1.00		
				1.10		
1.30			D	1.30		
	SC/CL	SANDY CLAY/CLAYEY SAND: medium dense/ grey with yellow brown mottle, wet		1.40		
1.60		… with red and yellow mottle				
1.70		… increase in iron content (red zones)		1.70		
			D	1.80		
1.95		… dense	D	1.90		
		Effective hand auger refusal due to continual hole collapse of saturated sands, DCP extended to 2.80m				
2.40		… very dense				

RIG: Not applicable DRILLER: SK

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Significant seepage below 0.60m, standing water below 2.00m

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Collaroy Projects Pty Ltd **DATE:** 26/07/2023 **BORE No.:** 2

PROJECT: New residential & Commercial development **PROJECT No.:** 2023-150 **SHEET:** 1 of 1

LOCATION: 1010-1014 Pittwater Road, Collaory **SURFACE LEVEL:** RL 12.88m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		Topsoil/Fill: loose, brown, moist silty sand with building refuse and roots				
0.40	SM	SILTY SAND: loose, pale grey, fine to medium grained sand with silt				
0.60		… with yelow brown clay zones				
0.80	SM/SC	… pale yellow/grey with some clay				
0.90	SC/CL	SANDY CLAY/CLAYEY SAND: medium dense, grey with yellow/brown mottle, moist				
1.50	SC	… grey no mottle, decrease in clay content				
			D	1.80		
2.00		… dense, increase in clay content		2.00		
2.30		… very dense				
2.80	SW	… decrease in silt and clay content, increase in sand, wet/saturated				
			D	3.00		
				3.20		
3.50	SM	… increase in silt				
3.80	SC	… increase in clay, decrease in silt	D	3.80		
				3.90		
4.00		Effective hand auger refusal due to continual hole collapse of saturated sands				

RIG: Not applicable DRILLER: SK

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Standing water below 2.80m

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Collaroy Projects Pty Ltd **DATE:** 26/07/2023 **BORE No.:** 3

PROJECT: New residential & Commercial development **PROJECT No.:** 2023-150 **SHEET:** 1 of 1

LOCATION: 1010-1014 Pittwater Road, Collaory **SURFACE LEVEL:** RL 13.80m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		Topsoil/Fill: loose, brown, moist silty sand with building refuse and roots				
0.40	SM	Silty SAND: dark grey, loose, fine to medium grained sand with silt				
0.60		… moist/wet pale grey				
0.90		… medium dense				
1.00	SM/SC	… with yellow brown mottle and trace clay				
1.30		… dense				
1.60		… very dense				
2.00	SC/CL	CLAYEY SAND/SANDY CLAY: very dense, pale grey/yellow clayey sand with quartz grains/gravels, saturated	D	2.15 2.30		
2.30		Effective hand auger refusal due to continual hole collapse of saturated sands				

RIG: Not applicable DRILLER: SK

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Significant seepage below 0.60m , standing water below 1.00m

REMARKS: CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Collaroy Projects Pty Ltd **DATE:** 26/07/2023

PROJECT: New residential & Commercial development **PROJECT No.:** 2023-150

LOCATION: 1010-1014 Pittwater Road, Collaroy **SHEET:** 1 of 1

	Test Location									
Depth (m)	1	2	3	4						
0.00 - 0.10	2	0	0	0						
0.10 - 0.20	1	2	1	1						
0.20 - 0.30	2	1	1	1						
0.30 - 0.40	1	2	2	2						
0.40 - 0.50	5	2	1	2						
0.50 - 0.60	13	3	2	1						
0.60 - 0.70	0	5	1	4						
0.70 - 0.80	0	7	1	2						
0.80 - 0.90	1	10	1	0						
0.90 - 1.00	1	7	3	1						
1.00 - 1.10	1	5	4	3						
1.10 - 1.20	3	7	5	4						
1.20 - 1.30	4	12	6	5						
1.30 - 1.40	4	12	7	6						
1.40 - 1.50	6	8	8	7						
1.50 - 1.60	10	6	9	9						
1.60 - 1.70	16	5	10	12						
1.70 - 1.80	9	5	14	17						
1.80 - 1.90	6	4	16	20						
1.90 - 2.00	5	5	End	End						
2.00 - 2.10	7	7								
2.10 - 2.20	7	7								
2.20 - 2.30	7	9								
2.30 - 2.40	7	10								
2.40 - 2.50	10	14								
2.50 - 2.60	11	16								
2.60 - 2.70	13	-								
2.70 - 2.80	15	-								
2.80 - 2.90	End	-								
2.90 - 3.00		-								
3.00 - 3.10		-								
3.10 - 3.20		-								
3.20 - 3.30		-								
3.30 - 3.40		-								
3.40 - 3.50		-								
3.50 - 3.60		-								
3.60 - 3.70		-								
3.70 - 3.80		-								
3.80 - 3.90		-								
3.90 - 4.00		30								

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 of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (soil slide ≤10m³) at crest of excavation		Excavation to 4.0m depth through sandy soils	a) Dwelling immediately adjacent to excavation, impact 20% b) Dwelling immediately adjacent to excavation, impact 30% c) Dwelling immediately adjacent to excavation, impact 10% d) Pathway immediately adjacent to excavation, impact 30%		a) Person in house 20hrs/day avge. b) Person in house 20hrs/day avge. c) Person in house 20hrs/day avge. d) person on pathway 3hrs/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate d) Unlikely to not evacuate	a) Person in building minor damage only b) Person in building minor damage only c) Person in building minor damage only d) Person in open space, buried	
			Likely	**Prob. of Impact**	**Impacted**				
		a) Dwelling of No.1000-1008 Pittwater Road	0.01	0.25	0.20	0.8333	0.75	0.05	**1.56E-05**
		b) Dwelling of No.1016 Pittwater Road	0.01	0.20	0.30	0.8333	0.75	0.05	**1.88E-05**
		c) Dwelling of No.26 Ocean Grove	0.01	0.10	0.10	0.8333	0.75	0.01	**6.25E-07**
		d) Pittwater Road Pedestrian Pathway	0.01	0.25	0.30	0.1250	0.5	1.00	**4.69E-05**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems
* likelihood of occurrence for design life of 100 years
* Spatial Impact -

* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <10m³) from soils at crest of excavation for lower ground floor	a) Dwelling of No.1000-1008 Pittwater Road	Possible	The event could occur under adverse conditions over the design life.	Catastrophic	Site structures completely destroyed, significant stabilising or MAJOR damage to neighbouring property.	Very high
		b) Dwelling of No.1016 Pittwater Road	Possible	The event could occur under adverse conditions over the design life.	Catastrophic	Site structures completely destroyed, significant stabilising or MAJOR damage to neighbouring property.	Very High
		c) Dwelling of No.26 Ocean Grove	Possible	The event could occur under adverse conditions over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	High
		d) Pittwater Road Pedestrian Pathway	Possible	The event could occur under adverse conditions over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	High

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

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

Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

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

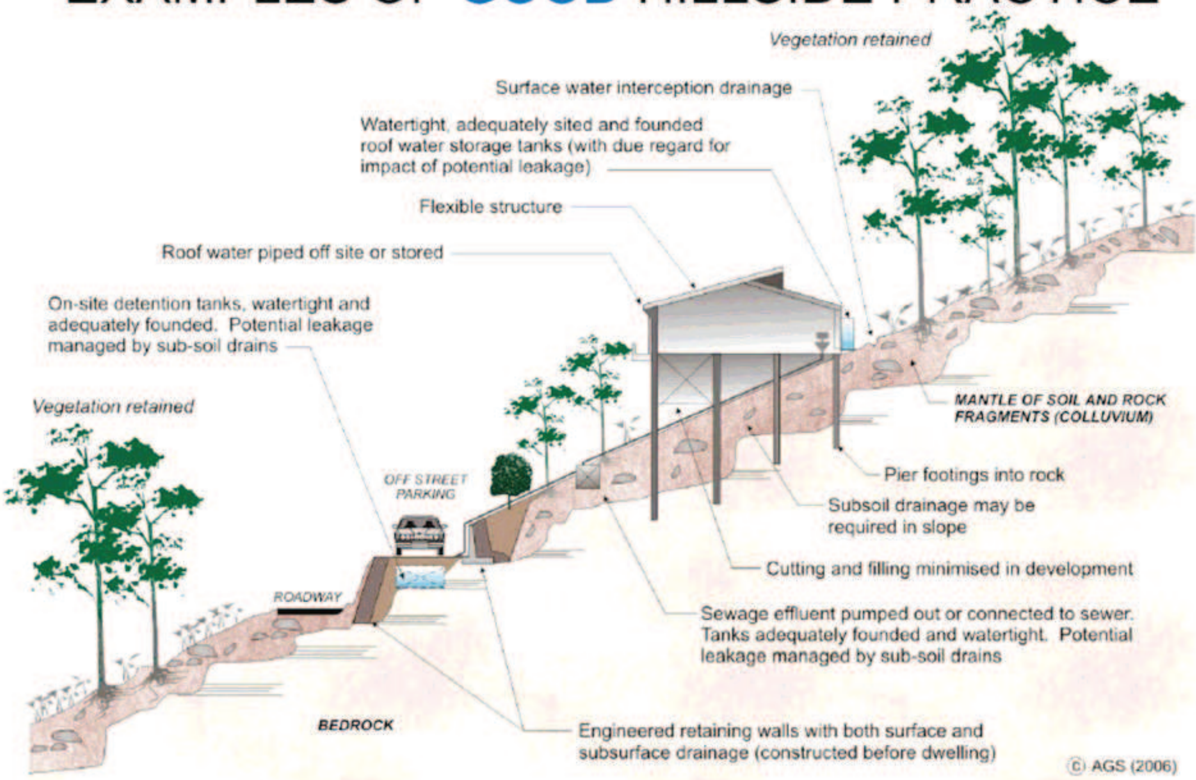
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE