Crozier Geotechnical Consultants
Unit 12/ 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd
ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED SWIMMING POOL

at

47 CUTLER ROAD, CLONTARF

Prepared For

Isabelle Durrenburger

Project No.: 2020-235

December, 2020

Document Revision Record

Issue No	Date	Details of Revisions
0	1st December 2020	Original issue

TABLE OF CONTENTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants a division of PJC Geo-Engineering Pty Ltd

Date: 1st December 2020
Project No: 2020-235

GEOTECHNICAL REPORT FOR NEW POOL
47 CUTLER ROAD, CLONTARF, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for the construction of a new pool at No.47 Cutler Road, Clontarf, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Isabelle Durrenburger.

It is understood that the proposed works involve the construction of a soft landscaped upper lawn (RL48.10m), a lower garden with seating and a fire pit (RL47.74m) and swimming pool (water line RL47.50m) within a lawn area to the rear of the site residence. Bulk excavation will be required to approximately 1.5m depth for the proposed new pool which will require the demolition of an existing retaining wall supporting the south end of the lawn.

The majority of the site is located within Landslip Risk Class 'G2', whilst the south-east corner of the block is within Landslip Risk Class 'G1' as identified within Northern Beaches (Manly) Councils – Development Control Plan 2013 – Schedule 1 Map C.

A review of the proposed development works and comparison against Council Slope Stability Checklist indicates the Development Application requires a site stability (geotechnical) report. This report must detail how the development may be achieved to ensure geotechnical stability and good engineering practice. The report must also include a risk assessment for existing/potential instability as per the AGS March 2007 publication.

The report includes a description of the site, borehole logs, in-situ test results, a borehole location plan, the results of site mapping, a geotechnical assessment of the development, a landslip risk assessment and provides recommendations for design, construction and stormwater control.

The investigation was undertaken as per the Proposal Number: P20-522, Dated: 10th November 2020 and comprised:

a) 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 two Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface conditions.

The following plans and drawings were supplied for the work:

- Architectural Drawings – by Leung Architects, Dated: October 2020, Job No.: 2008, Drawing No. GA.04 and GA-06, Issue A.
- Survey Drawing – by True North Surveyors, Date: 15/10/2020, Drawing No.: 2066, Revision: 0.

2. SITE FEATURES:

2.1. Description:

The site is a rectangular shaped block located on the low south side of Cutler Road with north/south boundaries of approximately 10.7m and east/west boundaries of approximately 44.2m approximately covering an area of 470m^2 in plan, as referenced from the provided detail survey. An aerial view of the site and surrounds obtained from Google Earth is provided in Photograph 1.

Photograph 1:Aerial view of the site (outlined red) and immediate surrounds

Site surface elevations vary between a high of approximately RL53.8m within the north west corner and a low RL53.5m of adjacent to the south east corner of the site. The north east corner of the site contains a cliff line which trends broadly north south and extends through the adjacent property to the west (No.51 Cutler Road).

The site dwelling comprises a two storey rendered structure accessed via a south sloping concrete driveway and paved courtyard at the front of the residence. The rear of the site contains a small garden area and a timber deck constructed on the south of the residence.

The rear garden is supported by a sandstone block retaining wall which reaches a maximum height of approximately 1.5m. A series of sandstone steps lead down from the west side of the retaining wall to a rockery within the south of the site.

Two general views of the site are provided in Photograph 2 and Photograph 3.

Photograph 2: View of the rear garden of the property looking broadly south east.

Photograph 3: View of the front courtyard and cliff line near the western shared boundary

The site is bordered to the north, east, south and west by Cutler Road easement, the unformed Weekes Road, No.30 Ogilvy Road ('No.30') and No.49 to No.51 Cutler Road ('No.49 to No.51') respectively.

Cutler Road comprises an asphalt pavement which dips towards the east with concrete kerbing and gutters where it passes the site. To the north of the site the easement comprises a raised garden bed and a concrete access ramp connecting to Cutler Road.

The unformed Weekes Road alignment contains a mixture of overgrown bush/vegetation with rock outcrops and dips moderately towards the south and east.

No.30 contains a three storey rendered residence approximately 7.50m from the shared boundary. The ground surface elevation within No.30 is approximately 3.50m -5.0m below site levels immediately adjacent to the shared boundary below the cliff.

No.49 to No.51 contains a three storey rendered residence with garage and in-ground pool constructed on an elevated position at the top of the cliff which extends from the site through No.49 to No.51. The inground pool is approximately 2.0m from the shared boundary. The surface elevation within No.49 to 51 is broadly similar to the site immediately adjacent to the shared boundary with the majority of the structures constructed on the cliff top approximately 9.0m above the elevation of the sites rear lawn.

.

2.2. Geology:

Reference to the Sydney 1 : 100,000 Geological Series sheet indicates that the site is underlain by Hawkesbury Sandstone which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. An extract of the Sydney Series sheet is provided as Extract 1.

Extract 1: Sydney Series Sheet with the approximate site location circled red

Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes. These slopes often consist of sandstone terraces and cliffs with steep colluvial slopes below. The terraced areas above these cliffs often contain thin sandy (low plasticity) soil profiles with intervening rock (ledge) outcrops. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint pattern, many cliff areas are undercut by differential weathering. Slopes below these cliffs are often steep 15° to 23° with moderately thick sandy colluvial soil profile that are randomly covered by sandstone boulders.

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 19th November 2020 by a Senior Engineering Geologist.

The walk over inspection comprised geological/geomorphological field mapping and observation of structures/conditions within and adjacent to the site to assess topography, slopes, structures as well as bedrock outcrops where applicable. The inspection was restricted to observations made from the ground surface of the site/adjacent, accessible land. Vegetation clearance or close inspection of elevated bedrock exposures was not undertaken as part of the inspection. Photographs of relevant observations were taken for inclusion in the report and to allow the creation of a photographic record to be made prior to commencement of construction works within the site.

The subsurface investigation comprised the drilling of two boreholes (BH1 and BH2) using hand augers due to access constraints and DCP testing at two locations (DCP1/1a and DCP2).

DCP testing was carried out from ground surface adjacent to the boreholes in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer” to estimate near surface ground conditions.

Soil samples were recovered from the augers for geotechnical logging and purposes which was undertaken in accordance with AS1726:2017 ‘Geotechnical Site Investigations’.

On completion the boreholes were backfilled with arisings and surface compacted.

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

3.2. Field Observations:

The front of the site contains a vertical cliff face up to approximately 4.0m above the level of the courtyard and comprises sandstone bedrock of low to medium strength. Within the site, signs of detachment or potential instability were not observed in the cliff exposure. It was noticeable that the upper surface of the cliff contained vertical defects up to approximately 0.3m in depth and areas of the upper surface of the sandstone outcrop appear to have been mortared to maintain stability.

The site dwelling external walls and paved areas appeared in good condition with no evidence of cracking which likely indicates structure footings extend to bedrock which is consistent with the outcrops observed.

The area immediately adjacent to the south and below the proposed pool within the rear of the property comprised a combination of what is considered to represent detached boulders as well as sections of intact bedrock. However, due to the size of the blocks a clear distinction was difficult to identify based on observations alone.

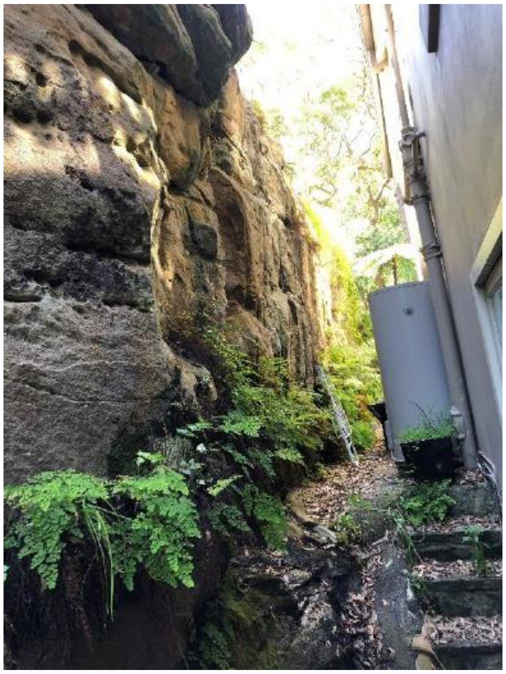
The area within the south east corner of the site and within the adjacent unformed Weekes Road alignment displayed indications of previous movement including detached sandstone blocks apparently undergoing downslope creep and, likely more recently, evidence of separation of mortar within stabilized blocks. The gradual slope within the area becomes steep to the south and it is considered that the movement likely represents a creep or downslope migration of colluvial soils rather than a stability hazard.

Photograph 4 and 5: View of the rear south of the site looking broadly north east at detached blocks apparently creeping downslope (4) and separation within mortared blocks

To the south east of the proposed works location a stormwater water pipe was observed discharging onto the slope however indications of erosion or rilling were not apparent immediately below the outlet (see Photograph 6).

Photograph 6 and 7: View of stormwater pipe discharging onto slope below site (6) and adequate stormwater transfer away from site residence

The property structures to the west of the site (No.49 - No.51) are constructed on the top of the cliff line (shown in Photograph 8) and appear in good condition will no unfavourable, continuous defects observed within the cliff to indicate a potential mechanism for instability. There was evidence of relatively large-scale prior instability (likely predevelopment within the area) and at least one large detached block was observed in No.49 to No.51 to the south of the main dwelling as shown in Photograph 9.



Photograph 8 and 9: View of cliff line near the west boundary of the site and detached boulder in No.49 to No.51

The neighbouring property to the south of the site, No.30, did not display any obvious signs of cracking or distress.

The neighbouring properties and structures were inspected from the site or road reserves, however visible aspects showed no indications of geotechnical hazard that may impact the site.

3.3. Ground Conditions:

The boreholes (BH1 and BH2) were drilled using a hand auger with refusal encountered at 0.6m and 0.7m depth within BH1 and BH2 respectively. DCP refusal was encountered between 1.2m and 0.9m depth within DCP1 and DCP2 respectively.

For a detailed description of the subsurface conditions encountered at the borehole locations, the Borehole Log sheets should be consulted, however, in general the investigation identified the following geological profile:

- **TOPSOIL –** this layer was encountered at both test locations to a maximum depth 0.45m (BH1) below the existing ground surface. It comprised fine grained silty sand with rootlets and a trace of sub-angular sandstone gravel.
- **FILL –** Underlying the topsoil, fill comprising clayey sand was encountered to a maximum of 0.9m depth which did not appear to display a high degree of compaction.

- **SANDSTONE –** Below the fill, bedrock was encountered which appeared to contain joints/defects within the upper surface based on the results of the DCP testing undertaken which extended to well below the depth at which rock was encountered within the boreholes.

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 a layer of predominately sandy fill to a maximum depth of 0.9m underlain by a layer of jointed/fractured sandstone underlain by better quality sandstone which appeared to be at least very low strength from around 0.9m-1.0m and likely significantly stronger based on adjacent bedrock exposures.

The site investigation did not identify signs of previous or impending shallow or deep-seated landslip instability within the site and there were no active landslip hazards identified within the neighbouring properties that would be expected to negatively impact the site.

It is understood that up to 1.0m of fill is proposed to be placed under the south of the pool and to assist in landscaping along with excavation of up to 1.5m depth.

Considering the scope of works and ground conditions encountered during the investigation it is considered that the following key elements will need to be addressed as part of the development.

- Short- and long-term batter stability
- Excavation support
- Excavation machinery and methodology
- Fill placement
- Presence of Sydney Water asset
- Allowable bearing pressures
- Adequate storm water disposal

The proposed works involve alterations and additions including an excavation for a new pool up to 1.5m depth which will be approximately >20m, 3.0m, 3.3m and 3.3m from the shared boundaries to the north, east, south and west respectively. Based on the separation distances from the shared boundaries it appears temporary batters in conjunction with a retaining wall construction/pool shell installation post excavation would be suitable to provide support.

Footings should be founded within bedrock of similar strength (≥very low strength) unless the potential for differential movement can be tolerated by the proposed structures.

The exact strength of the bedrock that will be encountered during the excavation is unconfirmed and is likely to vary with depth and across the proposed excavation. There is a potential for the lower/deeper portions of the excavation to intersect low strength and potentially medium strength bedrock which would require rock excavation equipment (i.e. rock hammer). The use of rock hammers will create ground vibrations which could impact adjacent structures. Since the excavation is to be carried out in a relatively small area, rock hammers should be limited to ≤300kg which will maintain low ground vibrations levels as per the AS2187.2-2006 requirements. This will result in slow excavation progress however should large hammers be proposed then a geotechnical professional should be consulted for equipment assessment and controls.

It is understood that fill is to be placed for landscaping and under the southern section of the proposed pool. Fill should not be used as structural fill (load bearing) for any parts of the development unless movement can be tolerated between sections found on bedrock and sections founded on fill. All fill should be placed in acceptance with AS3798-2007 *Guidelines on earthworks for commercial and residential developments*.

A Sydney Water (SW) asset is understood to underlie the site at the location indicated approximately on Figure 1. Available information indicates that asset may have been constructed sometime between 1924 and 1987 (based on the '*metropolitan water sewerage drainage board*' operational period). It is recommended that Sydney Water be contacted as soon as possible to confirm what asset protection measures may be required as measures required to protect SW assets can have significant impact on project design as well as construction timetables, particularly where older assets are potentially impacted.

All stormwater pipes (existing or future) should divert stormwater away from site and into the council drainage network.

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 rock, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent initial design of the proposed works.

4.2. Site Specific Risk Assessment:

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

A. Earth slide of fill soils due to excavation.
B. Rockslide/dislodgement of detached blocks/boulders

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

The Risk to Life from Hazard A was estimated to be up to **7.81×10^{-7},** whilst the Risk to Property was considered to be **'Very Low'**. The hazard was therefore considered to be 'Acceptable' when assessed against the criteria of the AGS 2007.

The Risk to Life from Hazard B was estimated to be up to **3.13×10^{-6},** whilst the Risk to Property was considered to be **'Very Low'**. The hazard was therefore considered to be 'Acceptable' when assessed against the criteria of the AGS 2007.

4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011	Class 'P ' due to thickness of fill. Where all fill or soil is removed from below proposed structures/footings and founds in sandstone bedrock a revised classification of 'A' would be appropriate.
Type of Footing	Strip/Pad or Slab at base of excavation
Maximum Allowable Bearing Capacity (kPa)	Sandstone: Very low strength: 700kPa Low strength: 1000kPa Medium strength: 2000kPa

Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – Rock site
Remarks: It is recommended that the base of footing excavations be inspected and tested where required by a geotechnical professional prior to concrete or steel placement to verify the subsurface conditions and the preliminary bearing capacities provided above. All new footings should be founded within material of similar strength to reduce the potential differential settlement.	

4.3.2. Excavation:

Property Separation Distances:

The table below shows the properties adjacent to the proposed excavation, anticipated excavation depths and boundary/structure separation distances.

Boundary	Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)*	
				Boundary	Structure
North	Cutler Road easement	Services*[1]	1.5	<20.0	-
East	Weekes Road	Vacant		3.0	-
South	No.30 Ogilvy Road	House		3.3	11.5
West	No.49 - No.51 Cutler Road	Pool		3.3	9.0

* Approximate only, subject to site set out. *[1]Unconfirmed

Type of Material to be Excavated	Sandy Topsoil (up to 0.20m depth)
	Sandy Fill (up to 0.70m depth)
	Sandstone, likely low to medium strength or stronger

Guidelines for <u>un-surcharged</u> batter slopes for are tabulated below:

Material	Safe Batter Slope (H:V)*	
	Short Term/ Temporary	Long Term/ Permanent
Fill	1:1.5	2.5:1
Fractured/jointed sandstone	Subject to defect orientation	
Defect free low to medium strength sandstone	Vertical*	

Remarks: Seepage through soils 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. Geotechnical inspection of batters will be required at

regular intervals. Based on the proposed design these batter slopes appear achievable for the majority of the pool excavation.		
Equipment for Excavation	Topsoil/Sandy Soils	Excavator with bucket
	Low to medium strength Sandstone	Excavator with bucket and ripper and rock hammer/saw.
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	5mm/s for houses/residential structures 3mm/s for services subject to confirmation with SW	
Vibration Calibration Tests Required	If hammers heavier than 300kg proposed for use	
Full time vibration Monitoring Required	Subject to calibration test results	
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • For assessment of proposed and constructed batter slopes • Where unexpected ground conditions are identified, or any other concerns are held. • Following footing excavations to confirm founding material strength	
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 soils. 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, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.		

4.3.3. Retaining Structures:	
Require	For pool and lower garden
Types	Steel reinforced concrete/concrete block walls post excavation where required designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures where a pool shell is not used or where delay

	between excavation and installation of pool shell is envisaged.
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)	
Clayey/Silty Sand Fill	18	$\phi' = 29°$	0.35	0.52	N/A
Fractured/jointed sandstone	20	$\phi' = 30°$	0.25	0.36	3.25

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	No
Site Location and Topography		Low south side of the road, within moderately south sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not feasible or recommended.

Remarks: Trenches, as well as all new and existing building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site.

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior Construction Certificate.
2. Inspect new footings to confirm compliance to design assumptions with respect to allowable bearing pressure prior to the placement of steel or concrete.

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

5. CONCLUSION:

The site investigation identified the presence of a layer of predominately sandy fill to a maximum depth of 0.9m underlain by sandstone bedrock contains defects within the upper surface.

The distance to all shared boundaries from the excavation perimeter is such that safe temporary batters appear achievable and it is considered that support prior to excavation is not required.

It is expected that excavation will extend through sand fill then potentially hard bedrock. Therefore, the vibration generation potential may need to be further assessed during initial excavations within the site.

The risks associated with the proposed development can be maintained within 'Acceptable' 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
	MAIG. RPGeo; 10197

6. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
4. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
5. Australian Standard AS 2870 – 1996, Residential Slabs and Footings – Construction
6. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
Unit 12/ 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Email: info@croziergeotech.com.au
Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

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: -

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.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

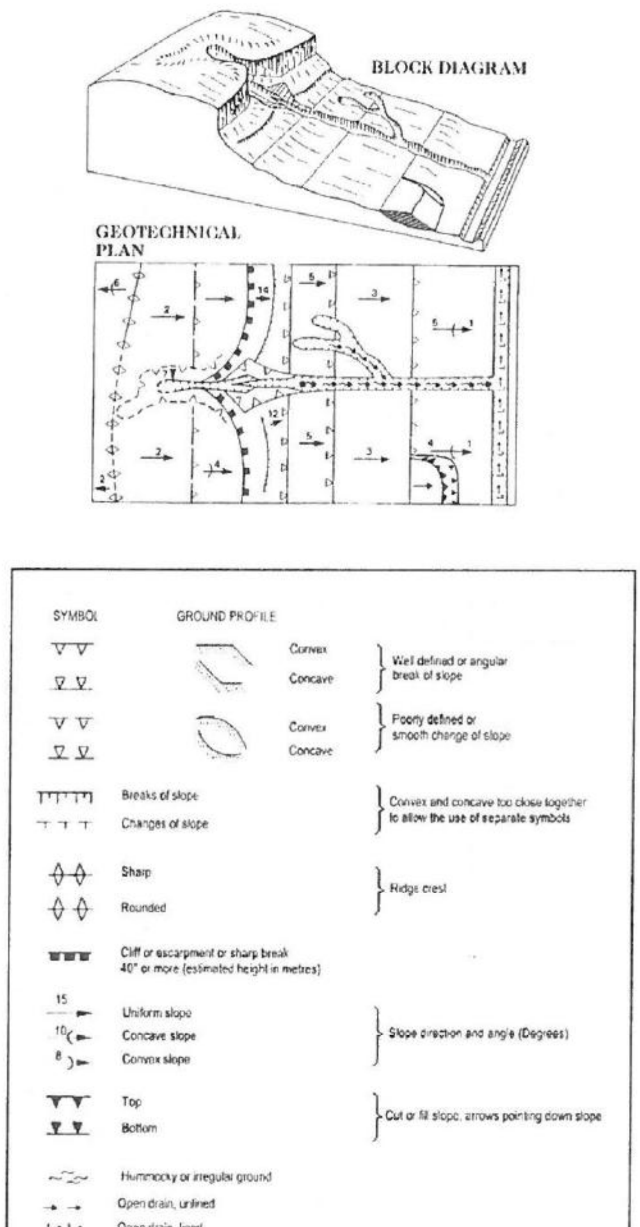
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

Reproduction of Information for Contractual Purposes

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

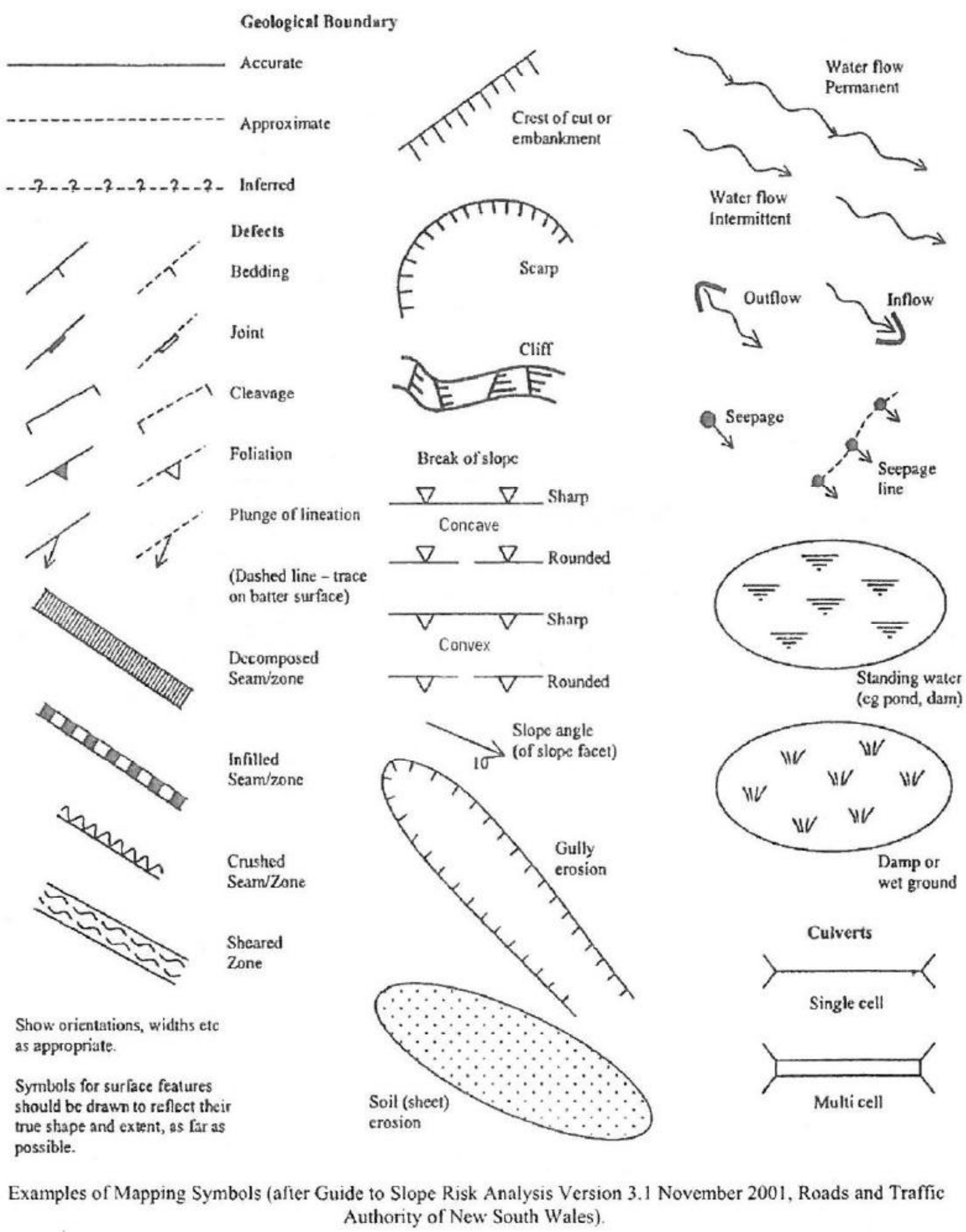
Site Inspection

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



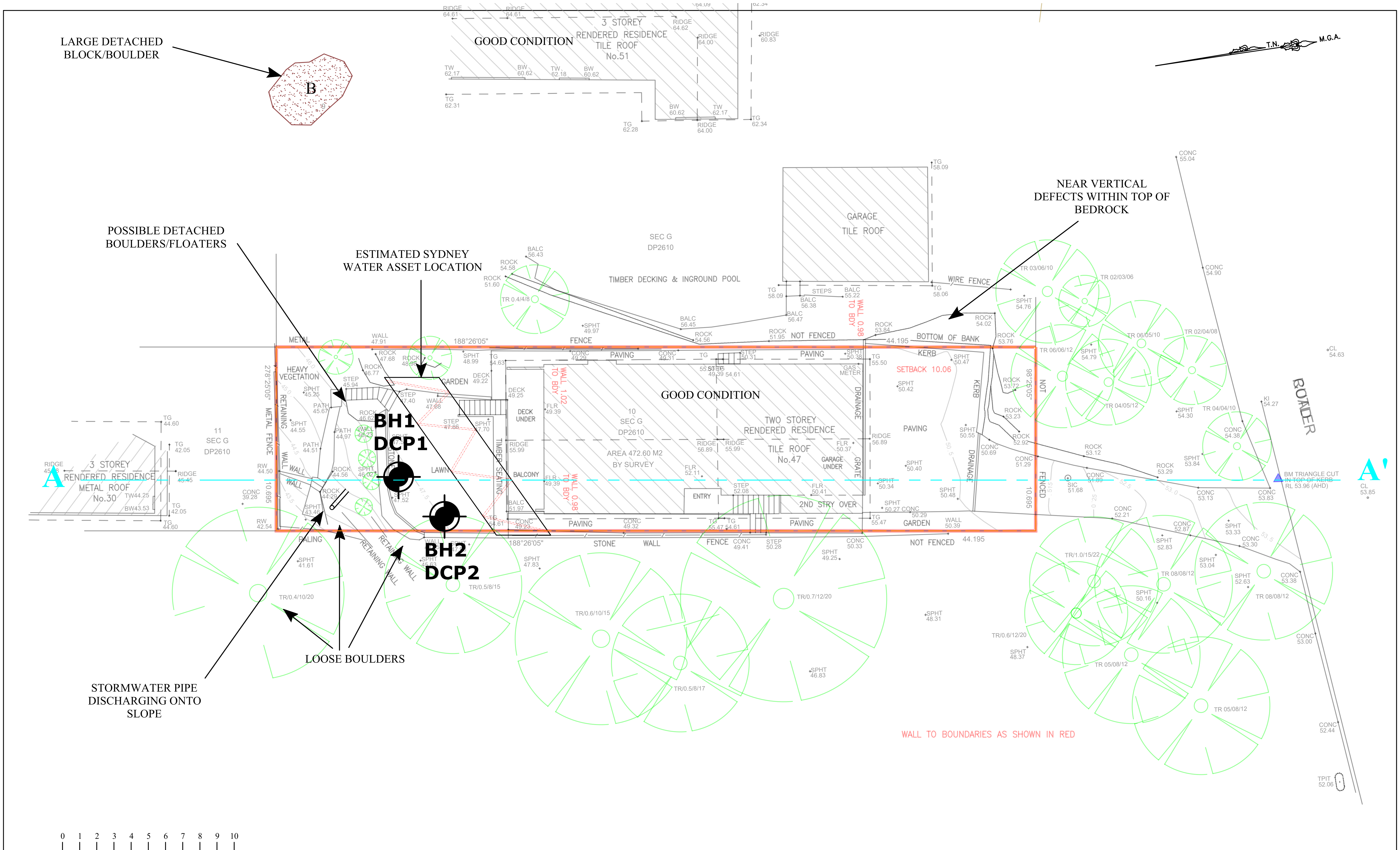
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN 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
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

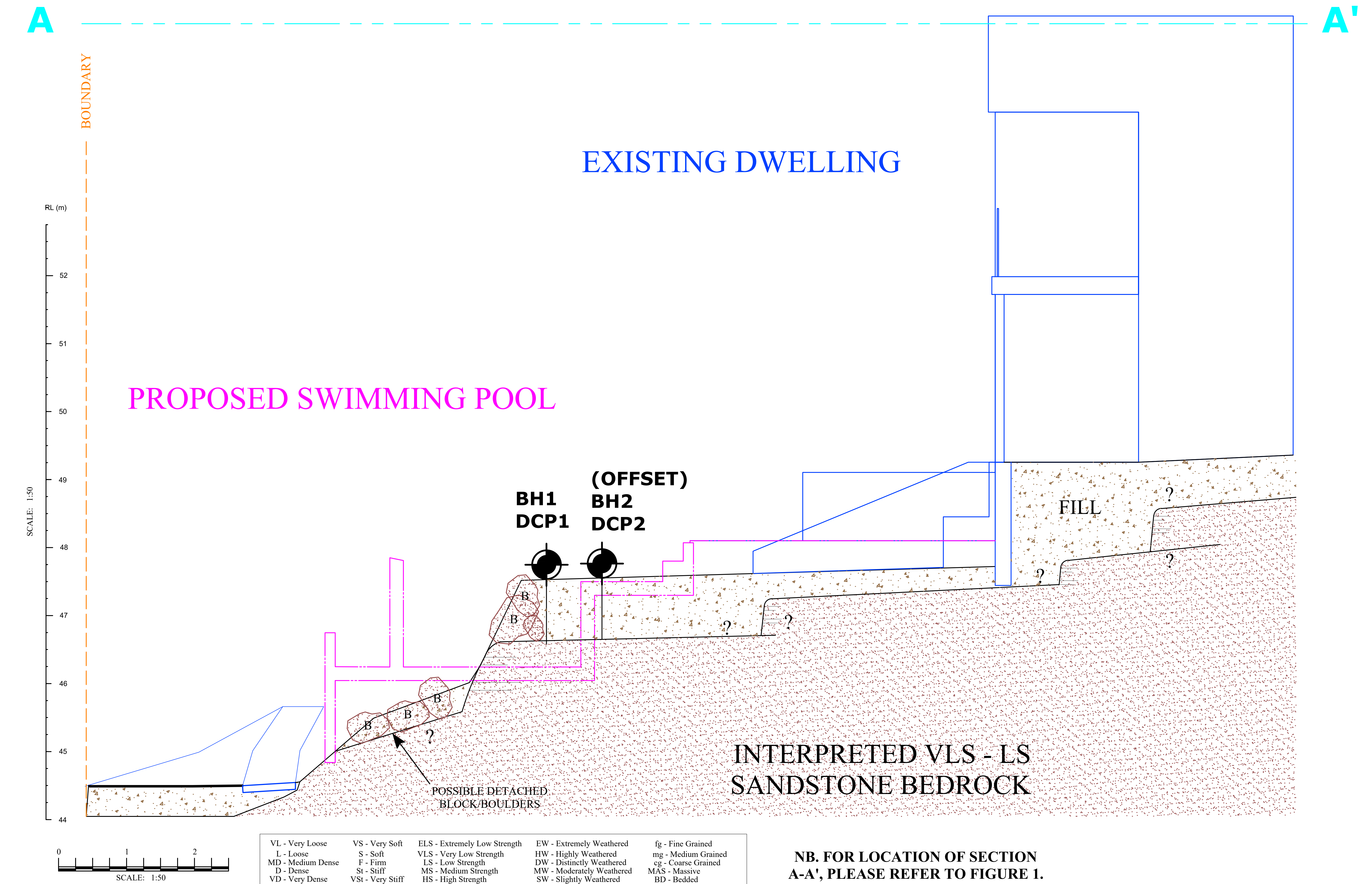
LEGEND

A —— A'	CROSS-SECTION REFERENCE LINE	BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
	PROPERTY BOUNDARY	B	SANDSTONE BOULDER
	APPROXIMATE ZONE OF SYDNEY WATER SEWER ASSET		

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	30/11/2020
APPROVED BY:	KN
DRAWN BY:	JC
PROJECT:	2020-235

PREPARED FOR:
Isabelle Durrenberger & Richard Polglase

ADDRESS:
47 Cutler Road, Clontarf



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

NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

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
Fax: (02) 9939 1883

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

LEGEND

A —— A'	CROSS-SECTION REFERENCE LINE	BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION		SOIL/FILL
	PROPERTY BOUNDARY		EXISTING STRUCTURES		PROPOSED STRUCTURES
	SANDSTONE BEDROCK				

SCALE:	1:50 @ A3
DRAWING:	FIGURE 2
DATE:	30/11/2020
APPROVED BY:	KN
DRAWN BY:	JC
PROJECT:	2020-235

PREPARED FOR:
Isabelle Durrenberger &
Richard Polglase

ADDRESS:
47 Cutler Road, Clontarf

BOREHOLE LOG

CLIENT: Isabelle Durrenberger and Richard Polglase
DATE: 19/11/2020
BOREHOLE: 1

PROJECT: New Swimming Pool
PROJECT No.: 2020-235
SHEET: 1 of 1

LOCATION: 47 Cutler Road, Clontarf
SURFACE LEVEL: RL47.5m

Depth (m)	Classification	Consistency or Density	Description of Strata **PRIMARY SOIL** - colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00			FILL: Brown silty sand, trace subangular to subrounded gravel of sandstone				
0.45			é pale grey brown and orange clayey sand fine to medium grained, trace gravel, locally grading to a sandy clay.				
0.58			SANDSTONE COBBLES/BEDROCK				
			Refusal at 0.60m depth-Interpreted cobble/fractured sandstone bedrock of at least low strength.				
1.00							
2.00							

RIG: Not applicable
DRILLER: JC

METHOD: Hand Auger
LOGGED: KN

GROUND WATER OBSERVATIONS: Not encountered during drilling

REMARKS:
CHECKED:

BOREHOLE LOG

CLIENT: Isabelle Durrenberger and Richard Polglase

DATE: 19/11/2020

BOREHOLE: 2

PROJECT: New Swimming Pool

PROJECT No.: 2020-235

SHEET: 1 of 1

LOCATION: 47 Cutler Road, Clontarf

SURFACE LEVEL: RL47.5m

Depth (m)	Classification	Consistency or Density	Description of Strata **PRIMARY SOIL** - colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00			FILL: Brown silty sand, trace subangular to subrounded gravel of sandstone				
0.30			é pale grey brown and orange clayey sand fine to medium grained, trace gravel, locally grading to a sandy clay.				
0.65			...concrete fragment encountered at 0.65m depth				
			End of borehole at 0.70m-no recovery/advancement of borehole achieved below 0.70m.				
1.00							
2.00							

RIG: Not applicable

DRILLER: JC

METHOD: Hand Auger

LOGGED: KN

GROUND WATER OBSERVATIONS: Not encountered during drilling

REMARKS:

CHECKED

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Isabelle Durrenberger and Richard Polglase
DATE: 19/11/2020
PROJECT: New Swimming Pool
PROJECT No.: 2020-235
LOCATION: 47 Cutler Road, Clontarf
SHEET: 1 of 1

	Test Location							
Depth (m)	DCP1	DCP1a	DCP2					
0.00 - 0.15	2	2	1					
0.15 - 0.30	2	3	1					
0.30 - 0.45	2	1	3					
0.45 - 0.60	2	4	2					
0.60 - 0.75	1	3	2					
0.75 - 0.90	2	3	3					
0.90 - 1.05	4	12	13					
1.05 - 1.20	5	B@1.02m	B@0.90m					
1.20 - 1.35	End							
1.35 - 1.50								
1.50 - 1.65								
1.65 - 1.80								
1.80 - 1.95								
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
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

CROZIER
GEOTECHNICAL CONSULTANTS

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 <2m^3) of fill soils from excavation works.		Possibly 1.0m of loose sand fill in excavation walls, likely unstable	a) Pathway within 2.0-3.0m of excavation may impact small area b) Instabilty of sands would impact perimeter of excavation only		a) Person on pathway 0.25hrs/day avge. b) Person in excavation 10hr/day avge.	a) Unlikely to not evacuate b) Likely to not evacuate	a) Person in open, unlikely engulfed b) Person in open, unlikely engulfed	
			Possible	**Prob. of Impact**	**Impacted**				
		b) Path in south end of site	0.001	0.30	0.10	0.0104	0.25	0.05	**3.91E-09**
		a) Base of excavation	0.001	1.00	0.05	0.4167	0.75	0.05	**7.81E-07**
B	Rock falls (<3m^3) during excavation works through unstable blocks.		Defects are anticpated in the bedrock surface and floaters/ boulders maybe present in/below retaining wall requiring removal and potentially in excavation walls	a) detached blocks may impact a small section of the path b) Detached blocks would impact perimeter of the excavation c) Detached blocks may roll down hill and impact small section of boundary retaing wall to the south.		a) Person on pathway 0.25hrs/day avge. b) Person in excavation 10hr/day avge. c) Person on/adjacent to boundary 0.25 hr/day	a) Unlikely to not evacuate b) Likely to not evacuate c) Unlikely to not evacuate	a) Person in open, unlikely impacted by boulder b) Person in open, unlikely impacted by boulder b) Person in open, unlikely impacted by boulder	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Path in south end of site	0.001	0.20	0.10	0.0104	0.25	0.20	**1.04E-08**
		b) Base of Excavation	0.001	1.00	0.05	0.4167	0.75	0.2	**3.13E-06**
		c) No.30 Ogilvy Road	0.001	0.10	0.05	0.005	0.25	0.2	**1.25E-09**

* hazards considered in current condition and/or without remedial/stabilisation measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (soil slide <2m3) of fill soils from excavation works.	b) Path in south end of site	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		a) Base of excavation	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Rock falls (<3m3) during excavation works through unstable blocks.	a) Path in south end of site	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		b) Base of Excavation	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		c) No.30 Ogilvy Road	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

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

CROZIER
GEOTECHNICAL CONSULTANTS

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}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	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.

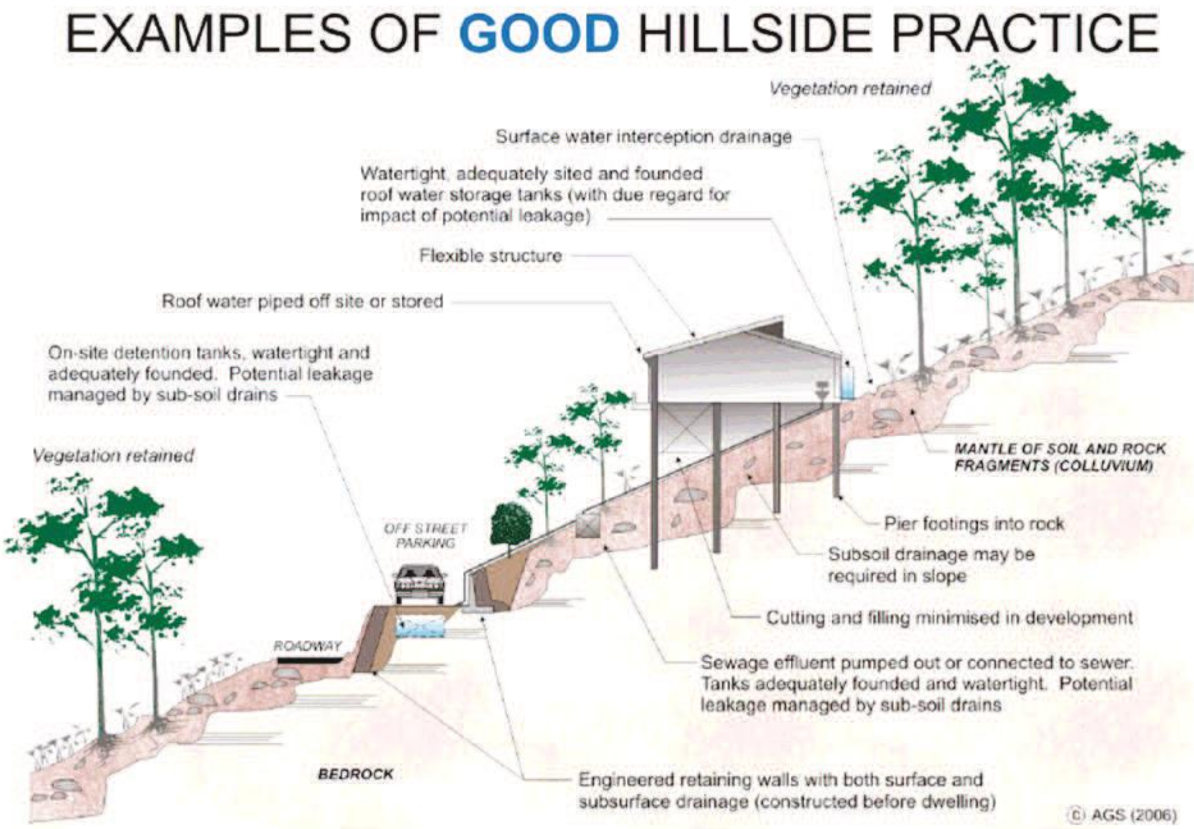
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

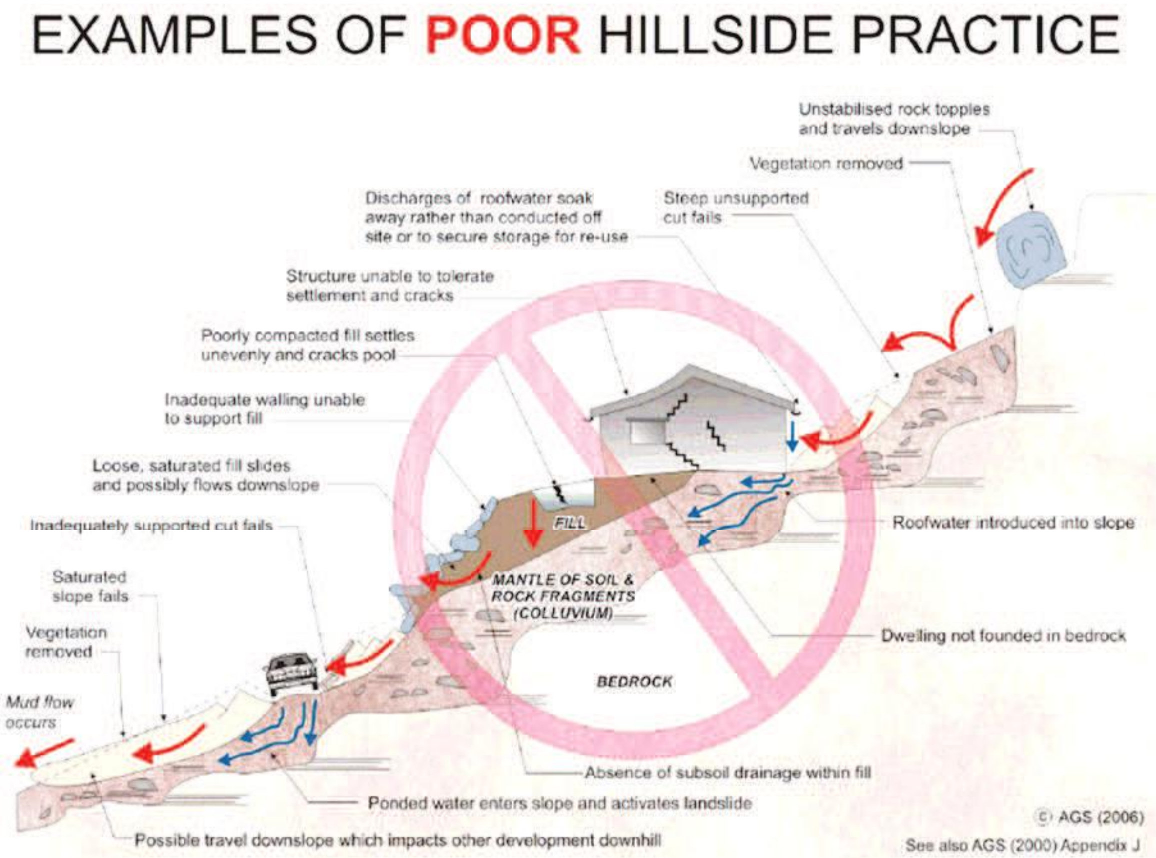
CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
© AGS (2006)
See also AGS (2000) Appendix J