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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

ALTERATIONS AND ADDITIONS

at

1126 BARRENJOEY ROAD, PALM BEACH, NSW

Prepared For

Adam Messenger

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GEOTECHNICAL INVESTIGATION FOR PROPOSED ALTERATIONS AND ADDITIONS AT 1126 BARRENJOEY ROAD, PALM BEACH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 1126 Barrenjoey Road, Palm Beach, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of Laura Cook on behalf of the owner, Adam and Lee-Anne Messenger.

It is understood that the proposed works involve the demolition of the existing veranda structure at the front of the dwelling and the construction of a new outdoor kitchen, spa, and pergola. It is understood that the development will require a minor excavation to accommodate the construction of new footings and no bulk excavation or mass filling is proposed.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009). To meet the Councils Policy requirements for land classified as H1 a detailed Geotechnical Report is required which meets the requirements of Paragraph 6.5 of that policy. However, it is understood that the works are being completed under a CDC, therefore the investigation and report were required for design and construction purposes only.

The field investigation was limited by the existing structures however it comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineering professional,
b) DBYD plan request and onsite service location and clearing of borehole locations by an accredited contractor.
c) The drilling of two boreholes using hand tools along with Dynamic Cone Penetrometer (DCP) tests adjacent to the boreholes and at three additional locations to investigate the subsurface conditions.
d) All fieldwork was conducted under the supervision of an experienced Geotechnical Professional.

The following plans and diagrams were supplied and relied upon for fee proposal preparation, assessment and reporting:

- Survey Drawing – Survey Plus, Reference No.: 24004_DET_1A, Dated: 6/01/2025
- 3D CAD Rendering – Rendering of proposed works, "Messenger_SKETCHUP_250310"
- Architectural Drawings – Laura Cook, Drawing No.: DA.01-.04(P1), Dated: 11/04/2025

2. PROPOSED DEVELOPMENT:

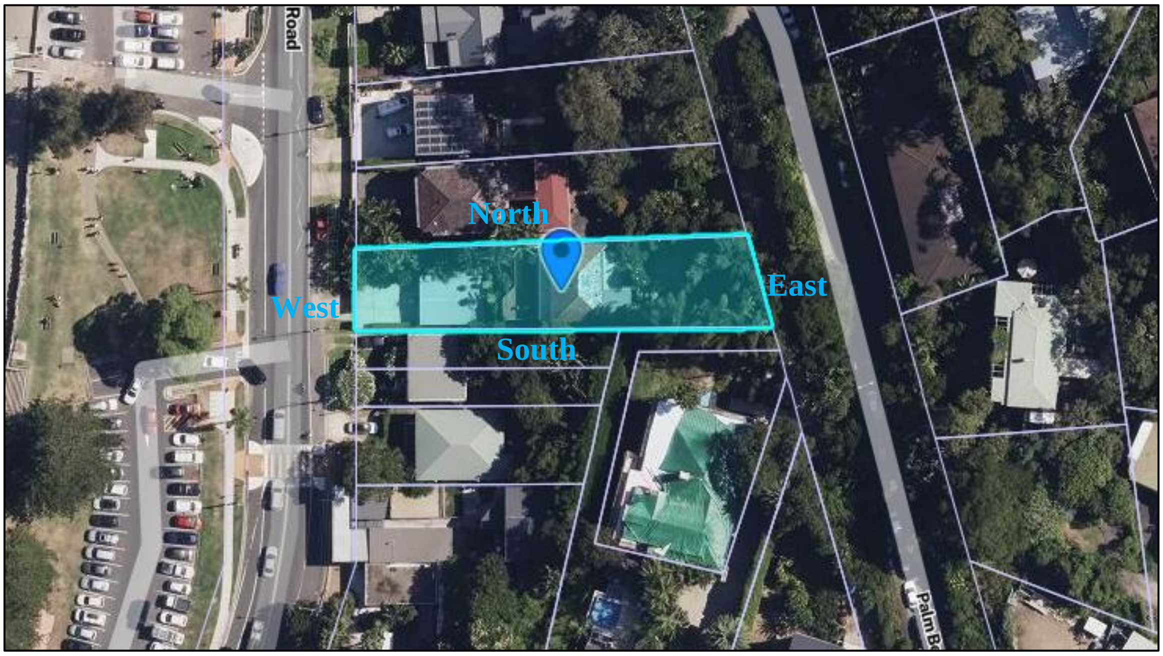
It is understood that the proposed works involve the demolition of the existing veranda structure at the front west side of the dwelling to allow for the construction of a new outdoor kitchen, spa and pergola. The new outdoor kitchen is proposed to be founded using the existing concrete piers. However, if the existing piers are shown to be founded within loose soils the structure will likely require additional supports. The additional supporting works are expected to require minor excavation to accommodate the new structures footings.

3. SITE FEATURES:

3.1. Site Description:

The site (No. 1126 Barrenjoey Road) is a rectangular shaped block orientated east to west which covers an area of approximately 878.9m^2 as per the provided survey. The site is situated within steep west sloping topography at the base of a steep north to south striking ridgeline with gentle sloping foreshore area to the west and is accessed by a concrete driveway. The front portion of the property is retained by a series sandstone and masonry structures that appear to be backfilled. The rest of the site's structures are comprised of a two storey timber and masonry dwelling with moderate to steep sloping gardens to the rear.

An aerial photograph of the site and surrounds with boundary designations is provided below (Photograph 1), as sourced from Six Maps Spatial Data.



Photograph-1: Aerial photo of site (outlined red) and surrounds

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is located near the boundary between the Hawkesbury Sandstone (Rh) and underlying Upper Narrabeen Group Shales (Rnn). Site inspection confirmed that the site is underlain by Newport Formation (Upper Narrabeen Group) rock 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.

Narrabeen Group rocks are dominated by shales and thin siltstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise shale beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints. The shale often forms deeply weathered silty clay soil profiles (medium to high plasticity) with thin silty colluvial cover.

An extract of the relevant geological sheet is provided as Extract 1.

Extract-1: Sydney (9130 Geology Series Map): 1: 100000 – Geology underlying the site

4. FIELD WORK:

4.1. Methods:

The field investigation comprised geotechnical inspection/mapping and a subsurface investigation which were both undertaken/supervised by a Geotechnical Engineer on 26th March 2025. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of bedrock outcrops, existing structures and limited inspection of neighbouring properties.

The investigation comprised the drilling of two hand auger boreholes (BH1,BH2) to investigate sub-surface geology. Dynamic Cone Penetrometer (DCP) testing was carried out adjacent to the boreholes and in three additional locations (DCP3 to DCP5) in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer Test" to estimate near surface soil conditions and assist in determining depth to bedrock.

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: 1.

4.2 Field Observations

The site is situated on the eastern side of Barrenjoey Road within steep west dipping topography. Barrenjoey Road comprises a bituminous sealed pavement and is separated from the site by a concrete pavement, gutter and kerb, and nature strip. No signs of excessive settlement or cracking were observed within any of the structures within the road reserve. Photograph-2 shows a view of the site from the road reserve.

Photograph-2: View of the front of the site from Barrenjoey Road.

Access is gained by a concrete driveway at the west of the site to a carport and a garage of masonry construction at the front southwest corner of the property. The garden situated between the main dwelling and carport/garage slopes steeply (~26 ^{o}) and comprises of a tiered retaining structure of masonry and sandstone block construction, which appeared to be backfilled to raise the ground surface levels. A concrete slab supporting a rain-water tank is located under the southern portion of the veranda, atop a brick retaining wall.

During the investigation two portions of this wall were observed to be rotated beyond vertical (Photograph-3). The portion of the wall supporting the water-tank and slab (Photograph-3 [left]) appears to be under rotating due excessive load from the tank structure and inadequate construction. The portion of the wall directly south (Photograph-3 [right]) was either formed with a defined slope or experiencing foundational failure from circular slip. A concrete staircase leads up the north side of the main dwelling and provides access to the existing veranda.

Photograph-3: View of brick retaining wall failing.

The main dwelling is positioned within the centre of the block and consists of a two-storey building of masonry and weatherboard construction. From available aerial historical imagery, the dwelling appears to be of <60 years original construction age, while the veranda to the west of the dwellings appears to be of <20 years construction age.

The site is bounded to the south by the property at No.1124 Barrenjoey Road which contains a two-storey residence set back 0.70m from the common boundary with the site which is primarily constructed of weatherboard and masonry. The levels within the property appeared to be <0.50m lower than to those within the site across the common boundary and are retained by a brick wall. Structures appeared from the limited visibility into the property to be in good condition.

The site is bounded to the north by the property at No.1128 Barrenjoey Road which contains a three-storey residence and semidetached one-storey dwelling connected by a walkway set back 1.70m from the common boundary with the site. The dwelling is primarily of masonry construction. The levels within the property appeared to be similar than to those within the site across the common boundary and the two properties aren't separated by any sort of fence structure. Structures appeared from the limited visibility into the property to be in good condition.

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 Field Testing

The boreholes (BH1, BH2) were drilled within the terraced garden area to the west of the property encountering a variable layer of topsoil/fill from existing ground surface to a depth of 0.80m in BH1 and 0.60 in BH2. BH1 subsequently intersected silty/sandy clay which extended to refusal atop interpreted sandstone bedrock of at least low strength, encountered at 2.00m depth. The fill in BH2 was similarly underlain by silty clays which extended until termination at 1.10m depth.

Dynamic penetrometer (DCP) tests were carried out adjacent to the boreholes and at three other locations, all encountering refusal atop interpreted sandstone bedrock of at least Very Low Strength (VLS) at depths varying from 2.30m to 4.60m (DCP3, DCP4 respectively).

For a detailed description of the ground conditions encountered at each investigation location, the Borehole Log sheets should be consulted. However, based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **FILL/TOPSOIL** – this layer was encountered at all test locations and extended to a maximum of depth of 0.80m. It was classified as loose to medium dense, dark brown, medium to coarse grained, moist, silty sand trace clay with some sub-angular sandstone gravels, building materials and rootlets.
- **Sandy/Silty CLAY**– This material was intersected within all boreholes underlying the fill and extended to a maximum depth of 2.00m (BH1). BH2 was terminated within this layer at a target depth of 1.10m. It typically comprised of firm to stiff, grey mottled yellow/orange, moist, clayey sand with ironstone gravels.
- **SANDSTONE/SILTSTONE BEDROCK** - This material was interpreted from refusal within BH1 and all DCP test locations. DCP1, DCP2 and DCP3 were located at the base of the retaining wall underneath the veranda and extended to refusal on interpreted sandstone/siltstone bedrock at varying depths 2.10m to 2.30m (≈RL4.40m). DCP4 and DCP5 were located behind/up-slope the retaining wall and extended to refusal atop interpreted sandstone/siltstone bedrock at varying depths of 2.70m and 3.80m (DCP5 and DCP4) from ground level (≈RL 5.20m). Note, DCP4 intercepted a 0.80m void between veranda and ground level. The bedrock is anticipated to comprise weathered at least very low strength sandstone/siltstone of the Narrabeen Group with the potential for shale bedrock to be present.

A free-standing ground water table was not intersected within the borehole above the bedrock surface, however minor seepage was observed on the face of the existing retaining wall.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified the presence of a variable layer of fill, generally overlaying silty clays, which extended to a maximum depth of 2.00m in BH1, before intersecting interpreted sandstone bedrock of at least very low strength. Additionally, DCP testing throughout the site confirmed the variability of the fill behind the existing retaining wall as well as identified the existence of rock at varying depths 2.30m, 3.50m, and 2.70m from ground level (DCP3, DCP4, DCP5).

During the investigation field observations identified that the existing veranda appeared to be founded using bored pier footings, however the depth at which these footings extended could not be determined. A free-standing ground water table was not intersected within the borehole above the bedrock surface, however minor seepage was observed on the face of the existing retaining wall.

The proposed works involve the demolition of the existing veranda structure at the front west side of the dwelling to allow for the construction of a new outdoor kitchen, spa and pergola. The new spa appears proposed to supported on the concrete slab in the vicinity of DCP4, while the new outdoor kitchen is proposed to be founded using the existing concrete piers for the house.

The founding conditions of the existing pile footings could not be confirmed therefore it is recommended that the builder undertake further investigation upon the demolition of the veranda. It is expected that the existing piles will be extended at least to found within very stiff residual clays, located approximately 1.70m below the current veranda level. Recommended allowable bearing pressures associated with various founding materials are outlined in Section 5.3.

In the event that the founding material of the existing footings do not meet required bearing pressure requirements, minor excavation for new footings will be required. It is envisioned that these footings will comprise of piers down to bedrock or to material of adequate bearing pressure.

The existing brick retaining walls below the veranda and supporting the existing concrete slab and tank appear to be failing. As such they will need additional support or replacement, especially if structures are proposed to be supported upslope. Any excavation will require care to ensure it does not impact the existing dwelling or its footings therefore further assessment is recommended following demolition of the veranda and initial site mark out.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and isolated boreholes and DCP testing. 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. However, the results of the investigation provide a reasonable basis for subsequent preliminary design of the proposed works.

5.2. Design and Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.2.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'P'
Type of Footing	Slab at the base of the spa, piers to bedrock/adequate material.
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	Stiff residual clay: 100kPa Very stiff residual clay: 200kPa Hard residual clay: 400kPa VLS Sandstone bedrock: 900kPa
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: These values are subject to confirmation by geotechnical testing/inspection during construction. All footings should be founded within soils of similar strength to prevent differential settlement. It may prove sensible to extend new additional footings to bedrock as they will be subject to negligible 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.	

5.2.2. Excavation:	
Depth of Excavation	Minor <1.00m anticipated, deeper potentially required for remedial works
Type of Material to be Excavated	Topsoil/clayey fill with cobbles and rubble to <1.00m
	Residual soils generally at <1.00m 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 natural soils	1.5:1	2.0:1

*Dependent on defects and assessment by engineering geologist.

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.

Should further detail on rock strengths or conditions for excavation costing be required, then cored boreholes and laboratory testing will be required.

Equipment for Excavation	Fill/natural soils	Bucket
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Prior to any excavation to assess zone of influence • Inspection of exposed existing footings • Following completion of excavation or during support installation	

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.

5.2.3. Retaining Structures:

Required	New retaining structures will be required to remediate failing existing structures
Types	Concreate block and steel reinforced retaining wall.

Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill	18	$\phi' = 28°$	0.35	0.52	N/A
Residual Clay Soils (Stiff)	20	$\phi' = 30°$	0.33	0.47	3.25
VLS -LS bedrock	22	$\phi' = 38°$	0.10	0.20	200kPa

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 backfilled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

5.2.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Yes – Minor seepage along retaining wall face
Excavation likely to intersect	Water Table	No
	Seepage	Negligible
Site Location and Topography		High east side of the road with steep west dipping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not recommended or required
Remarks: Trenches, as well as all new building gutters, downpipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which preferably discharges to the Council’s stormwater system off site.		

5.3. Conditions Relating to Design and Construction Monitoring:

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

1. Review the structural drawings, for inclusion of slope stability, underpinning and/or excavation support measures and compliance with the recommendations of this report,
2. Conduct inspections as per the recommendations of Section 5.3 in this report including inspection of 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,
3. Inspect the complete development to ensure all retention and stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

6. CONCLUSION:

The site investigation identified the presence of a variable layer of fill, generally overlaying silty/sandy clays, which extended to a maximum depth of 2.00m in BH1, before intersecting sandstone bedrock of at least very low strength. Additionally, DCP testing throughout the site confirmed the variability of the fill behind the existing retaining wall as well as identified the existence of rock at varying depths from ground level.

DCP1, DCP2 and DCP3 were located at the base of the retaining wall underneath the veranda and extended to refusal on interpreted sandstone bedrock at varying depths 2.10m to 2.30m (≈RL4.40m). DCP4 and DCP5 were located behind/up-slope the retaining wall and extended to refusal atop interpreted sandstone bedrock at varying depths of 2.70m and 3.80m (DCP5 and DCP4) from ground level (≈RL 5.20m). Note, DCP4 intercepted a 0.80m void between veranda and ground level.

The investigation field observations identified that the existing veranda appeared to be founded using bored pier footings, however the depth at which these footings extended could not be determined.

A free-standing ground water table was not intersected within the borehole above the bedrock surface, however minor seepage was observed on the face of the existing retaining wall.

The proposed works involve the demolition of the existing veranda structure at the front west side of the dwelling to allow for the construction of a new outdoor kitchen, spa and pergola.

The new spa is anticipated to be founded on a concrete slab, however the existing slab supporting the water tank appears itself supported by a failing brick retaining wall. Therefore, further investigation into this structure is required following deck removal and it is expected that a new support/retaining wall will be required with a new foundation system constructed to support the spa.

It is also recommended that upon demolition on the existing veranda structure that the existing dwelling footings are inspected by the builder to confirm the founding conditions. Geotechnical inspection will be required to assess the foundation conditions and bearing capacity if they are to be utilised to support any new structure. The recommended allowable bearing pressure will be determined by the depth of the footing and associated founding material as outlined in Section 5.3.

Care will be required where demolition / removal of the existing retaining walls is proposed as any excavation can induce instability in the slope. Therefore, geotechnical inspection should occur prior to any proposed excavation to allow assessment of the zone of influence.

Any excavation is not expected to intersect the groundwater table, and the development is therefore not expected to impact local hydrogeology or the groundwater levels in neighbouring properties. However, seepage is likely within the fill material and the soil to bedrock interface, particularly following periods of heavy rainfall. This seepage can impact excavation stability.

Prepared by:	Reviewed by:
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7. REFERENCES:

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Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination.
The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT “N” Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7 as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

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

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

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

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

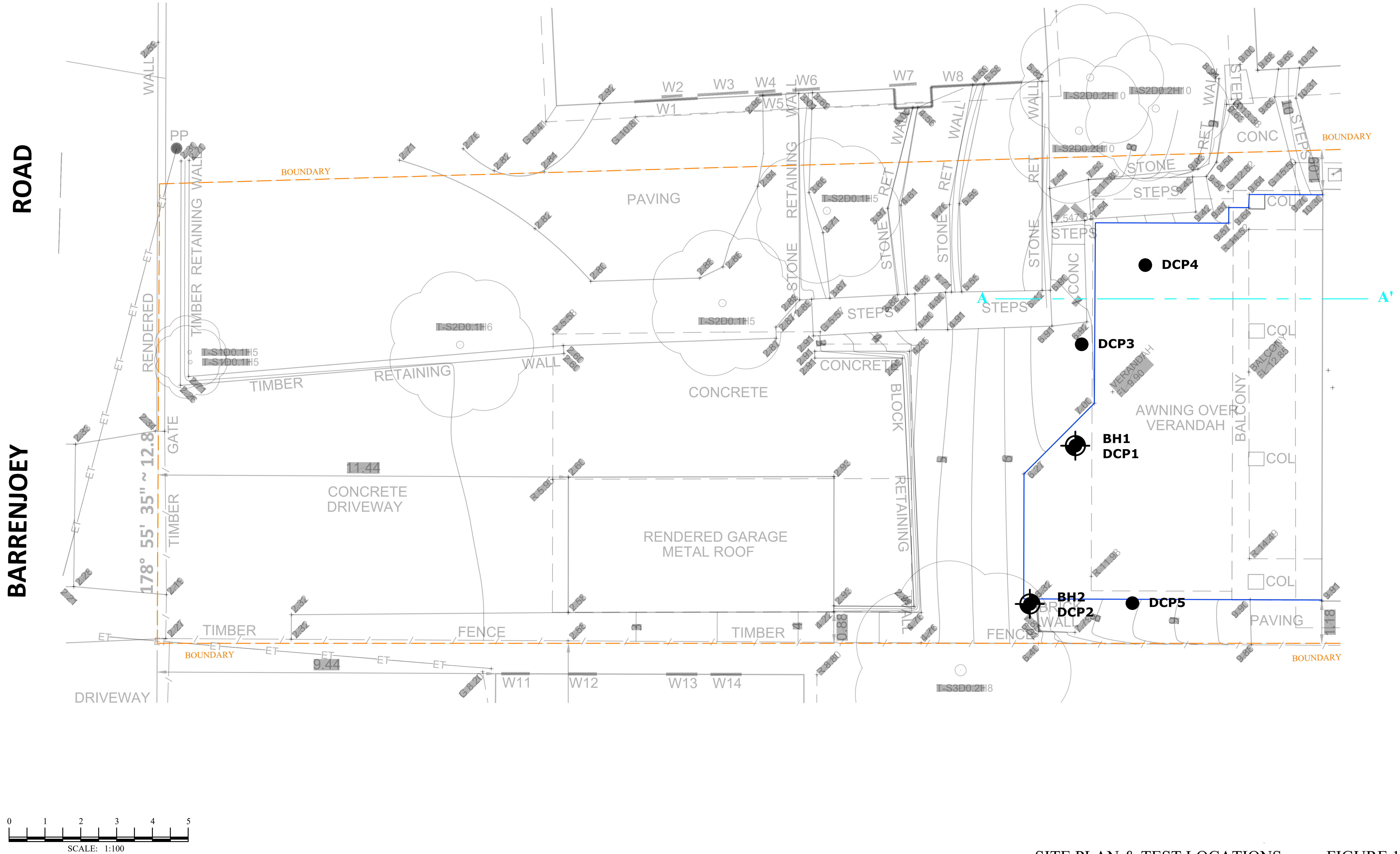
Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including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

Site Inspection

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

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

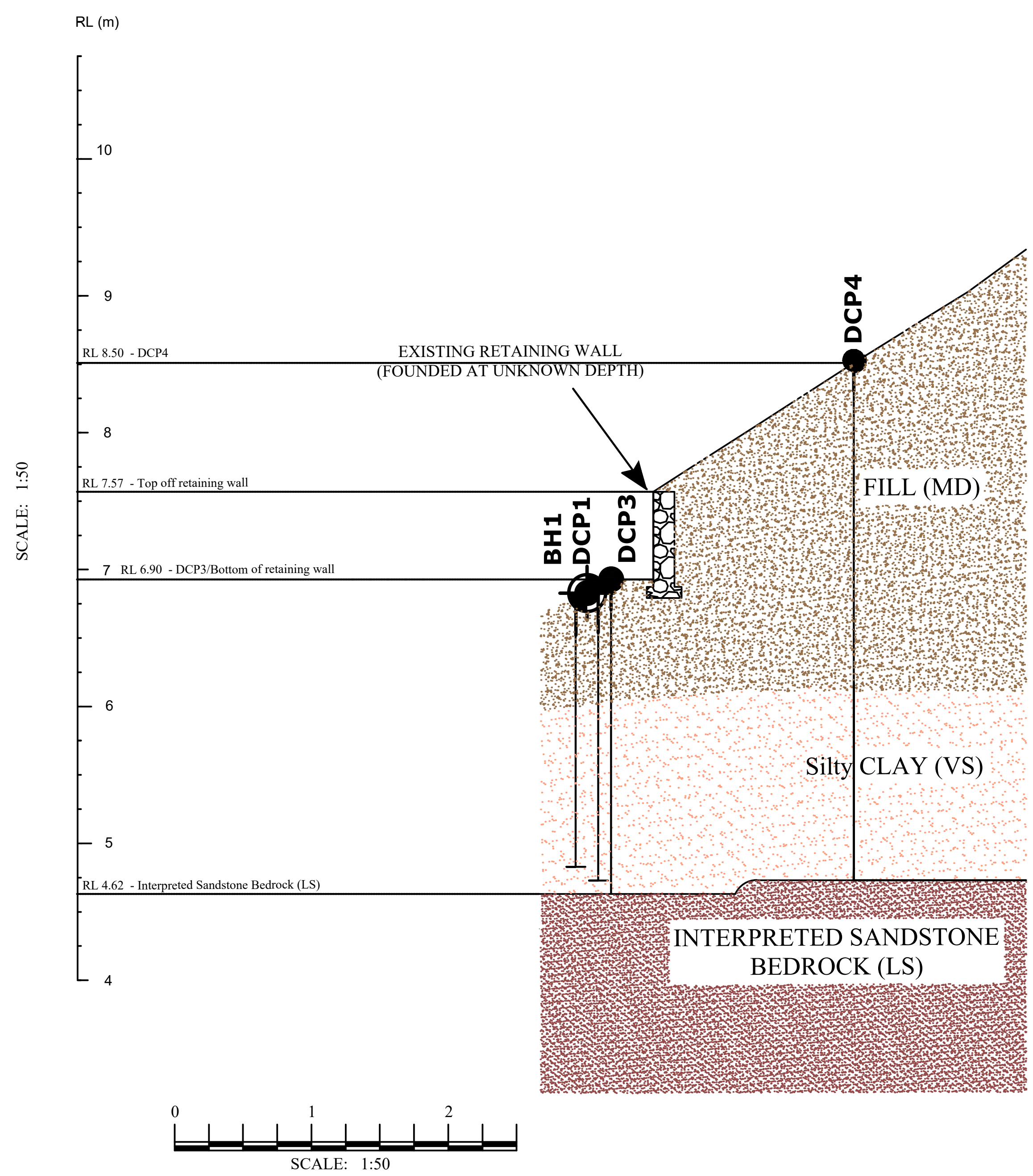
	PROPERTY BOUNDARY
	EXISTING STRUCTURES
DCP	DYNAMIC CONE PENETROMETER
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION

SCALE:	1:100 @ A3
DRAWING:	FIGURE 1
DATE:	03 /2025
APPROVED BY:	TMC
DRAWN BY:	TS
PROJECT:	2025-044

PREPARED FOR:
ADAM MESSENGER

ADDRESS:
1126 BARRENJOEY ROAD, PALM BEACH, NSW

A ——— A'



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

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP	DYNAMIC CONE PENETROMETER
	SOIL/FILL
	SILTY CLAY
	SANDSTONE BEDROCK
A —— A'	SECTION LINE

SCALE:	1:50 @ A3
DRAWING:	FIGURE 2
DATE:	05/2025
APPROVED BY:	TMC
DRAWN BY:	TS
PROJECT:	2025-044

PREPARED FOR:
ADAM MESSENGER

ADDRESS:
1126 BARRENJOEY ROAD, PALM BEACH, NSW

NON-CORE DRILL HOLE LOG

HOLE NO: BH1

PROJECT : Proposed Alterations and Additions
LOCATION : 1126 Barrenjoey Rd, Palm Beach NSW 2108
JOB NO: 2025-044
Sheet 1 of 1 FINAL DEPTH: 2.00 m

POSITION : N: 6281381.5 E: 344133.2 (GDA94 / MGA zone 56) SURFACE ELEVATION : 7.00 m (AHD) ANGLE FROM HORIZONTAL : 90°

RIG TYPE : CONTRACTOR : DRILLER : AC

DATE STARTED : 26/03/25 DATE COMPLETED : 26/03/25 DATE LOGGED : 26/03/25 LOGGED BY : TS CHECKED BY : TMC

CASING DIAMETER : BARREL (Length) : BIT : BIT CONDITION :

METHOD	WATER	SAMPLES & FIELD TESTS	RL (m AHD)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / RELATIVE DENSITY
HA			7.0	0.0			FILL - Silty SAND: fine to medium grained, dark brown and cream; with clay; fill.	D to M	MD
				0.40			FILL - Sandy CLAY: low to medium plasticity, orange mottled grey / yellow; fill.		
			6.0	0.80		CL-CI	CL-CI Silty CLAY: low to medium plasticity, pale grey mottled yellow / brown; with sand, banding; residual soil.	M	St
				1.50		CI	CI Silty CLAY: medium plasticity, orange mottled grey; with gravel, ironstone; residual soil.		VSt to H
				1.70		CL	CL Silty CLAY: low plasticity, dark red; with gravel, ironstone; residual soil.		
				1.80		CL-CI	CL-CI Silty CLAY: low to medium plasticity, grey; residual soil.		
				1.90		CL-CI	CL-CI CLAY: low to medium plasticity, dark red; residual soil.		
			5.0	2.00			Terminated at 2.00 m. Refusal.		

CROZIER GEOTECHNICAL CONSULTANTS

NON-CORE DRILL HOLE LOG

HOLE NO: BH2

PROJECT : Proposed Alterations and Additions
LOCATION : 1126 Barrenjoey Rd, Palm Beach NSW 2108
JOB NO: 2025-044
Sheet 1 of 1 FINAL DEPTH: 1.10 m

POSITION : (GDA94 / MGA zone 56) SURFACE ELEVATION : 6.80 m (AHD) ANGLE FROM HORIZONTAL : 90°
RIG TYPE : CONTRACTOR : DRILLER : TS
DATE STARTED : 26/03/25 DATE COMPLETED : 26/03/25 DATE LOGGED : 26/03/25 LOGGED BY : TS CHECKED BY : TMC
CASING DIAMETER : BARREL (Length) : BIT : BIT CONDITION :

METHOD	WATER	SAMPLES & FIELD TESTS	RL (m AHD)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / RELATIVE DENSITY
HA				0.0			FILL - Silty SAND: fine to medium grained, dark brown; trace clay; building materials ; fill.	M	L to MD
				0.60		CL-CI	CL-CI Silty CLAY: low to medium plasticity, pale grey and pale brown; residual soil.		F
			6.0	1.0					
				1.10			Terminated at 1.10 m. Target depth.		
			5.0	2.0					

CROZIER GEOTECHNICAL CONSULTANTS

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Adam & Lee-Anne Messenger
DATE: 26/03/2025
PROJECT: Proposed Alterations and Additions
PROJECT No.: 2025-044
LOCATION: 1126 Barrenjoey Rd, Palm Beach NSW 21(
SHEET: 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP4 (cont)	DCP5				
0.00 - 0.10	8	1	2	--	11	sw				
0.10 - 0.20	4	1	6	--	15	2				
0.20 - 0.30	2	4	4	--	15	1				
0.30 - 0.40	2	3	4	--	19	2				
0.40 - 0.50	5	3	3	--	19	1				
0.50 - 0.60	15	2	3	--	17	3				
0.60 - 0.70	16	1	3	--	**(PR)** at 4.60m depth	1				
0.70 - 0.80	7	1	2	--		1				
0.80 - 0.90	5	1	2	sw		3				
0.90 - 1.00	5	0	2	1		3				
1.00 - 1.10	5	1	2	2		3				
1.10 - 1.20	4	2	2	2		3				
1.20 - 1.30	4	2	2	2		2				
1.30 - 1.40	5	2	2	2		1				
1.40 - 1.50	4	1	2	1		2				
1.50 - 1.60	7	3	4	1		1				
1.60 - 1.70	6	2	3	10		0				
1.70 - 1.80	7	3	4	15		1				
1.80 - 1.90	12	27	3	8		2				
1.90 - 2.00	7	27	3	6		3				
2.00 - 2.10	9	14	7	6		4				
2.10 - 2.20	**(B)** at 2.10m depth	**(B)** at 2.10m depth	18	6		3				
2.20 - 2.30			11	5		4				
2.30 - 2.40			**(B)** at 2.30m depth	4		6				
2.40 - 2.50				5		5				
2.50 - 2.60				6		22				
2.60 - 2.70				6		10				
2.70 - 2.80				7		**(B)** at 2.70m depth				
2.80 - 2.90				8						
2.90 - 3.00				7						
3.00 - 3.10				8						
3.10 - 3.20				8						
3.20 - 3.30				8						
3.30 - 3.40				6						
3.40 - 3.50				6						
3.50 - 3.60				7						
3.60 - 3.70				6						
3.70 - 3.80				8						
3.80 - 3.90				9						
3.90 - 4.00				10						

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

REMARKS:

-- No test undertaken at this level due to prior excavation of soils
(B) Test hammer bouncing upon refusal on a solid object
(PR) Pratical Refusal: continuous 3 intervals with ≥15 Blows/100mm or a single interval with ≥25 Blows
(SW) Rod settlled due to the self-weight of equipment

Appendix 3

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}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-3}$	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-4}$	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-5}$	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}	$5x10^{-6}$	1,000,000 years	200,000 years	The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

Notes:
(2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.
(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.
(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

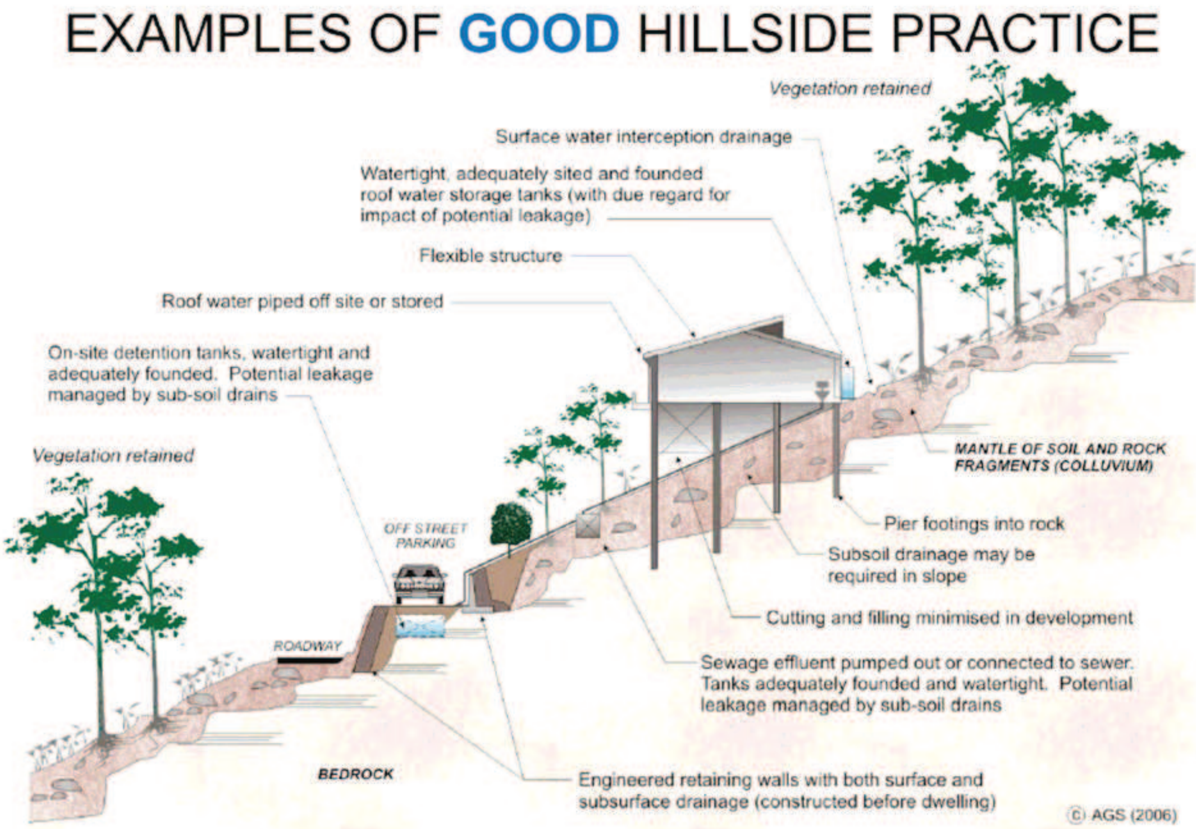
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

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

Appendix 4

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
OFF STREET PARKING
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
ROADWAY
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
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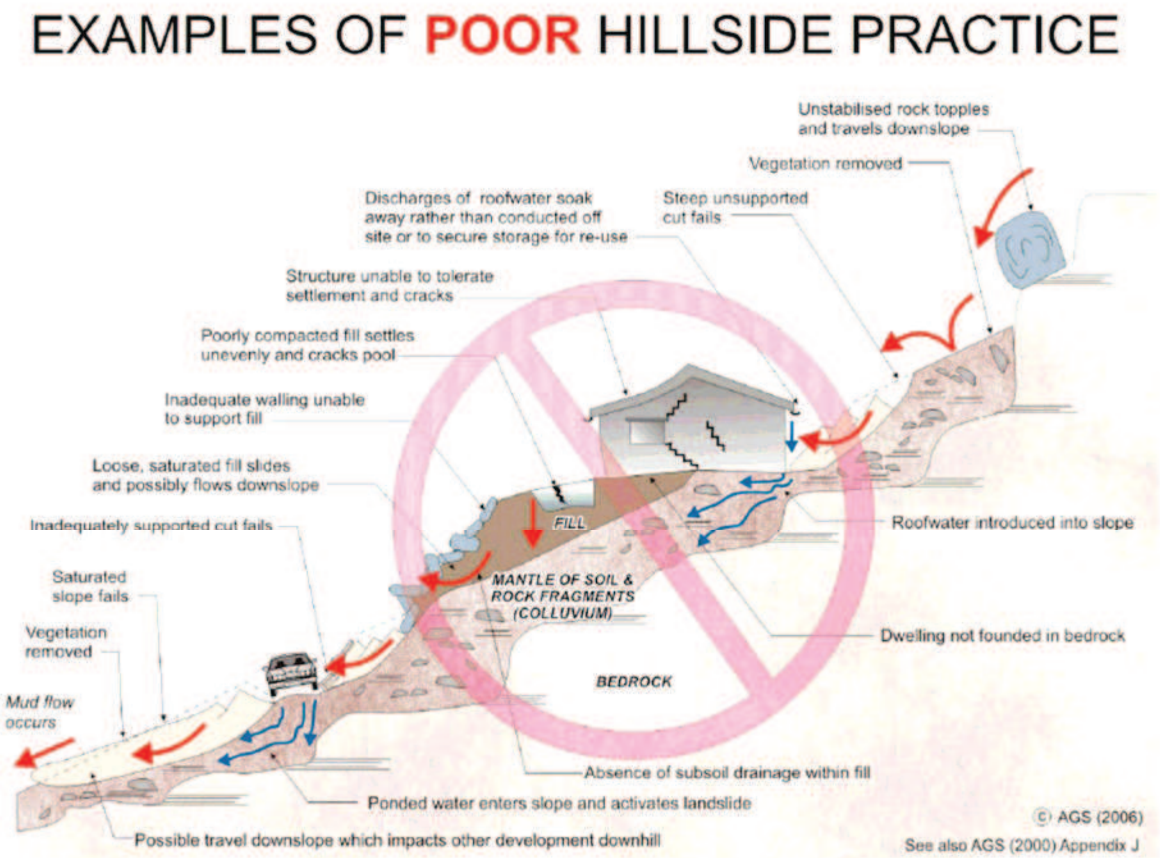
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Dwelling not founded in bedrock
Vegetation removed
BEDROCK
Mud flow occurs
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
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See also AGS (2000) Appendix J