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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

2 DIXON AVENUE, FRENCHS FOREST

Prepared For

Iain Davison

Project No.: 2021-002

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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS
2 DIXON AVENUE, FRENCHS FOREST, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at No.2 Dixon Avenue, Frenchs Forest, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client Iain Davison.

It is understood a basement garage is to be constructed below the north west corner of the site house with an inground swimming pool proposed within the rear of the property. Further works include a first floor extension, a front first floor deck and a rear ground floor deck, along with internal alterations to both living levels. Bulk excavation is anticipated to be required to 1.8m depth for the swimming pool and 2.2m depth for the proposed garage.

A review of Warringah Council's LEP/DCP identified that located predominately within 'Class A' landslip hazard zone (LSR_008A). The proposed works involve excavation >2.00m depth and therefore a 'full' geotechnical report will be required as part of the Development Application (DA).

This report includes a description of site and sub-surface conditions, a geotechnical assessment of the development, site mapping/plan, a geological section, site risk assessment in accordance with AGS March 2007 publication and provides recommendations for design, construction and stormwater disposal.

The investigation and reporting were undertaken as per the Proposal P20-574, Dated: 16th December 2020. The investigation and reporting were prepared to assist in the Development Application and preliminary design and construction tendering.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and inspection of adjacent properties by a Geotechnical Engineer.

b) Drilling of four boreholes using hand tools and one borehole using a restricted access drill rig along with three Dynamic Cone Penetrometer (DCP) tests at specific locations to investigate sub-surface conditions.

The following plans and drawings were supplied for the work:

- Preliminary Architectural Drawings – by client, Rev.: A, Drawn by: LP, Drawing No.: 2020-01-A101 to 2020-01-A107, 2020-01-A117 and 2020-01-A118.
- Survey Drawing – by Bee & Lethbridge, Ref. No.: 21516, Drawn: SC, Drawing No.: 21516 and Sheet 1 of 1 and Rev. No.: 00.

2. SITE FEATURES:

2.1. Site Description:

The site is an irregular pentagonal shaped block covering an area of 628.3m^3, with a front north boundary of 24.4m, a north eastern side boundary of 4.4m, an eastern side boundary of 23.8m, a western side boundary of 27.4m and a rear south boundary of 24.1m, as referenced from the provided Survey Plan. The site is located on the high south side of Dixon Avenue, within gentle (≈3°) north east dipping topography.

Photograph 1. Aerial photograph of site and surrounds

The site contains a one and two storey rendered house located within the central portion of the block. A metal carport is positioned in the north western corner of the site, with a concrete driveway providing vehicle access to Dixon Avenue. The areas surrounding the site house are largely covered by grassed lawns with a small garden situated in the south eastern corner of the site, along with a hedge-line along the boundaries adjacent to the surrounding road reserves.

Photograph 2. View of the front of the house, facing south

2.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is located on the border between Hawkesbury Sandstone (Rh) and Shale of the Wianamatta Group (Rhs) which are both of Triassic age. The Hawkesbury Sandstone rock unit typically comprises of medium to coarse grained quartz sandstone with minor lenses of shale and laminite. The bedrock of the Wianamatta Group rock unit typically comprises of shale and laminites and are interbedded with sandstone, formed within abandoned river channels. They are generally iron rich and commonly contain groundwater. An extract of the Sydney Series sheet is provided as Extract 1.

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 a moderately thick sandy colluvial soil profile that are randomly covered by sandstone boulders. Outcropping sandstone bedrock was observed within the nearby Jimada Reserve.

Extract 1:100,000 Sydney Series Geological reference extract

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection, mapping of the site and limited inspection of adjacent properties on the 13th January 2021 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing features and ground conditions.

It also included the drilling of four auger boreholes (BH1, BH2, BH2a and BH2b) using hand tools and one borehole (BH3) using a restricted access to investigate sub-surface geology.

DCP testing was carried out from the 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 soil conditions and confirm depths to bedrock.

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

3.2. Field Observations:

The site is situated on the corner of Dixon Avenue and Myra Street, within gentle north east dipping topography. Both Dixon Avenue and Myra Street are formed with bitumen pavement and concrete kerb/gutter, Dixon Avenue slopes gently east whilst Myra Street slopes gently north. There were no signs of significant cracking within the surrounding road reserves to suggest any underlying movement.

The site contains a one and two storey rendered house positioned centrally within the site, vehicle access is provided to the north western carport via a concrete driveway which connects to Dixon Avenue. A ground floor timber verandah extends along the northern and eastern sides of the structure, with the first floor level only occupying the northern half of the structure. Grassed lawns surround the house with a hedge-line continuing around the northern and eastern boundaries of the site. Two large trees (gum tree and paperbark tree) are located adjacent to the front entrance to the site, the gum tree is situated approximately 1.0m from the front gate. It appears the root system of the gum tree has lead to root jacking and movement of the front pavers and fence, as shown in Photographs 3 and 4.

Photograph 3: View of the gum tree adjacent to the front entrance of the site, facing south

Photograph 4: Movement within the pavers due to underlying roots of the adjacent gum tree, facing north west

The main structure appeared to be in good condition, however some cracks were observed on the exterior walls of the southern half of the structure. Horizontal cracks within the render were noted on the south eastern corner of the structure, extending up to 3.0m long and up to 1mm wide. Further horizontal cracking was observed within the south western corner and western side of the structure, up to 3.0m long and up to 0.5mm wide. Some of these cracks are shown in Photographs 5 – 7. The planar formation of the cracks observed within the southern portion of the structure indicate that the cracks are most likely due to shrinkage issues within the render and not related to any movement within the underlying geology.

Photograph 5: Horizontal cracking within south eastern corner of structure, facing north west

Photograph 6: Cracking within south western corner of structure, facing north east

Photograph 7: Cracking within western wall of structure, facing north east

Stormwater drainage and gutter systems were observed across the site structure, with down pipes discharging into drainage pits situated adjacent to the base of the structure. A sewer main intersects the north eastern portion of the site, with both redundant clay pipe and active PVC local sewer lines extending from the eastern side of the site house to the adjacent sewer main. Based on available DBYD information it is understood the sewer main comprises a 225mm diameter vitrified clay pipe.

The neighbouring property to the south (No.4 Myra Street) contains a one and two storey rendered cottage positioned 2.0m from the common boundary with the site. The front of the property contains a concrete driveway with surrounding low masonry retaining walls supporting grassed lawns. An inground swimming pool is located within the rear of the property with a grassed lawn adjacent to the rear of the structure. The garage level of the structure appears to be approximately 0.5m – 1.0m below the natural surface level, therefore it is likely that the property has undergone previous bulk excavation. Stormwater drains and gutter systems were identified across the structure, with downpipes extending to unknown discharge points.

The neighbouring property to the west (No.4 Dixon Avenue) contains a single storey brick cottage, with a brick garage which extends to within 0.2m of the common boundary with the site. The property appeared to be currently undergoing construction works with a construction fence positioned at the front of the block, as a result limited inspection of the property was undertaken.

3.3. Ground Conditions:

For a description of the ground conditions encountered at the borehole/DCP test locations, the Borehole Log and DCP results sheets should be consulted however a very broad summary of the subsurface conditions encountered is provided below.

- **TOPSOIL –** this layer was encountered from surface at all test locations to a maximum depth of 0.35m (BH2b) below the existing ground surface. Fill soils surrounding pipes and concrete fragments were identified in BH2 and BH2a, both boreholes refused within such soils.
- **SAND –** Natural sand was encountered in all boreholes except for BH2 & BH2a, to a maximum drilled depth of 0.50m depth (BH2b). The deposit comprised very loose, light brown, medium to coarse grained, moist to wet, sand with fine to medium sub rounded ironstone gravel identified in BH1.
- **SANDSTONE BEDROCK** – Bedrock comprising extremely weathered/extremely low strength sandstone grading to at least very low strength sandstone was encountered within BH1, BH2b & BH3 underlying the natural sand layer. Sandstone bedrock of at least very low strength was interpreted as being encountered between 0.38m (DCP3) and 0.72m (DCP1) depth. Low to medium strength sandstone was encountered at 0.42m depth within BH3.

Seepage was encountered at the soil – bedrock interface within BH1, BH2b and BH3 with a maximum of 100mm of water recorded at the base of BH2b post auger refusal on interpreted sandstone bedrock.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of silty sand topsoil and fill to a maximum depth of 0.35m (BH2b), underlain by natural sand to depths between 0.35m (BH3) and 0.55m (BH2b). Extremely weathered/extremely low strength sandstone bedrock quickly grading to interpreted low to medium strength was encountered underlying the natural sand layer between depths of 0.38m (DCP3) and 0.72m (DCP1). BH2 and BH2a refused within fill soils on a clay pipe and concrete fragments ≤ 0.30m depth. Minor to moderate seepage was encountered at the soil – bedrock interface with the maximum amount of water recorded at the bottom of any of the boreholes following auger refusal was 100mm in BH2b.

The proposed works include alterations to the site house including the extension of the first floor level to the existing southern side of the structure and the construction of front and rear decks. Further works include the construction of a basement garage floor, below the existing northern portion of the structure, with an inground swimming pool also proposed within the south western corner of the site. Bulk excavation to 2.2m depth for the basement garage and 1.8m for the swimming pool are envisaged. It is anticipated that excavation for the basement garage will be located 1.0m from the western boundary of the site. The exact location of the swimming pool is unconfirmed, however it is also anticipated that excavation will extend to within 1.0m from the southern boundary and approximately 2.0m from the western boundary of the site.

It is expected that the excavations will extend through sandstone bedrock of low and potentially medium to high strength along with intersecting sandy fill/natural sand soils to a maximum of approximately 0.5m depth. The excavation of medium to high strength bedrock will require the use of rock excavation equipment which has the potential to create significant ground vibrations of a level that can potentially cause damage to neighbouring structures. Therefore, selection of suitable equipment and a sensible methodology are critical. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use. It is recommended that a rock saw and small (≤200kg) rock hammers are used for the excavation of any bedrock, due to the proximity of the proposed excavations to the boundaries and neighbouring structures.

Considering the depth of the proposed excavation and subsurface conditions it appears that safe batter slopes adjacent to the north, east and south shared boundaries as detailed in Section 4.3 may be achievable. Details of the swimming pool and basement garage floor are limited in the provided architectural drawings therefore exact dimensions and setback distances are unknown. Achievable safe batter slopes will also be subject to the presence of defects within the bedrock encountered during excavation. It is anticipated that seepage will be encountered during excavation (with inflow rate higher during and following periods of rainfall) which will potentially destabilise any batter slopes, regardless of how flat.

The proposed pool excavation appears to be within 0.5m of the southern shred boundary. The proposed garage will be approximately 1.9m from the shared west boundary.

Where batter slopes are not feasible due to proximity of site boundaries or site structures, support prior to excavation will be required. Therefore, shoring walls will need to be installed around any of the excavation perimeters which do not comply with the safe batter slope requirements detailed in Section 4.3. This will likely be required for the swimming pool excavation.

The depth and founding conditions of the existing house footings is unknown. Prior to bulk excavation below the existing footings geotechnical inspection will be required to confirm whether underpinning the existing house footings will be necessary to ensure the stability of the structure.

Existing landslip hazards were not identified however the excavation will create potential stability hazards though these are not expected to impact adjacent properties but will need to be considered during construction.

It is understood that a Sydney Water (SW) sewer underlies the site and near the proposed garage excavation. CGC as not undertaken any investigation into the construction/type/depth etc. of the sewer. Based on previous experience it is recommended that Sydney Water be contacted as soon as possible to determine what requirements may exit in order to protect the asset. This will likely be a condition of the CC however the precautions SW may require could impact the scope of required field investigation, geotechnical reporting and footing design.

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

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and a limited scope of investigation using augering techniques only. This investigation 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 analysis and subsequent design of the proposed works.

4.2. Site Specific Risk Assessment:

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

A. Toppling/sliding of unstable block of rock formed by intersecting defects within basement garage excavation.
B. Toppling/sliding of unstable block of rock formed by intersecting defects within swimming pool excavation.
C. Landslip of surficial soils from excavation works for swimming pool.

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

The **Risk to Life** from **Hazard A, B and C** were estimated to be up to **2.67 x 10^{-7}** for a single person, whilst the **Risk to Property** from the hazards were considered to be up to **'Moderate'**.

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

4.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' due to the sandy nature of the soils and the relatively shallow depth to bedrock
Type of Footing	Strip/Pad or Slab at base of excavations
Sub-grade material and Maximum Allowable Bearing Capacity	- Weathered, VLS Sandstone: 800kPa - Weathered LS Sandstone: 1000kPa - Weathered MS Sandstone: 2000kPa

Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – rock site
Remarks: All footings should be founded off bedrock of similar strength to prevent differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

4.3.2. Excavation:	
Depth of Excavation	2.2m for basement garage, approximately 1.8m for swimming pool
Type of Material to be Excavated	Very loose fill/topsoil to maximum of 0.35m depth
	Loose sand to maximum of 0.55m depth
	ELS-VLS bedrock from between 0.35m and 0.65m depth
	>0.65m LS – MS/HS bedrock

Guidelines for batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill and natural soils	1:1	2:1
Extremely Low to Low strength fractured bedrock	1:1	1.25:1
Low strength defect free bedrock	Vertical*	0.25:1.0
Medium strength, defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by geotechnical engineer

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

Equipment for Excavation	Topsoil and residual soils	Excavator with bucket
	ELS bedrock	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS – HS bedrock	Rock hammer and saw

VLS – very low strength, LS – low strength, MS – medium strength, HS – high strength

<table>
<tr><td colspan="2">

Remarks: It is recommended that the hard rock excavation perimeter be saw cut prior to rock hammering, this will generally reduce the amount of rock support required, reduce deflection of rock across boundary and under neighbouring structures and will provide a slight buffer distance to ground vibrations for the use of rock hammers.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength sandstone bedrock, to achieve the specified low level of vibration the below tabulated hammer weights and buffer distances are required:

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

Onsite calibration will provide accurate vibration levels for the site specific conditions and will generally allow for larger excavation machinery or smaller buffer distances to be used. Calibration of rock excavation machinery will need to be carried out prior to commencement of bulk rock excavation works and will determine the need for full time monitoring.
</td></tr>
<tr><td>Recommended Vibration Limits
(Maximum Peak Particle Velocity (PPV))</td><td>No.4 Dixon Street = 5mm/s
No. 4 Myra Street = 5mm/s
Service Lines = 3mm/s</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td>Recommended</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td>Pending proposed equipment and vibration calibration testing results</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td>Yes, recommended that these inspections be undertaken as per below mentioned sequence:
• During installation of any support measures
• Following clearing of bedrock surface
• Every 1.50m depth interval of the main excavation where unsupported
• Where low to medium strength bedrock is exposed
• At completion of the excavation.</td></tr>
<tr><td>Dilapidation Surveys Requirement</td><td>On neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit</td></tr>
</table>

	and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

4.3.3. Retaining Structures:	
Required	Surficial soils above and near excavation crests be retained prior to bulk excavation to minimize the risk of a soil slide into the excavation. This can be achieved by either clearing the soils away from the excavation crests (batters), horizontal benches or by construction of a temporary and/or permanent retaining structure founded off the bedrock surface.
Types	Soldier piles/piers, steel reinforced concrete/concrete block wall or a plank wall supported by dowels founded into bedrock designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill (sandy) (very loose)	18	$\phi' = 27°$	0.38	0.55	N/A
Sand (loose)	18	$\phi' = 29°$	0.35	0.52	N/A
ELS bedrock	22	$\phi' = 38°$	0.15	0.20	400 kPa
LS bedrock	23	$\phi' = 40°$	0.10	0.15	700kPa

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		Minor to moderate seepage encountered in all boreholes drilled across the site.
Excavation likely to intersect	Water Table	No
	Seepage	Moderate
Site Location and Topography		On high side of road within gentle north east dipping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not required or recommended
Remarks: Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow. This can result in relaxation of excavation faces causing instability. Therefore excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional. A stormwater diversion drain should be installed upslope of excavation crests to intercept stormwater runoff and prevent erosion and softening of the excavation faces. An excavation trench should also be installed at the base of excavation cuts to below floor slab levels to reduce the risk of long term dampness. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site.		

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural design drawings and final architectural drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Supervise installation of any support measures,
3. Inspect any exposed low to medium strength bedrock and the proposed excavation equipment prior to its use
4. Inspect excavation at 1.50m depth intervals

5. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
6. Inspect completed works to ensure no new landslip hazards have been created by site works and that all required stabilisation and drainage measures are in place.

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 very loose topsoil and fill to a maximum of 0.35m depth (BH2b), natural sand was encountered underlying the topsoil and fill to a maximum depth of 0.55m depth (BH2b). Extremely weathered/extremely low strength sandstone bedrock grading to at least low strength bedrock was interpreted underlying the natural soils, with refusal encountered between 0.42m (BH3) and 0.72m depth (DCP1).

The proposed works involve bulk excavation to a maximum of 2.2m depth for a basement garage floor and inground swimming pool. Further works include the extension of the existing first floor level, construction of front and rear decks, along with internal renovations.

Careful consideration should be taken during the excavation phase for the swimming pool and basement Garage Floor, as it is anticipated that the excavations will extend to a minimum of approximately 1.0m from the boundaries, within close proximity of neighbouring structures. The majority of excavation will intersect sandstone bedrock of low to medium/high strength, therefore suitable rock excavation equipment (i.e. rock saws and rock hammers ≤200kg) along with an appropriate excavation methodology should be used.

It appears batter slopes may be feasible to support the excavation sides however further details of the proposed pool excavation will be required to confirm this. Achievable safe batter slopes will also be subject to the presence of defects within the bedrock encountered during excavation.

There were no existing/credible landslip hazards identified and the proposed works are relatively minor from a geotechnical perspective and should not create any new instability provided the recommendations of this report are implemented. However, due to the poor condition of the retaining wall adjacent to the swimming pool, it is recommended that remedial work is undertaken prior to any construction or excavation work.

The risks associated with the proposed development can be maintained within 'Acceptable' levels (AGS 2007) with negligible impact to the 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:
Josh Cotton	Kieron Nicholson
Engineer	Senior Engineering Geologist

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

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

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

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

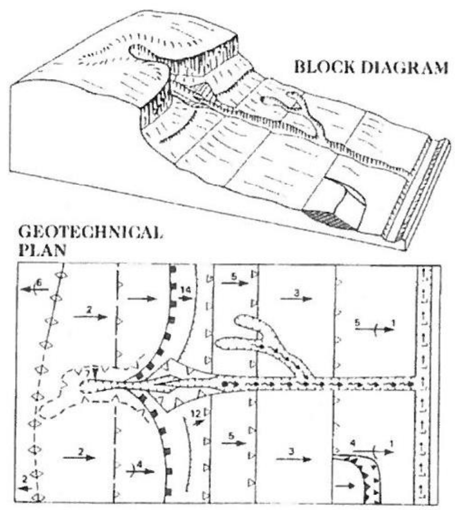
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

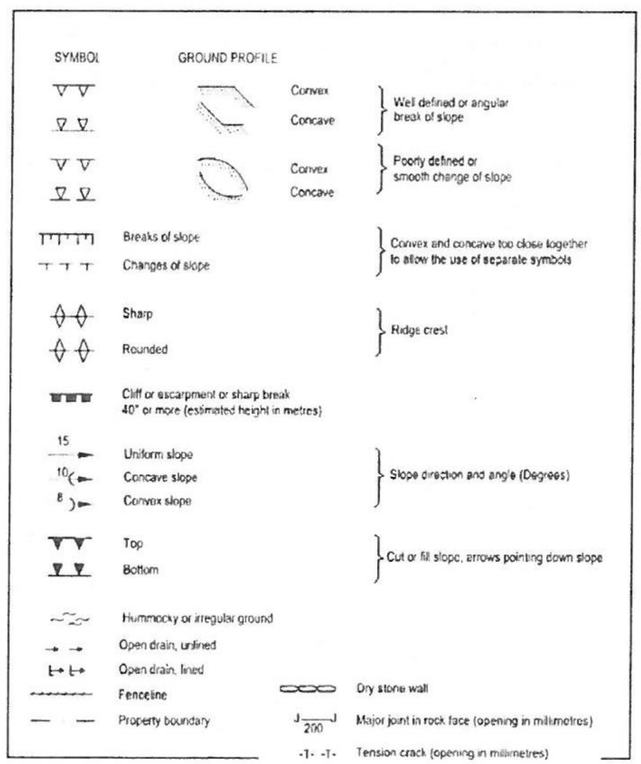
Reproduction of Information for Contractual Purposes

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

Site Inspection

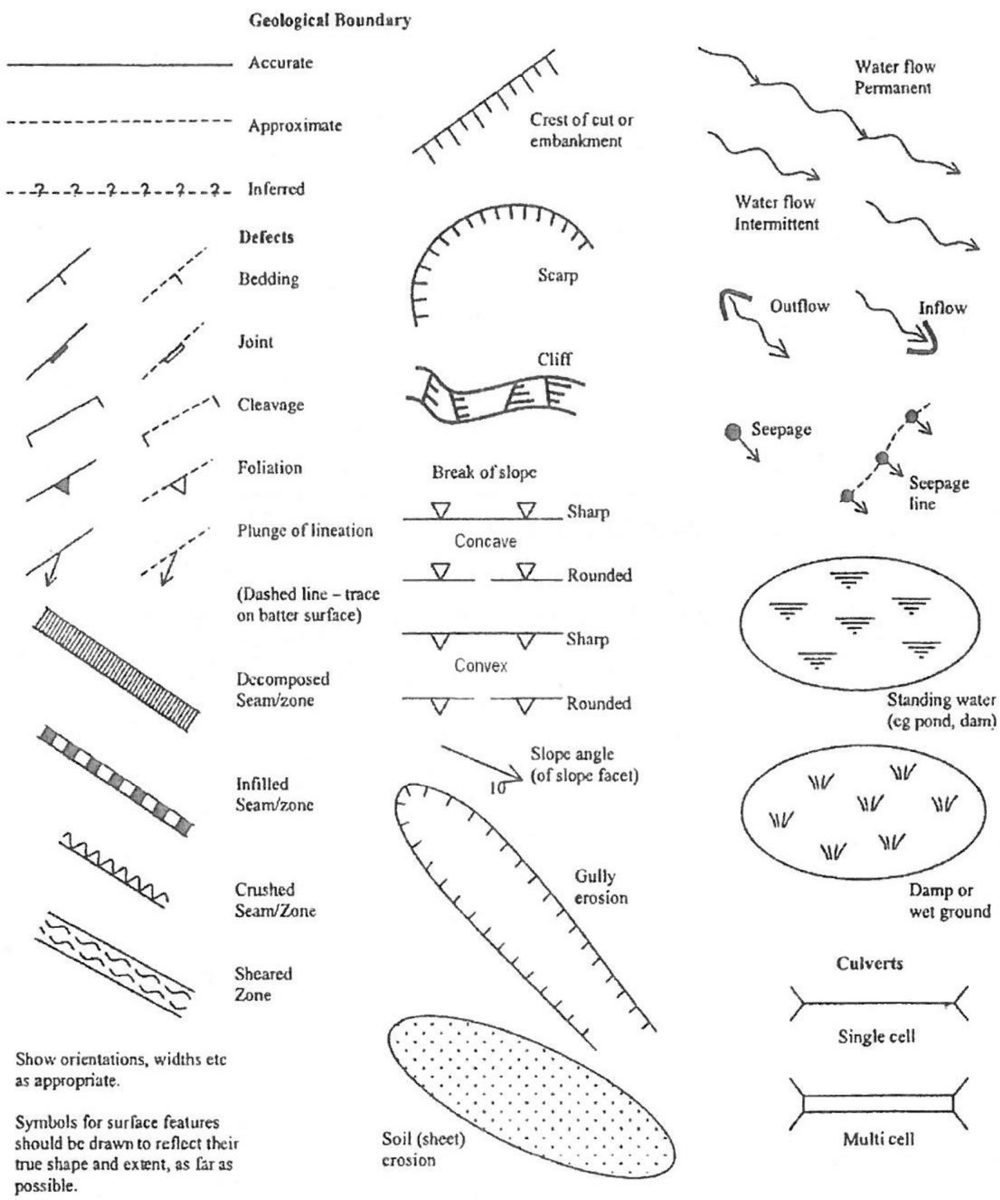
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

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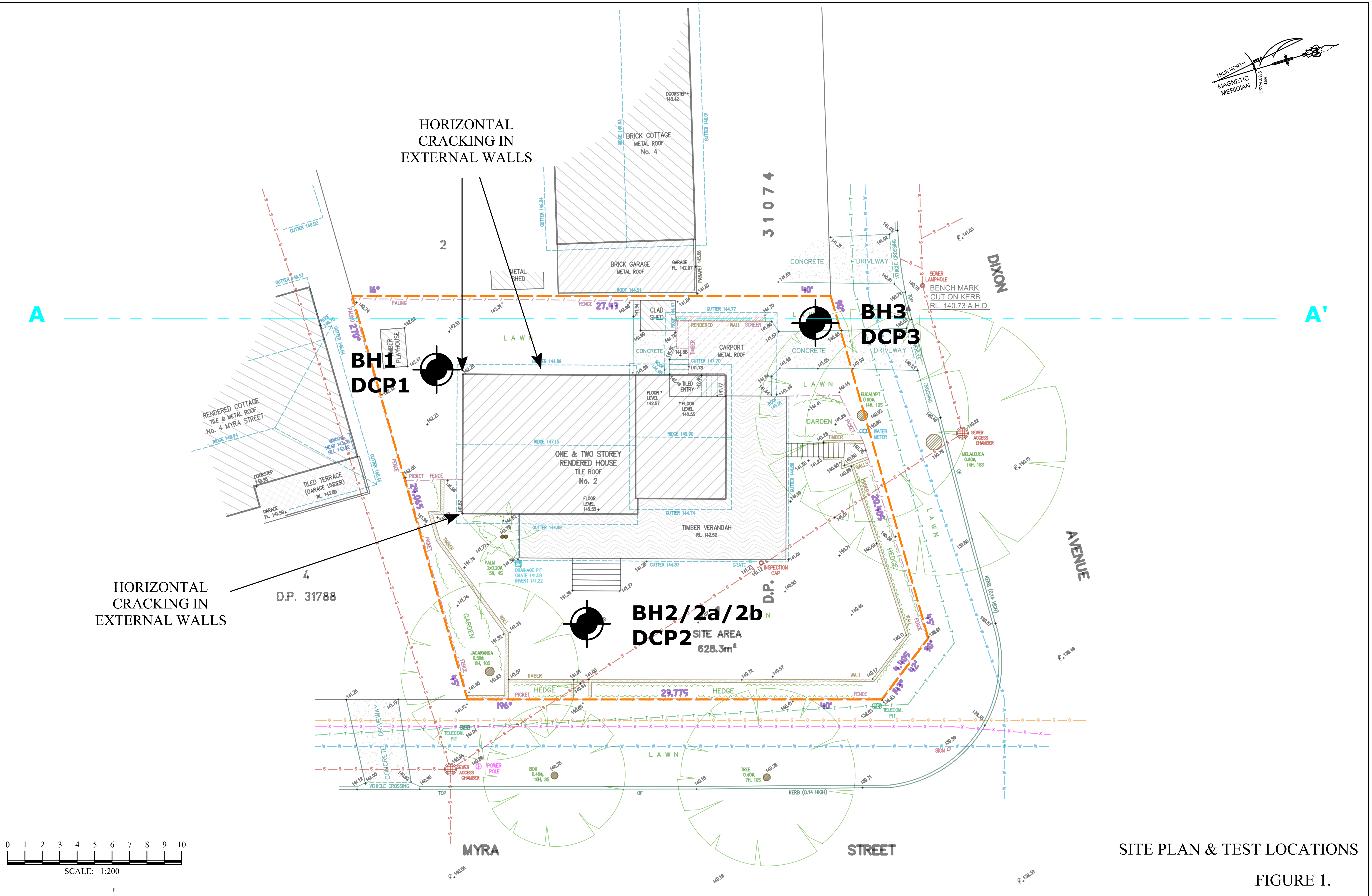
Appendix 2



SITE PLAN & TEST LOCATIONS

FIGURE 1.

LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
A —— A'	CROSS-SECTION REFERENCE LINE
(orange dashed line)	PROPERTY BOUNDARY

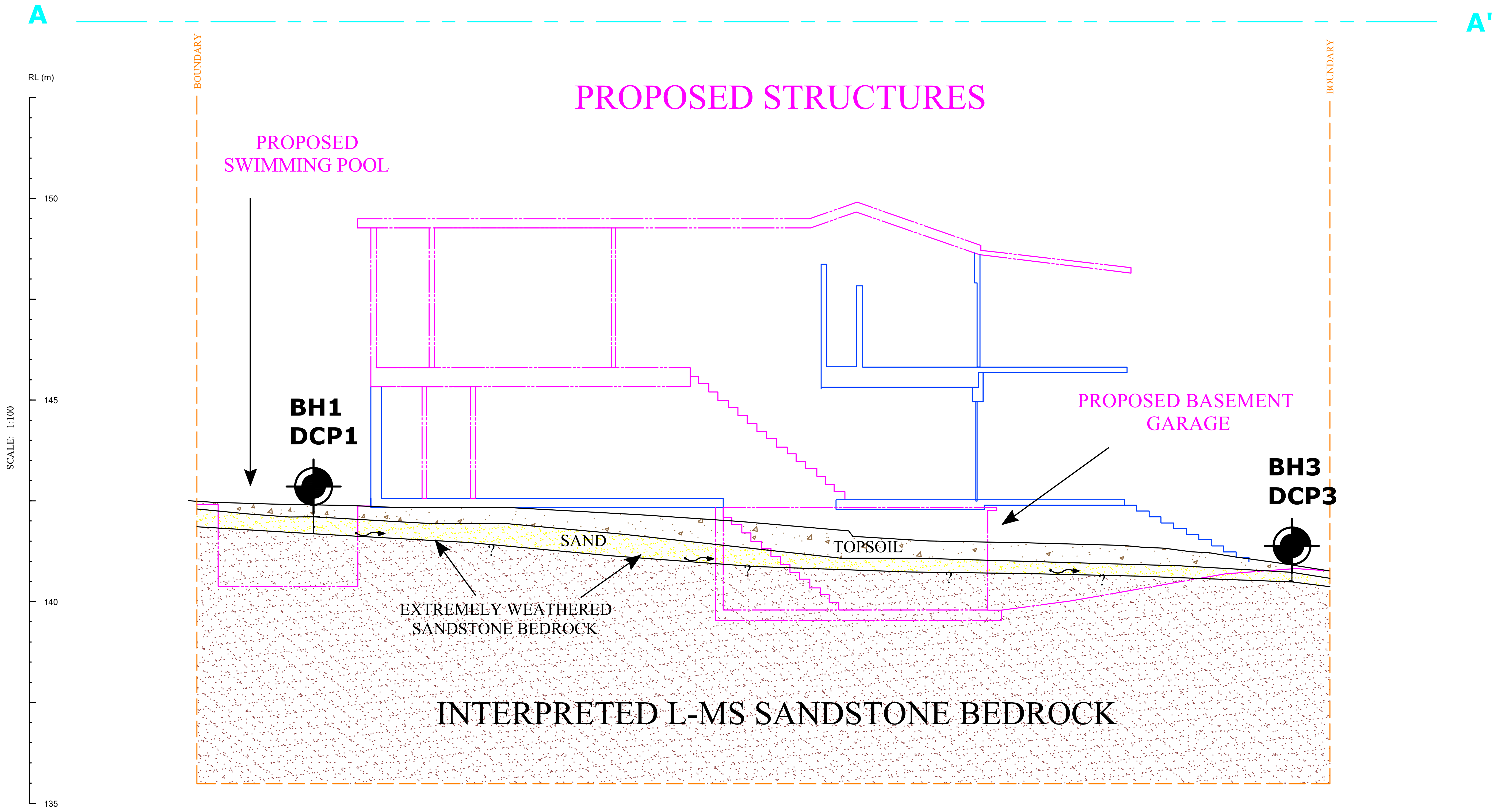
CROZIER GEOTECHNICAL CONSULTANTS

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

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	14/01/2021
APPROVED BY:	TMC
DRAWN BY:	JC
PROJECT:	2021-002

PREPARED FOR: IAIN DAVISON

ADDRESS: 2 DIXON AVENUE, FRENCHS FOREST



0 1 2 3 4 5

SCALE: 1:100

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

BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION

A — A' CROSS-SECTION REFERENCE LINE

EXISTING STRUCTURE

PROPOSED STRUCTURES

PROPERTY BOUNDARY

TOPSOIL/FILL

SAND

SANDSTONE BEDROCK

SEEPAGE

SCALE: 1:100 @ A3
DRAWING: FIGURE 2
DATE: 14/01/2021

APPROVED BY: TMC
DRAWN BY: JC
PROJECT: 2021-002

PREPARED FOR:
IAIN DAVISON

ADDRESS:
2 DIXON AVENUE,
FRENCHS FOREST

BOREHOLE LOG

CLIENT: Iain Davison
DATE: 13/01/2021
BORE No.: 1

PROJECT: Alterations and Additions including construction of a basement garage and swimming pool
PROJECT No.: 2021-002
SHEET: 1 of 1

LOCATION: 2 Dixon Avenue, Frenchs Forest
SURFACE LEVEL: RL= 142.4m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL: Very loose, dark brown, medium grained, moist, silty sand				
0.30	SP	SAND: Loose, light brown, medium to coarse grained, moist, sand trace clay				
0.50		…sand with fine to medium, sub-rounded ironstone gravel				
0.55		AUGER REFUSAL @ 0.55m depth on interpreted sandstone bedrock of at least very low strength				
1.00						
2.00						

RIG: N/A
DRILLER: AC
METHOD: Hand Auger
LOGGED: JC
GROUND WATER OBSERVATIONS: Minor seepage at soil - bedrock interface

REMARKS:
CHECKED: KN

BOREHOLE LOG

CLIENT: Iain Davison **DATE:** 13/01/2020 **BORE No.:** 2

PROJECT: Alterations and Additions including cons **PROJECT No.:** 2021-002 **SHEET:** 1 of 1

LOCATION: 2 Dixon Avenue, Frenchs Forest **SURFACE LEVEL:** RL= 141.2m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL: Very loose, dark brown, fine to medium grained, moist to wet, silty sand with rootlets				
0.20		…brown with gravel				
0.25		…aggregate				
0.30		AUGER REFUSAL @ 0.30m depth on clay pipe				
1.00						
2.00						

RIG: N/A DRILLER: AC

METHOD: Hand Auger LOGGED: JC

GROUND WATER OBSERVATIONS:

REMARKS: CHECKED: KN

BOREHOLE LOG

CLIENT: Iain Davison **DATE:** 13/01/2020 **BORE No.:** 2a

PROJECT: Alterations and Additions including cons **PROJECT No.:** 2021-002 **SHEET:** 1 of 1

LOCATION: 2 Dixon Avenue, Frenchs Forest **SURFACE LEVEL:** RL= 141.2m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling: Type	Sampling: Tests	In Situ Testing: Type	In Situ Testing: Results
0.00		TOPSOIL: Very loose, dark brown, fine to medium grained, moist to wet, silty sand with rootlets				
0.20		AUGER REFUSAL @ 0.2m depth on concrete				
1.00						
2.00						

RIG: N/A DRILLER: AC

METHOD: Hand Auger LOGGED: JC

GROUND WATER OBSERVATIONS: None encountered during auger drilling

REMARKS: CHECKED: KN

BOREHOLE LOG

CLIENT: Iain Davison | **DATE:** 13/01/2020 | **BORE No.:** 2b

PROJECT: Alterations and Additions including cons | **PROJECT No.:** 2021-002 | **SHEET:** 1 of 1

LOCATION: 2 Dixon Avenue, Frenchs Forest | **SURFACE LEVEL:** RL= 141.2m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSOIL: Very loose, dark brown, fine to medium grained, moist to wet, silty sand with rootlets				
0.35	SP	SAND: Very loose, brown, medium to coarse grained, wet, sand				
0.50		SANDSTONE: Extremely weathered, pale grey, medium to coarse grained	D	0.50		
0.60		…saturated		0.60		
0.65		AUGER REFUSAL @ 0.65m depth on interpreted sandstone bedrock of at least very low strength				
1.00						
2.00						

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: JC

GROUND WATER OBSERVATIONS: Seepage encountered at 0.6m depth, 100mm of water in bottom of borehole after drilling

REMARKS:

CHECKED: KN

BOREHOLE LOG

CLIENT: Iain Davison **DATE:** **BORE No.:** 3

PROJECT: Alterations and Additions including cons **PROJECT No.:** 2021-002 **SHEET:** 1 of 1

LOCATION: 2 Dixon Avenue, Frenchs Forest **SURFACE LEVEL:** RL= 141.0m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSIOL: Very loose, dark brown, medium grained, wet, silty sand with rootlets				
0.20	SP	SAND: Very loose, light brown, medium to coarse grained, moist, sand				
0.35		SANDSTONE: Extremely weathered, pale grey, coarse grained	D	0.35		
0.42		AUGER REFUSAL @ 0.42m depth on interpreted sandstone bedrock of low to medium strength		0.42		
1.00						
2.00						

RIG: Dingo DRILLER: AC

METHOD: Single stem spiral flight tungsten carbide bit LOGGED: JC

GROUND WATER OBSERVATIONS: Wet topsoil layer with 0.05m of water at bottom of borehole after drilling

REMARKS: CHECKED: KN

DYNAMIC PENETROMETER TEST SHEET

CLIENT: **Iain Davison** **DATE:** 13/01/2021

PROJECT: **Alterations and Additions including construction of a basement garage and swimming pool** **PROJECT No.:** 2021-002

LOCATION: **2 Dixon Avenue, Frenchs Forest** **SHEET:** 1 of

	Test Location							
Depth (m)	DCP1	DCP2	DCP3					
0.00 - 0.15	1	1	1					
0.15 - 0.30	2	1	2					
0.30 - 0.45	2	0	18*B @0.38m					
0.45 - 0.60	3	1						
0.60 - 0.75	30*B @0.72m	28*B @0.70m						
0.75 - 0.90								
0.90 - 1.05								
1.05 - 1.20								
1.20 - 1.35								
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

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

-- No test undertaken at this level due to prior excavation of soils

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (rock slide/topple <5m³) within rock excavation for basement garage floor		Excavation to 2.2m depth, up to 1.9m through bedrock, likely unfavourable defects in some portion	a) House 1.3m from 2.2m deep excavation, impacted 10% b) Front lawn 1.0m from 2.2m deep excavation, impacted 15% c) Site house adjacent to excavation, impacted 20%		a) Person in house 16hrs/day avge. b) Person in garden 1hrs/day avge. c) Person in house 16hr/day avge.	a) Possible to not evacuate b) Unlikely to not evacuate c) Possible to not evacuate	a) Person in building, minor damage only a) Person in open space, possible buried c) Person in building, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No.4 Dixon Avenue	0.001	0.10	0.10	0.6667	0.5	0.01	**3.33E-08**
		b) Front lawn No.4 Dixon Avenue	0.001	0.10	0.15	0.0417	0.25	0.50	**7.81E-08**
		c) Site house	0.001	0.40	0.20	0.6667	0.5	0.01	**2.67E-07**
B	Landslip (rock topple/slide ≤3m³) within rock excavation for swimming pool		Excavation to 1.8m depth, up to 1.1m through bedrock, possible unfavourable defects in some portions	a) House 2.5m from 1.8m deep excavation, impacted 5% b) Northern access path 0.5m from 1.8m deep excavation, impacted 80% c) House 0.5m from 1.8m excavation, impacted 10%		a) Person in house 16hrs/day avge. b) Person on path 0.5hr/day avge. c) Person in house 16hrs/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate	a) Person in building minor damage only b) Person in open space, possible buried c) Person in building, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No.4 Myra Street	0.001	0.10	0.05	0.6667	0.5	0.01	**1.67E-08**
		b) Northern access path No.4 Myra Street	0.001	0.10	0.80	0.0208	0.5	0.50	**4.17E-07**
		c) Site house	0.001	0.10	0.10	0.6667	0.5	0.01	**3.33E-08**
C	Landslip (earth slide ≤0.5m³) at crest of excavation for swimming pool		Excavation to 1.8m depth, up to 0.7m through soils at top of excavation	a) House 2.5m from 1.8m deep excavation, impacted 2% b) Northern access path 0.5m from 1.8m deep excavation, impacted 10% c) House 0.5m from 1.8m excavation, impacted 3%		a) Person in house 16hrs/day avge. b) Person on path 0.5hr/day avge. c) Person in house 16hrs/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate	a) Person in building minor damage only b) Person in open space, possible buried c) Person in building, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No.4 Myra Street	0.001	0.01	0.02	0.6667	0.5	0.01	**6.67E-10**
		b) Northern access path No.4 Myra Street	0.001	0.02	0.10	0.0208	0.5	0.50	**1.04E-08**
		c) Site house	0.001	0.02	0.03	0.6667	0.5	0.01	**2.00E-09**

* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.

* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide 2m³) from soils at crest of excavation for lower ground floor extensions	a) House No.4 Dixon Avenue	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		b) Front lawn No.4 Dixon Avenue	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		c) Site house	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
B	Landslip (rock topple/slide ≤3m³) within rock excavation for swimming pool	a) House No.4 Myra Street	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		b) Northern access path No.4 Myra Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		c) Site house	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Moderate
C	Landslip (earth slide ≤0.5m³) at crest of excavation for swimming pool	a) House No.4 Myra Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very Low
		b) Northern access path No.4 Myra Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very Low
		c) Site house	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

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Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	$5x10^{-2}$	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.

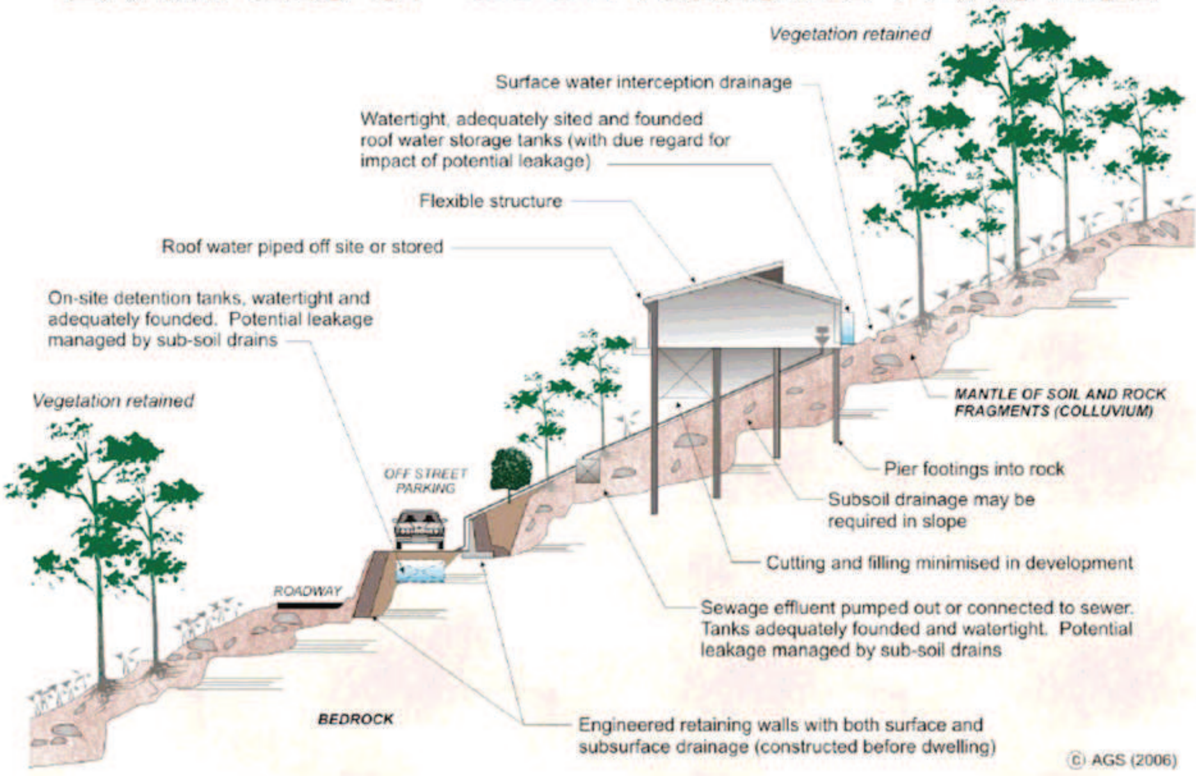
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE