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REPORT ON GEOTECHNICAL INVESTIGATION AND RISK ASSESSMENT

for

PROPOSED NEW RESIDENTIAL BUILDING

at

5 LAUDERDALE AVENUE, FAIRLIGHT, NSW

Prepared For

HPG Project Lauderdale Pty Ltd

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REPORT ON GEOTECHNICAL INVESTIGATION AND RISK ASSESSMENT FOR THE PROPOSED NEW RESIDENTIAL BUILDING AT 5 LAUDERDALE, FAIRLIGHT, NSW

1. INTRODUCTION:

This report details the results of a Geotechnical Investigation and Risk Assessment carried out to provide advice and recommendations for a Development Application (DA) for the proposed Class 2 Residential Building at 5 Lauderdale, Fairlight, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the client, HPG Project Lauderdale Pty Ltd.

2. PROPOSED DEVELOPMENT:

The following documents supplied by the Architects, were relied on for the preparation of the proposal and this report:

- Architectural Drawings for DA submission prepared by Platform Architects – Project No: 28, Drawing Nos:
  - DA0050, DA0100, DA0400, Revision. 1, dated 17 May 2024;
  - DA0500, DA1001 to DA1005, DA1950, DA2000 to DA2001, DA3000 to DA3001, Revision. 2, dated 24 May 2024;
- Survey Drawing prepared by Mitch Ayers Surveying Pty Ltd - Project No.: REF-231207, Sheets: 1-2, Dated: 21 December 2023.

  The datum in the survey plan is in Australian Height Datum (AHD), therefore, all the Reduced Levels (RL) mentioned in this report are henceforth in AHD.

Based on the provided documents, it is understood the proposed works involve the demolition of existing site structures and the construction of a Class 2, four-storey unit building with a basement level. The proposed basement has a designed Finished Floor Level (FFL) of RL 5.05m, which indicates a Bulk Excavation Level (BEL) of about RL 4.8m will be required. To achieve this, an excavation up to approximately 8.0m will depth be required. A locally deeper excavation to about RL 3.6m will be required for a car lift and other deeper excavations may be required for footings and service trenches.

Based on Schedule 1 Map C and the Northern Beaches Mapping portal of the Development Control Plan (DCP) 2013 from Northern Beaches (Manly) Council, the site is located within Landslip Risk Class 'G4' and adjacent to the 'G2' area.

A review of the preliminary slope stability assessment checklist and the proposed works identified that the Development Application will require a geotechnical report including a full site stability assessment. Therefore, a full report must detail how the development may be achieved to ensure geotechnical stability and good engineering practice and also include a risk assessment for existing/potential instability as per the AGS March 2007 publication.

The Acid Sulphate Soils (ASS) Map of Manly Local Environmental Plan 2013 indicates that the site is located within a Class 5 acid sulphate soils hazard zone. As such, a preliminary assessment is required to determine if the proposed works, where within 500m of adjacent Class 1 to 4 land, are likely to lower the water table below 1 meter AHD on the adjacent Class 1 to 4 land.

3. OBJECTIVES AND SCOPE OF WORK

This report is provided to satisfy Council requirements as part of a Development Application (DA) submission. It includes a description of the fieldwork, borehole logs, in-situ test results, a plan showing the test location, and a geological section. An assessment of Geotechnical Hazard Risk is also included to the methods of AGS 2007. The site investigation and reporting were undertaken as per the Fee Proposal P24-092, dated 15 March 2024.

The investigation comprised:

a) Dial Before You Dig (DBYD) plan review for service mains and onsite safety assessment;
b) Detailed geotechnical mapping of the site and adjacent properties with a photographic record and identification of geotechnical conditions and hazards related to the existing site and proposed works;
c) A photographic record of site conditions;
d) Drilling of five boreholes using hand auger with Dynamic Cone Penetrometer (DCP) tests to identify the sub-surface geology to refusal across the site;
e) Soil sample collection and logging as per "AS1726: 2017 Geotechnical Site Investigation".

All fieldwork was supervised by an experienced Geotechnical Engineer who logged all geotechnical data.

4. SITE FEATURES:

4.1. Description:

The site is a trapezoidal-shaped block (Lot A in DP24923) situated on the low, south side of Lauderdale Avenue within generally gently to moderately south-dipping topography that slopes down to Fairlight Beach and then North Harbour.

The site levels vary from a low of about RL 6.70m at the southwestern site corner to about RL 13.10m at the northwestern site corner. It has an angled front north boundary of about 19.1m, a west side boundary of 53.0m, an east side boundary of about 42.8m and a rear south boundary of about 20.7m as referenced from the provided survey plan.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government SIX Maps spatial data (Photograph-1). General views of the site are shown in Photograph 2 and Photograph 3.

Based on the orientation in the outcrops, the bay to the south probably indicates a local drainage gully.

Photograph-1: Aerial photo of site and surrounds (source: SIX Maps, access 21/6/2024)

4.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is located near the boundary between Hawkesbury Sandstone (Rh) and Coarse quartz Sand (Qhb). The Hawkesbury Sandstone (Rh) typically comprises medium to coarse-grained quartz sandstone with minor lenses of shale and laminate.

The Coarse Quartz Sand (Qhb) typically comprises coarse-grained sand with varying amounts of shell fragments and is deposited over the bedrock within the adjacent foreshore beach environment.

Extract-1 of Sydney: 1:100 000 – Geology underlying the site

5. FIELDWORK:

5.1. Methods:

The field investigation comprised a walkover within the site and a limited inspection of adjacent properties along with a subsurface investigation on 20 June 2024 both of which were supervised by a Geotechnical Engineer. It included a photographic record of site conditions as well as a geological/geomorphological inspection of the site and adjacent land.

The subsurface investigation comprised the drilling of five boreholes (BH1 – BH5) using a hand auger to investigate the sub-surface geology and collected samples as per "AS1726: 2017 Geotechnical Site Investigation".

Six Dynamic Cone Penetrometer tests (DCP1 to DCP6) were carried out from the ground surface adjacent to boreholes and then at the base of the boreholes, 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.

Following completion, the boreholes were backfilled with recovered soil and surface compacted.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown in Figure: 1, along with detailed bore log and DCP sheets in Appendix: 2. Two geological models/sections are provided in Figures: 2 and 3, Appendix: 2.

5.2. Field Observations:

During the investigation, the site was occupied by a single-storey brick dwelling (Photographs 2 to 5) located in the front northern half of the land with a front detached brick garage (Photograph 6) at the northwestern site corner. The main house and garage appeared in fair condition without any signs of significant cracking observed on the external walls. The remaining area was occupied by a front brick fence (Photograph 2), a rear colorbond fence (Photograph 3), concrete footpaths, stairs, a verandah timber deck, trees and a grassy lawn.

Outcropping sandstone was observed within Fairlight Beach area to the southern site boundary (Photograph 7). No indications of significant distress were observed within the structures within the site.

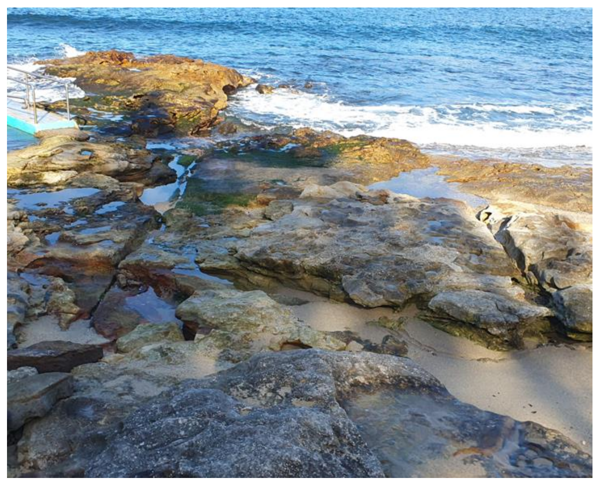
Photograph 2 – Front view of the property and brick-rendered fence, looking south from the roadway.

Photograph 3 – Rear view of the property with a rear colorbond fence, looking north from Fairlight Beach area.

Photograph 4 – Front view of the main dwelling, looking south from the front yard.

Photograph 5 – Rear view of the main dwelling, looking north from the rear yard.

Photograph 6 – View of the front brick garage, looking south from the roadway.



Photograph 7 – View of sandstone outcrops within Fairlight Beach, looking low south from the southern site boundary.

Inspection of the existing structures within the site did not indicate that any significant geotechnical issues exist within the property or neighbour land that may have an impact on the proposed development (e.g., back scars, cracking in brickwork, etc.) more than what would be anticipated based on the age, type of structures and anticipated ground conditions.

This site is located in a residential area with the surrounding description as follows:

- East: 3A and 3B Lauderdale Avenue, a two-storey brick-rendered residential dual occupancy with attached garages, concrete driveways, and grassy areas. The main structures have an offset of about 2.0m to the eastern site boundary. The main structure appears to be new-built within 10 years was observed in good condition without any significant sign of cracking or geotechnical issues. This property has a similar elevation and topography to the site.

- South: Fairlight Beach area, a sandy beach with a beach rock pool, a concrete-paved footpath, tree and grassy area. The footpath and the pool have offsets of about 2.0m and more than 10.0m to the boundary. No apparent sign of cracking was observed within the footpath. The beach area has a lower elevation than the site.
- West: 7 Lauderdale Avenue, a three-storey brick-rendered residential building (built within 10 years) with a basement, concrete driveway, a rear footpath, trees, and grassy areas. The main building including the basement appeared in good condition and has an offset of about 2.5m to the western site boundary. No signs of significant cracking were observed on the external walls during the investigation. This property has a similar elevation and topography to the site.
- North: Lauderdale Avenue, a two-lane, asphalt-paved roadway with concrete kerbs, concrete-paved footpaths, grassy easement and kerbside parking on both sides. The footpath abuts the northern 'site' boundary. The roadway has an offset of about 2.5m to the boundary and dips gently towards the east where it passes the site. No apparent sign of cracking was observed within the roadway. The roadway has a higher elevation to the site.

5.3. Investigation Results:

Based on the field borehole logs and DCP test results, the stratigraphy of the site can be classified into three geological units shown as follows. More detailed descriptions of subsurface conditions at each borehole location are provided on the individual borehole logs presented in Appendix: 1.

- **TOPSOIL/FILL – Silty SAND** was encountered from the surface to depths of between 0.6m (BH3) and 2.4m (BH5) below the existing ground surface with trace of rootlets and gravels;
- **NATURAL SOIL – Silty CLAY/Clayey (Silty) SAND** was encountered within BH1, BH3 and BH4 only and was interpreted from DCP6. It was below the fill profiles to depths varying from 0.9m (BH3) to about 1.0m depth (BH1 and BH4) below the existing ground surface grading into extremely weathered material with depth.
  - The clay soil is classified as firm to stiff, low to medium plasticity, moist, dry of plastic limit, and contains a trace of fine-grained sand and iron-stained gravels; and
  - The sandy soil is classified as very loose to very dense (generally increasing with the depth), fine to medium grained, moist, with trace of silt and clay.
- **BEDROCK – SANDSTONE** was encountered below the natural soil or fill profiles from the investigated depths varying from about 0.9m (RL 9.3m in BH3) to about 2.4m (RL 10.4m in BH5). Within the natural south-dipping topography, the sandstone bedrock is encountered at the level from about RL 7.2m (BH1) within the southern portion of the site to about RL 11.9m (BH4)within the northern portion of the site.

  The sandstone bedrock was interpreted as very low to low strength, moderately to highly weathered.

Very low (VLS) to medium strength (MS) sandstone and potential shale/siltstone were interpreted based on the results of the DCP tests and observed outcropping as MS sandstone bedrock within the beach area.

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

6. COMMENTS:

6.1. Geotechnical Assessment:

The site investigation identified the presence of topsoil/fill over the project site, overlying clayey/sandy natural soil encountered within BH1, BH3 and BH4 only, grading into extremely weathered materials with depth. Sandstone bedrock of at least very low strength was encountered/interpreted from the investigated depths varying from about 0.9 (RL 9.3m in BH3) to about 2.4m (RL 10.4m in BH5) below the existing ground surface. Sandstone outcrops of at least low strength were observed within the beach area to the southern site boundary and it is interpreted that the bedrock will grade gradually from low strength to medium strength with depth upon intersection; however, some strength inversions could be anticipated.

Due to the nature of DCP testing, it is required that the bedrock be confirmed within an additional geotechnical investigation in the form of rock coring prior to the design and Construction Certificate (CC).

Groundwater was not encountered in the investigation. However, minor seepage is expected at the soil-bedrock interface and along the defects in the sandstone and is expected within the anticipated depth of the proposed excavation based on the site location and topography.

The proposed works involve the demolition of existing site structures and the construction of a Class 2, four-storey unit building with a basement level, which requires a BEL of about RL 4.8m. To achieve this, an excavation up to approximately 8.0m depth will be required that will extend to about 1.8m (east), >4.5m (south), 1.4m (west) and >1.5m (north) of the boundary. A locally deeper excavation to about RL 3.6m will be required for the car lift.

No signs of existing, or potential natural instability or geotechnical issues were observed within the site or adjacent properties.

Prior to any demolition work, we recommend that detailed dilapidation surveys be carried out on all structures and infrastructures surrounding the site that fall within the zone of influence of the excavation considered to be 15.0m horizontal from the excavation perimeter to allow assessment of the adjacent structures and protect the client against spurious claims of damage.

Care will be required to ensure the existing neighbouring properties/structures are not destabilised during excavation especially where the excavation extends below the influence zone (1.0V:1.0H) of the existing footings of neighbour properties and they are not founded on stable medium-strength sandstone. Further geotechnical inspection is required at the initial site mark-out to assess all excavation perimeters and provide advice on any design/construction changes required to maintain stability.

Based on the investigation results, it is anticipated that the proposed bulk excavation will through all the units identified (topsoil, fill, natural soil and bedrock). Fill and natural soils, as well as very low strength bedrock, can be excavated using conventional earthmoving equipment such as buckets with light ripping, however low and greater strength bedrock will require the use of rock-breaking equipment (e.g., heavy ripper or rock hammers and rock saw).

Care will be required during the demolition, construction and excavation works where rock hammers are utilised to ensure the existing footings, structures and services are not adversely impacted by ground vibrations.

The use of rock hammers can create ground vibrations which could damage the neighbouring and adjacent structures even during demolition works. Care will be required during the demolition, construction, and excavation works to ensure the neighbouring properties, structures and services are not adversely impacted by ground vibrations. Small-scale equipment (<300kg rock hammer) along with a rock saw and a good excavation methodology are recommended to be used to maintain low vibration levels and avoid the need for full-time vibration monitoring, however, onsite calibration will be required to confirm this.

Based on the site investigation, groundwater seepage is anticipated within the proposed excavation; however, it is not expected to be a water table, and tanking and large-volume dewatering are unlikely to be critical hazards; however, this will require assessment through deeper investigation prior to design. Seepage collecting and disposal are required unless a tanked system is formed.

In view of the provided architectural plans, the local topography, anticipated excavation depth and the investigation results, temporary safe batter slopes are only achievable for the southern/rear bulk excavation boundary of the proposed basement. However, the stability of the bedrock in vertical stable batters within this location will require confirmation via a geotechnical inspection during the excavation. Preliminary design parameters for excavation batters and support systems are provided in **Section 6.3**. It must be ensured that no ponding occurs at the base of batters or surface flow on excavation of the face when they are implemented.

For the front and side excavation boundaries where the temporary batters are not feasible, a suitable retention system will be required for the support of soil to the bedrock interface, if unfractured sandstone bedrock of at least low strength can be identified. An additional geotechnical investigation in the form of rock coring to the depth of at least 3.0m below the BEL is required for the design and CC.

Due to the existence of sandy soil across the site, this may be best achieved on this site using contiguous/secant pile walls or similar to avoid the risk of sand collapse during the construction. Piles will need to be founded on unfractured sandstone bedrock of at least low strength or be installed below the excavation level within sandstone bedrock which is estimated to be of at least low strength. Preliminary design parameters are provided in **Section 6.3** as well. Additional geotechnical investigation in the form of cored boreholes is required to confirm the condition of bedrock below the existing investigation and proposed bulk excavation level.

Vertical cuts in medium to high-strength bedrock could be excavated without the need for pre-excavated support measures if it can be proved prior to final design in excavation. However, even where proven, a geotechnical inspection is required to assess for defects. Where defects are encountered, additional support may be required (i.e., rock bolts) to maintain stability. Permanent support (i.e., retaining wall) will be required where poor-quality bedrock exists. Geotechnical inspection will be required during any bedrock excavation at 1.50m depth intervals to assess if additional permanent support will be required (e.g., rock bolts/shotcrete).

Following the bulk excavation, the materials at the new basement floor are anticipated to be sandstone bedrock of at least low strength. The allowable bearing pressures appropriate for the conditions encountered underlying the site are provided in **Section 6.2.1**. For this site, shallow pad/strip footings founded on weathered bedrock at the base of the excavation may be suitable. It is recommended that all new footings for the proposed building be found within the same material of similar strengths to reduce the potential risk of differential settlement.

All footings must be inspected by an experienced geotechnical professional during construction and/or before concrete/steel is placed to verify the expected geology and depth for confirmation of assumed load capacity. Geotechnical inspections of foundations are required to determine or confirm it is achieved to meet the engineering design and to identify any potential variations between the boreholes.

Wong (2013) discussed excavation-induced ground movements within the Hawkesbury Sandstone as a result of excavation. That report and others have identified high north-south in situ horizontal stresses being the major in situ condition for the sandstone unit identified on site. It should be noted that movement from rock deformation in excavation shoring is not only noticeable in sand and clay excavations, but also found in rock excavations and cannot be prevented, regardless of shoring whilst movement in soils is also likely.

Measurements of deflection at the centre, crest of excavations within Sydney due to this high north south stress identified ratios of between 0.50mm and 2.00mm deflection per metre depth of excavation (D), with the deflection reducing to nil within 1.5 – 2.0D of the excavation perimeter.

The Acid Sulfate Soils Map of Manly Local Environmental Plan 2013 shows that the site is located within a Class 5 Acid Sulfate Soils zone. The investigation confirmed that the proposed work will not intersect acid sulfate soils and has very low potential to intersect or lower the water table. Therefore, the Preliminary Acid Sulfate Soil Assessment in line with the method of the Acid Sulfate Soil Manual (1998) indicates that an Acid Sulfate Soil Management Plan is not required. Confirmation of groundwater conditions as part of the additional geotechnical investigation for design will complete this assessment.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site provided the recommendations of this report are implemented in the design and construction phases. Some minor variations to the interpreted sub-surface conditions are possible, especially between test locations.

The recommendations and conclusions in this report are based on an investigation utilizing only surface observations and a limited number of test boreholes. 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.

6.2. 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 main hazards are:

A. Landslip (earth slide <3.0m³) from soils due to the bulk excavation of the new basement within the centre of the site.
B. Landslip (rock slide or topple, up to $10m^3$) due to instability in unsupported bedrock excavation;

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

The Risk to Life from **Hazard A** was estimated to be up to **5.63×10^{-5}** for any person in the existing pool or pool room of the site, while the Risk to Property was considered to be ‘**Moderate**’, which may be tolerated in certain circumstances but requires investigation, planning and implementation of treatment options to reduce the risk to Low.

Hazard B was estimated to have a Risk to Life of up to **4.00 x 10^{-4}** for a single person, while the Risk to Property was considered to be '**High**', , which is unacceptable without treatment. However, the assessments were based on excavations with no support or underpinning or planning.

Provided the recommendations of this report are implemented including installation of shoring wall prior to bulk excavation (or similar) the likelihood of any failure becomes 'Rare' and as such the consequences reduce and risk becomes within 'Acceptable' levels when assessed against the criteria of the AGS 2007. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

6.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

6.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' at the base of excavation into bedrock.
Type of Footing	Strip/Pad or Slab at the base of the basement excavation
Funded material and Maximum Allowable Bearing Capacity footings	Very Low Strength Sandstone: 800kPa Low Strength Bedrock: 1000kPa* Medium Strength Bedrock: 2000kPa*
Site sub-soil classification as per Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia	Class C_e – Shallow Soil The hazard factor (z) for Sydney is 0.08.
Remarks: * Due to the limits of the DCP test and hand auger, an additional inspection is required to determine the quality of sandstone bedrock as the founded material. All new footings must be inspected by an experienced geotechnical professional before concrete or steel is placed to verify the preliminary maximum bearing capacities provided above and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project. Individual structures should not be founded on materials with varying bearing and settlement characteristics unless the potential for differential movement has been allowed in the structural design.	

6.3.2. Excavation and retention:

<u>Proposed Excavation for the proposed two-level basement.</u>

Table 1: Basement excavation and structure separation distances

Boundary	Adjacent Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
			Boundary (m)	Structure
North	Footpath of Lauderdale Avenue	8.0m	⩾1.5	0
East	Main Dwelling at 3B Lauderdale Avenue.	Up to 8.0m	1.8	2.0
South	Footpath of Fairlight Beach	Up to 2.0.0m	⩾4.5	2.0
West	Main Dwelling at 7 Lauderdale Avenue	Up to 8.0m	1.4	2.5

Type of Material to be Excavated.	Fill/Residual Soil up to 2.4m
	VLS to LS and potential MS Sandstone bedrock VLS to LS siltstone interbeds, and occasional HS sandstone.

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

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

	Safe Batter Slope (H: V) *	
Material	**Short Term/Temporary**	**Long Term/Permanent**
Fill	1.5:1.0	2.0:1.0
Natural Sandy/Clayey Soil	1.5:1.0	2.0:1.0
Very low strength or fractured strength bedrock	0.25:1.0	0.5:1.0
At least Low Strength unfractured Sandstone bedrock*	Vertical Cut	Vertical Cut

*Dependent on defects and assessment by engineering geologist or geotechnical engineer.

Remarks:

Potential 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	Fill and residual soil	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-MS-HS bedrock	Rock grinder, rock saw and rock hammer

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Adjacent existing structure = 5mm/s to maintain human comfort and provide very low potential for structure damage. SW sewer: subject to the assessment of sewer condition
Vibration Calibration Tests Required	Where rock hammer ≥300 kg is proposed for use.
Full-time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results. Anticipate large equipment; therefore, recommended.
Dilapidation Surveys Requirement	Required on the neighbouring structures/infrastructures or parts within 15m of the excavation perimeter.

Remarks:

Based on previous testing of ground vibrations created by various rock excavation equipment within medium-strength bedrock, to maintain a vibration level below 5mm/s PPV the below 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 to the site-specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Calibration of rock excavation machinery should be carried out prior to the commencement of rock excavation works where ≥300kg rock hammers are proposed for use.

Rock sawing of the excavation perimeter is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal, the saw cuts will provide a slight increase in buffer distance for use of rock hammers whilst also reducing deflection of separated rock across boundaries.

The strength of bedrock below the maximum depth achieved during the investigation is unconfirmed and would require cored boreholes using specialist restricted access drilling equipment.

Excavation of soils to ELS will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.

6.3.3. Retaining Structures:

Required	New retaining structures will be required as part of the proposed development to support the excavation perimeters;
Types	Contiguous pile wall through soils to bedrock surface or to the base of excavation. Steel reinforced concrete/concrete block wall designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.

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

Material	Unit Weight (kN/m3)	Cohesion c (kPa) undrained	Friction angle ϕ'*	Earth Pressure Coefficients			Young's Modulus E (MPa)
				At Rest (K_0)	Active (Ka)	Passive (Kp)	
Fill and natural sand (Very loose to loose)	18	20	22°	0.63	0.45	2.2	-
Sandy Clay – Firm to Stiff	20	50	25°	0.58	0.41	2.46	20
Clayey Sand – Medium Dense to Very Dense	21	0	32°	0.47	0.31	3.25	25
VLS or fractured Sandstone Bedrock	23	100	35°	0.5	0.33	3.0	200

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.

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).

In such cases, a triangular pressure distribution can be assumed for the design of these supports, where the pressure increases linearly with depth, providing a more realistic and efficient representation of the actual earth pressure exerted on the retaining structure.

6.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No groundwater table or significant water seepage was observed during the investigation.
Excavation likely to intersect	Water Table	Not expected
	Seepage	Minor (<2.0L/min) possible at fill/bedrock interface and along the bedrock defects
Site Location and Topography		Low, south side of Lauderdale Avenue within gently to moderately south dipping topography.
Impact of development on local hydrogeology		Negligible impact expected, minor reduction in groundwater seepage in adjacent land due to creation of new low point in topography. Potential controllable via a sump pump system
Onsite Stormwater Disposal		Only possible via dispersion for a portion of stormwater at the rear boundary. Not recommended.

6.4 Conditions Relating to Design and Construction Monitoring:

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

1. A further geotechnical investigation to confirm the bedrock condition to below the BEL;
2. Review structural design drawings for implementation of the recommendations of this report and to confirm inspection/testing required to maintain site stability.
3. Inspect the site at the initial site mark-out to re-assess excavation perimeters and depths
4. Inspect installation of pre-excavation support systems and all excavation in rock and batter slopes in soils at 1.50 – 2.00m depth intervals.
5. Inspect all new footings to confirm compliance with design assumptions with respect to allowable bearing pressure and stability prior to the placement of steel or concrete.
6. Where ground conditions vary from those anticipated and outlined in this report are encountered.

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

7. CONCLUSION:

The site investigation identified the presence of varying topsoil/fill of shallow thickness (up to 2.4m) over the project site, overlying sandy/clay soil encountered within BH1, BH3 and BH4 to the investigated depth up to about 2.4m below the existing ground surface. Sandstone bedrock was observed/interpreted during the investigation from a depth varying from about 0.9 (RL 9.3m in BH3) to about 2.4m (RL 10.4m in BH5) below the existing ground surface. No groundwater table was identified during the investigation.

Minor seepage is expected at the soil-bedrock interface and along the defects in the sandstone and is expected within the anticipated depth of the proposed excavation based on the site location and topography. Seepage collecting and disposal are required unless a tanked system is formed. A groundwater table is not anticipated.

The proposed works involve the demolition of existing site structures and the construction of a Class 2, four-storey unit building with a basement level, which requires a BEL of about RL 4.8m and therefore up to 8.0m depth excavation. A locally deeper excavation to about RL 3.6m will be required for the car lift.

Care will be required to ensure the existing neighbouring properties/structures are not destabilised during excavation if the excavation extends below the influence zone (1.0V:1.0H) of the existing footings and they are not founded on stable medium-strength sandstone. A further geotechnical investigation is required prior to final design and CC whilst an inspection is required at the initial site mark-out to assess all excavation perimeters and provide advice on any design/construction changes required to maintain stability. However, the risk analysis indicates the potential for instability beyond property boundaries is considered to be ‘Unlikely’, as per Appendix C of the AGS 2007 guidelines.

Based on the investigation results, it is anticipated that the proposed bulk excavation will through all the units above (topsoil, fill, natural soil and bedrock) with bedrock potential up to high strength encountered.

The use of rock hammers can create ground vibrations which could damage the neighbouring and adjacent structures even during demolition works. Care will be required during the demolition, construction, and excavation works to ensure the neighbouring properties, structures and services are not adversely impacted by ground vibrations.

Temporary safe batter slopes are only achievable for the southern/rear bulk excavation boundary of the proposed basement. However, the stability of the bedrock in vertical stable batters will require confirmation via a geotechnical investigation prior to CC and the inspection during the excavation to reduce the risk of instability.

For the front and side excavation boundaries, a suitable retention system in the form of contiguous pile walls or similar will be recommended to avoid the risk of sand collapse during the construction. They should be installed on the interface of unfractured bedrock of at least low strength, or the entire depth of the excavation where the bedrock is estimated to be at least low strength.

Additional geotechnical investigation in the form of cored boreholes is required to confirm the condition of bedrock below the existing investigation and proposed bulk excavation level. This is recommended to be completed prior to the final design and CC application.

Following the bulk excavation, shallow pad/strip footings founded on weathered bedrock at the base of the excavation are suitable. It is recommended that all new footings for the proposed building be found within the same material of similar strengths to reduce the potential risk of differential settlement. Geotechnical inspections of foundations are required to determine or confirm the required bearing capacity is achieved to meet the engineering design and to identify any potential variations between the boreholes.

Based on Wong (2013), it should be noted that movement from rock deformation in excavation shoring is not only noticeable in sand and clay excavations but also found in rock excavations, cannot be prevented, regardless of shoring whilst movement in soils is also likely.

Based on Manly Local Environmental Plan 2013 and the investigation results, Preliminary Acid Sulfate Soil Assessment in line with the method of the Acid Sulfate Soil Manual (1998) indicates that an Acid Sulfate Soil Management Plan is not required.

The risks associated with the proposed development can be maintained within 'Acceptable Risk Management Criteria' of the Councils policy with negligible impact to neighbouring properties or site 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 provided in this report are followed.

Prepared by:

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Geotechnical Engineer

Reviewed by:

Troy Crozier
Principal
MIE Aust. CPEng
MAIG, RPGeo – Geotechnical and Engineering

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Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

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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: -

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

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

Qc = (12 to 18) Cu

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

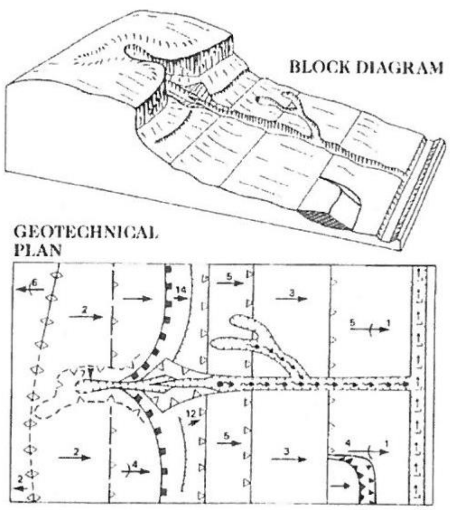
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

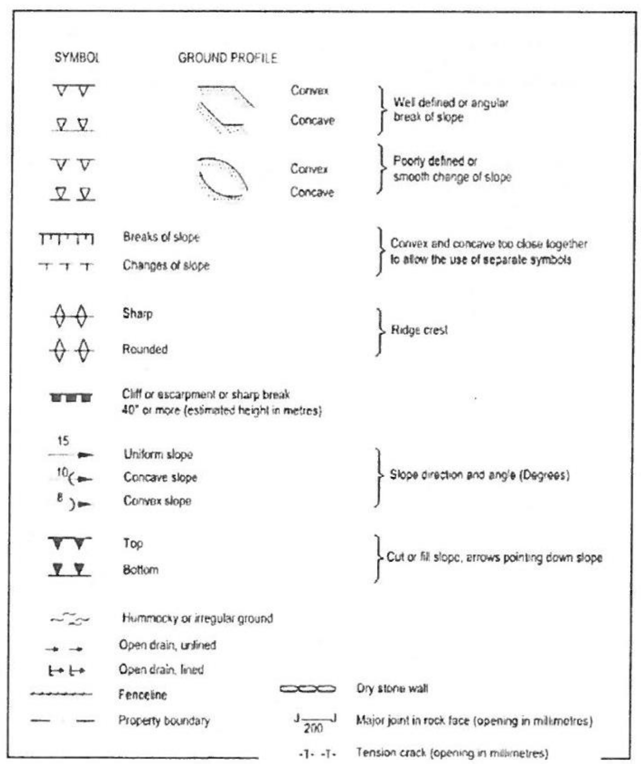
Reproduction of Information for Contractual Purposes

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

Site Inspection

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

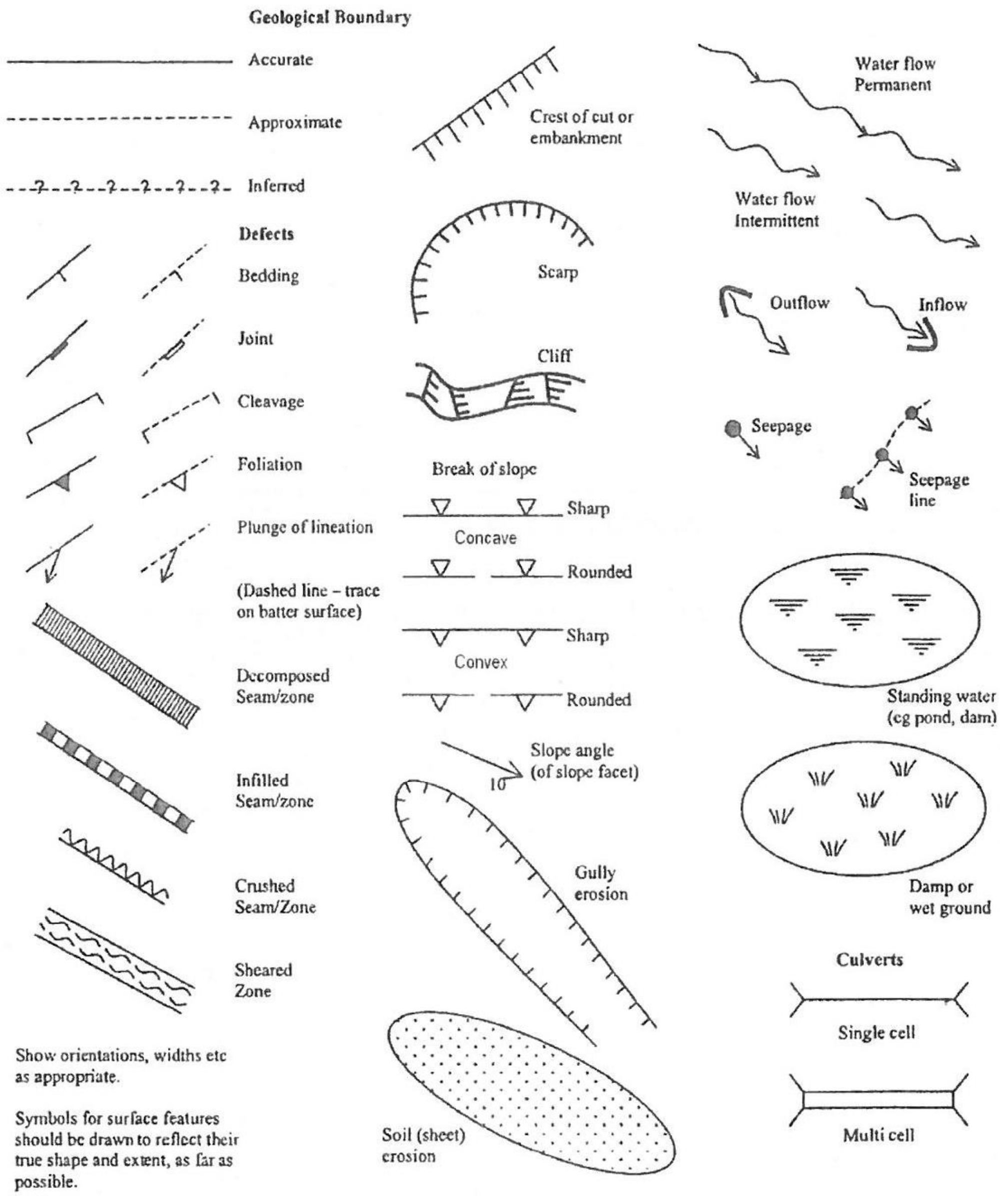




Example of Mapping Symbols

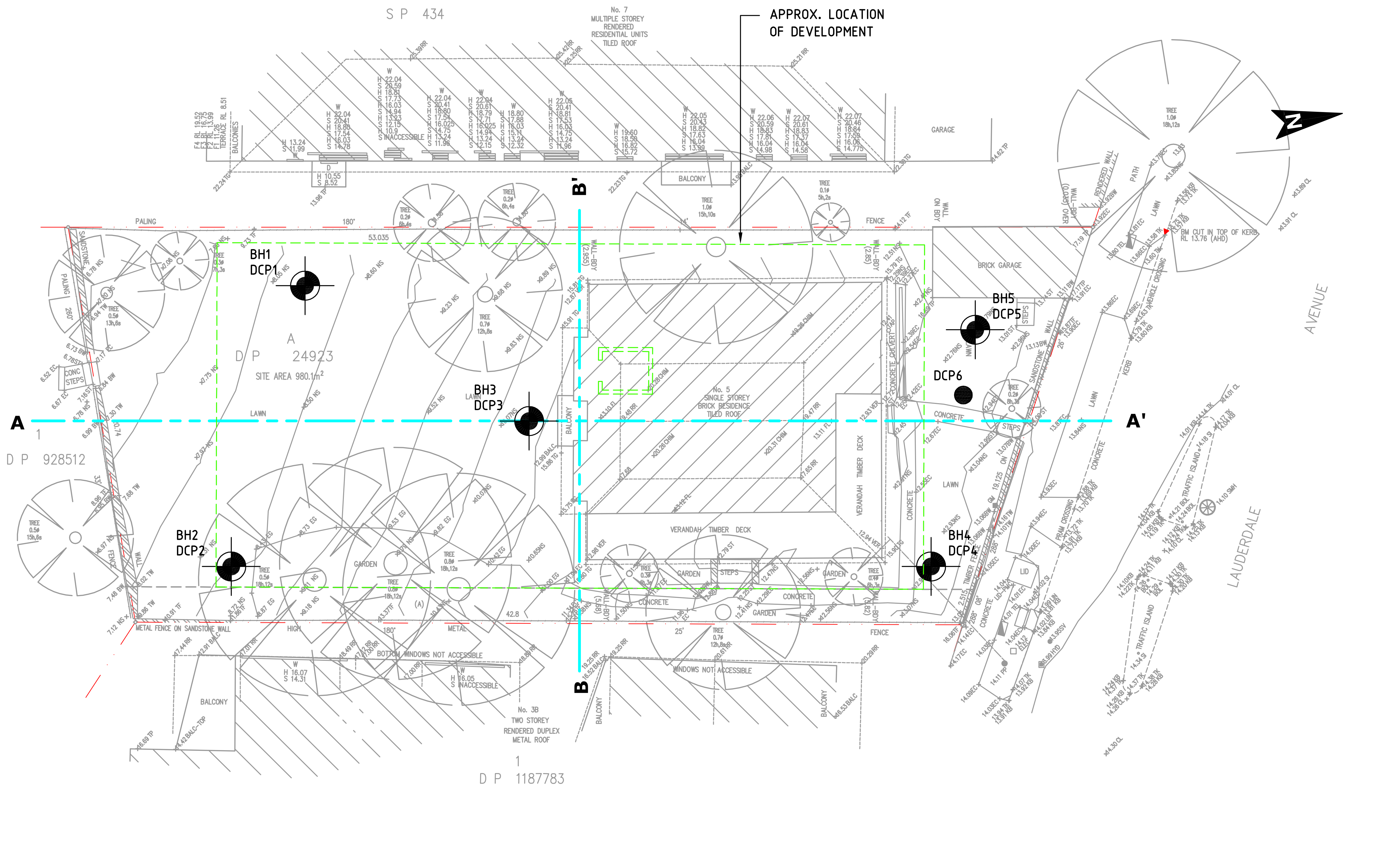
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



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

SITE PLAN AND TEST LOCATIONS FIGURE 1

CROZIER
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Crozier Geotechnical
Unit 12, 42-46 Wattle Road
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Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

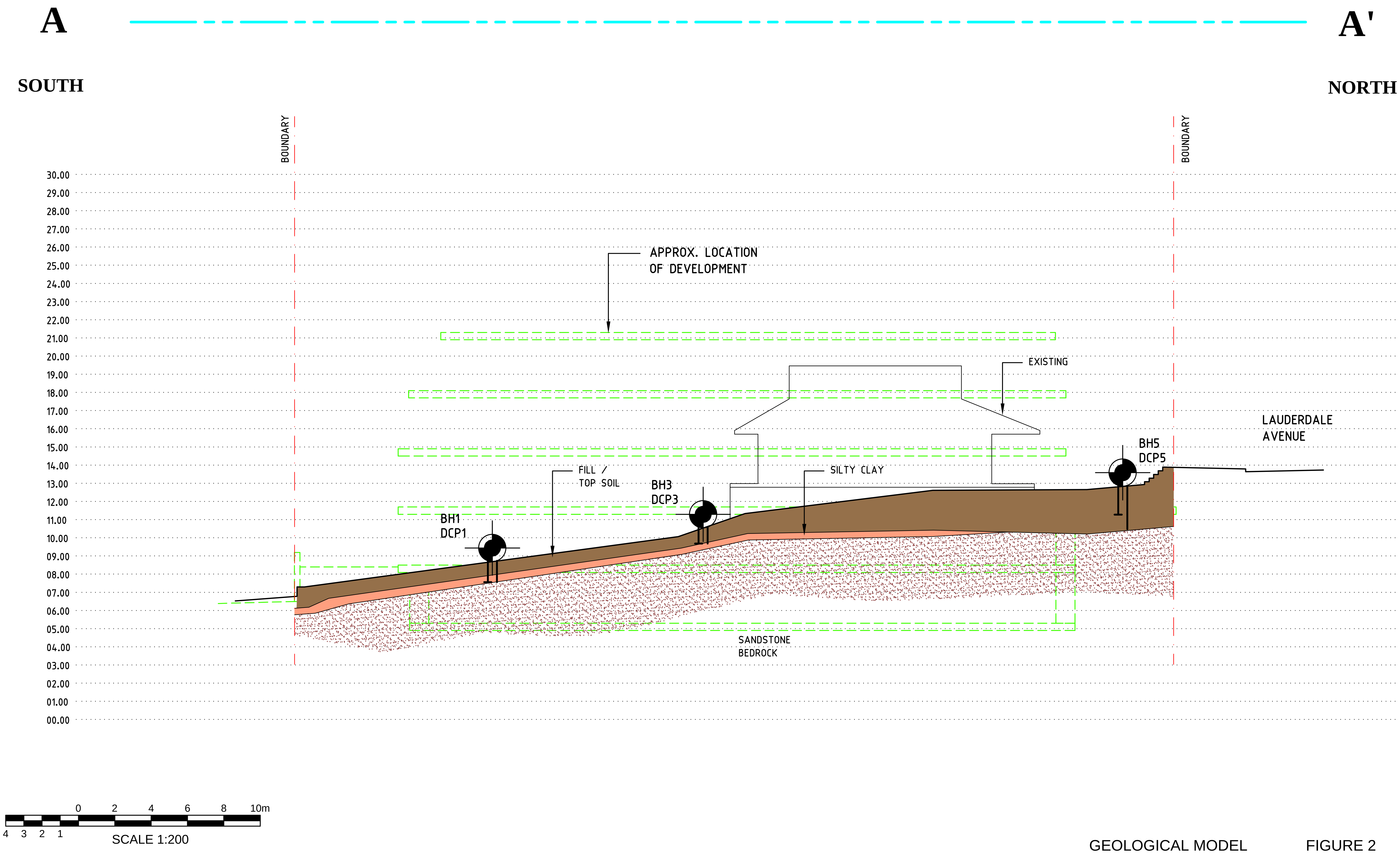
LEGEND

- BH- AUGER LOCATION
- TP- TEST PIT LOCATION
- DCP- DYNAMIC PENETRATION TEST
- CBR- CALIFORNIA BEARING RATIO LOCATION
- A — A' CROSS SECTION REFERENCE LINE
- BOUNDARY LINE
- x° SLOPE ANGLE

SCALE :	1:200
DRAWING :	FIGURE 1
DATE :	24.06.24
APPROVED BY :	J.L.
DRAWN BY :	A.C.W.
PROJECT :	2024-109

PREPARED FOR :
HPG PROJECT LAUDERDALE Pty Ltd

ADDRESS :
5 LAUDERDALE AVENUE
FAIRLIGHT N.S.W.



GEOLOGICAL MODEL

FIGURE 2

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
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Brookvale NSW 2100

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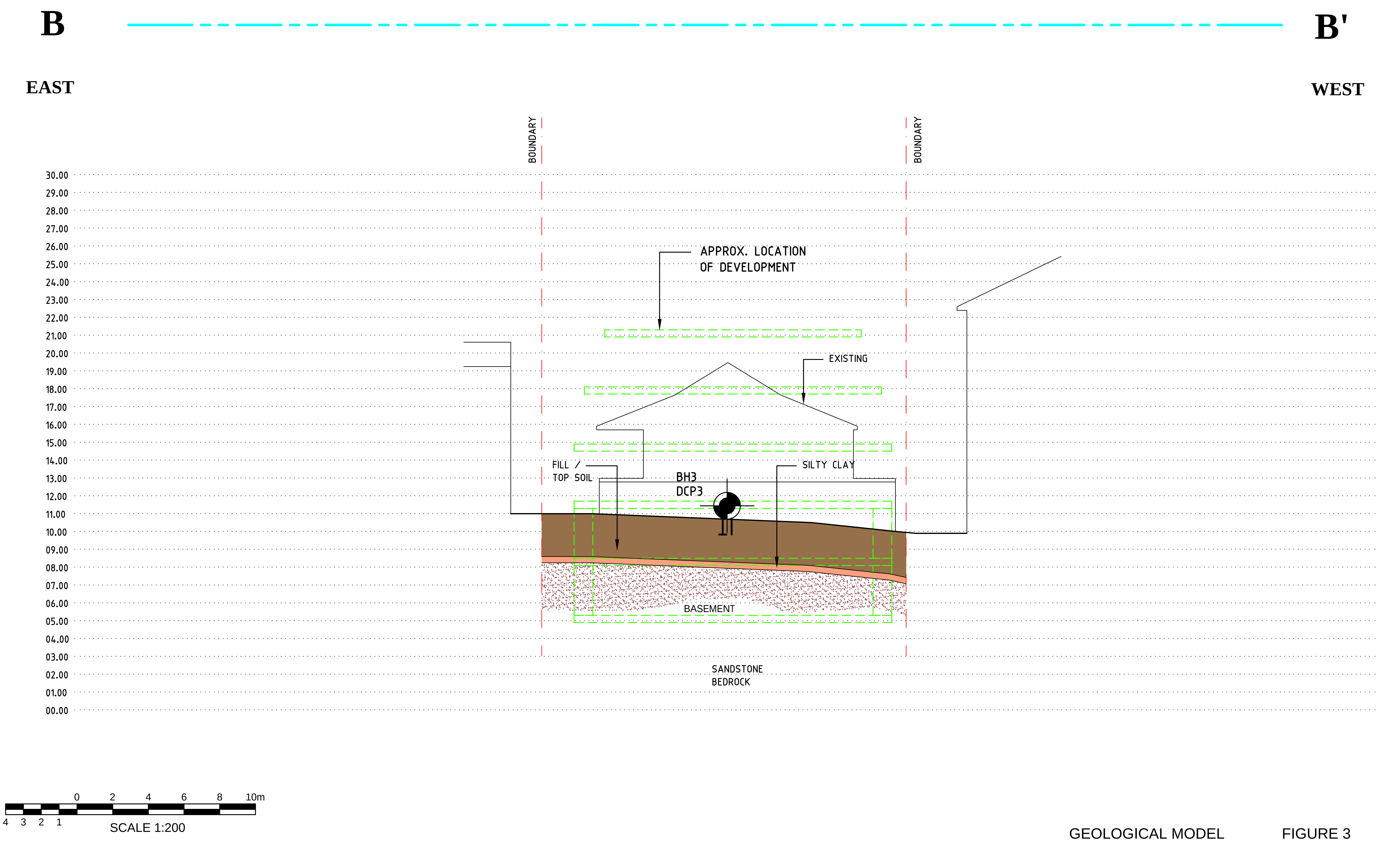
LEGEND

- AUGER LOCATION
- AUGER / DYNAMIC PENETRATION TEST
- DCP - DYNAMIC PENETRATION TEST
- BOUNDARY LINE
- BROWN SANDY TOPSOIL
- SANDSTONE BEDROCK
- SILTY CLAY

SCALE :	1:200
DRAWING :	FIGURE 2
DATE :	24.06.24
APPROVED BY :	J.L.
DRAWN BY :	A.C.W.
PROJECT :	2024-109

PREPARED FOR :
HPG PROJECT LAUDERDALE Pty Ltd

ADDRESS :
5 LAUDERDALE AVENUE
FAIRLIGHT N.S.W.



GEOLOGICAL MODEL

FIGURE 3

LEGEND

AUGER LOCATION

DYNAMIC PENETRATION TEST

BROWN SANDY TOPSOIL

SILTY CLAY

AUGER / DYNAMIC PENETRATION TEST

BOUNDARY LINE

SANDSTONE BEDROCK

CROZIER
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SCALE :	1:200
DRAWING :	FIGURE 3
DATE :	24.06.24
APPROVED BY :	J.L.
DRAWN BY :	A.C.W.
PROJECT :	2024-109

PREPARED FOR :
HPG PROJECT LAUDERDALE Pty Ltd

ADDRESS :
5 LAUDERDALE AVENUE
FAIRLIGHT N.S.W.

CROZIER GEOTECHNICAL CONSULTANTS

Borehole No.
BH1
Page 1 of 1

Engineering Log - Borehole

Project No.: 2024-109

Client:	HPG Project Lauderdale Pty Ltd	Commenced:	20/6/2024
Project Name:	Proposed Class 2 Residential Building	Completed:	20/6/2024
Hole Location:	Refer to Figure 1	Logged By:	JL
Hole Position:		Checked By:	TMC

Drill Model and Mounting:	-	Inclination:	-90°	RL Surface:	8.20 m		
Hole Diameter:	100 mm	Bearing:		Datum:	AHD	Operator:	AC

Drilling Information								Soil Description						Observations
Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Group Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional	Moisture Condition	Consistency Relative Density	Pocket Penetrometer UCS (kPa) 100 200 300 400 500	Structure and Additional Observations
HA			Groundwater not encountered						-	TOPSOIL: Silty SAND, fine grained, dark brown, trace rootlets and clay.	M	VL		0.00-0.20: TOPSOIL
									-	FILL: Silty SAND, fine to medium grained, dark brown, trace rootlets and gravels.	M	L		0.20-0.65: FILL
						7.7	0.5							
				D 0.70-0.80 m					CL-CI	Sandy CLAY: low to medium plasticity, brown grey mottled pale orange, trace sub-angular to sub rounded gravels.	w<PL	F		0.65-0.95: RESIDUAL SOIL
				D 0.95-1.05 m		7.2	1.0		SC-SM	Clayey Silty SAND: fine to medium grained, pale grey and yellow brown, traced sub-angular sandstone gravels, grading into extremely weathered sanstone.	M	VD		0.95-1.05: RESIDUAL SOIL
										Hole Terminated at 1.05 m Hand Auger Refusal on Weathered Bedrock				
						6.7	1.5							
						6.2	2.0							

Method
AS - Auger Screwing
ADV - Auger V Bit
ADT - Auger Tungsten Carbide Bit
RR - Rock Roller
WB - Washbore

Support
C - Casing

Penetration
No resistance ranging to refusal

Water
Level (Date)
Inflow
Partial Loss
Complete Loss

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
PP - Pocket Penetrometer

Classification Symbols and Soil Descriptions
Based on Unified Soil Classification System

Moisture Condition
D - Dry
M - Moist
W - Wet
w - Moisture Content
PL - Plastic Limit
LL - Liquid Limit

Consistency/Relative Density
VS - Very Soft
S - Soft
F - Firm
VSt - Very Stiff
H - Hard
Fr - Friable
VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

CROZIER GEOTECHNICAL CONSULTANTS

Engineering Log - Borehole

Project No.: 2024-109

Client:	HPG Project Lauderdale Pty Ltd	Commenced:	20/6/2024
Project Name:	Proposed Class 2 Residential Building	Completed:	20/6/2024
Hole Location:	Refer to Figure 1	Logged By:	JL
Hole Position:		Checked By:	TMC

Drill Model and Mounting:	-	Inclination:	-90°	RL Surface:	8.50 m		
Hole Diameter:	100 mm	Bearing:		Datum:	AHD	Operator:	AC

Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Group Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional	Moisture Condition	Consistency Relative Density	Pocket Penetrometer UCS (kPa) 100 200 300 400 500	Structure and Additional Observations
HA			Groundwater not encountered						-	TOPSOIL: Silty SAND, fine grained, dark brown, trace rootlets and clay.	M	VL		0.00-0.20: TOPSOIL
						8.0	0.5		-	FILL: Silty SAND, fine to medium grained, dark brown, trace galsses and gravels. From 0.8m, trace sandstone gravels.	M	L		0.20-0.90: FILL
						7.5	1.0			Hole Terminated at 0.90 m Hand Auger Refusal on Weathered Bedrock				
						7.0	1.5							
						6.5	2.0							

Method
AS - Auger Screwing
ADV - Auger V Bit
ADT - Auger Tungsten Carbide Bit
RR - Rock Roller
WB - Washbore

Support
C - Casing

Penetration
No resistance ranging to refusal

Water
Level (Date)
Inflow
Partial Loss
Complete Loss

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
PP - Pocket Penetrometer

Classification Symbols and Soil Descriptions
Based on Unified Soil Classification System

Moisture Condition
D - Dry
M - Moist
W - Wet
w - Moisture Content
PL - Plastic Limit
LL - Liquid Limit

Consistency/Relative Density
VS - Very Soft
S - Soft
F - Firm
VSt - Very Stiff
H - Hard
Fr - Friable
VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

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Borehole No.
BH3
Page 1 of 1

Engineering Log - Borehole

Project No.: 2024-109

Client:	HPG Project Lauderdale Pty Ltd	Commenced:	20/6/2024
Project Name:	Proposed Class 2 Residential Building	Completed:	20/6/2024
Hole Location:	Refer to Figure 1	Logged By:	JL
Hole Position:		Checked By:	TMC

Drill Model and Mounting:	-	Inclination:	-90°	RL Surface:	10.20 m		
Hole Diameter:	100 mm	Bearing:		Datum:	AHD	Operator:	AC

Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Group Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional	Moisture Condition	Consistency Relative Density	Pocket Penetrometer UCS (kPa) 100 200 300 400 500	Structure and Additional Observations
HA			Groundwater not encountered						-	TOPSOIL: Silty SAND, fine grained, dark brown, trace gravels and rootlets.	M	VL		0.00-0.20: TOPSOIL
						9.7	0.5		-	FILL: Silty SAND, fine to medium grained, brown grey, trace sandstone gravels and rootlets.	M	VL		0.20-0.60: FILL
									SM	Silty SAND: fine grained, grey, trace rootlets and clay.				0.60-0.80: RESIDUAL SOIL
				D 0.80-0.90 m					CL	Sandy CLAY: low plasticity, pale grey, mottled yellow brown, trace silt.	w<PL	St		0.80-0.90: RESIDUAL SOIL
						9.2	1.0			Hole Terminated at 0.90 m Hand Auger Refusal on Weathered Bedrock				
						8.7	1.5							
						8.2	2.0							

Method
AS - Auger Screwing
ADV - Auger V Bit
ADT - Auger Tungsten Carbide Bit
RR - Rock Roller
WB - Washbore

Support
C - Casing

Penetration
No resistance ranging to refusal

Water
Level (Date)
Inflow
Partial Loss
Complete Loss

Samples and Tests
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Fr - Friable
VL - Very Loose
L - Loose
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Engineering Log - Borehole

Project No.: 2024-109

Client:	HPG Project Lauderdale Pty Ltd	Commenced:	20/6/2024
Project Name:	Proposed Class 2 Residential Building	Completed:	20/6/2024
Hole Location:	Refer to Figure 1	Logged By:	JL
Hole Position:		Checked By:	TMC

Drill Model and Mounting: | Inclination: -90° | RL Surface: 12.90 m
Hole Diameter: 100 mm | Bearing: | Datum: AHD | Operator: AC

Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Group Symbol	Material Description (Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional)	Moisture Condition	Consistency Relative Density	Pocket Penetrometer UCS (kPa)	Structure and Additional Observations
HA			Groundwater not encountered				0.00–0.20	-	TOPSOIL: Silty SAND, fine grained, dark brown, trace gravels and rootlets.	M	VL		0.00-0.20: TOPSOIL
						12.4	0.20–0.70 (0.5)	-	FILL: Silty SAND, fine to medium grained, brown grey, trace sandstone gravels and rootlets.	M	L		0.20-0.70: FILL
				D 0.80-0.90 m			0.70–1.00	SM	Silty SAND: fine grained, yellow brown, trace clay. From 0.8m, trace sandstone gravels.	M	L - D		0.70-1.00: RESIDUAL SOIL
						11.9	1.0		Hole Terminated at 1.00 m Hand Auger Refusal on Weathered Bedrock				
						11.4	1.5						
						10.9	2.0						

Method
AS - Auger Screwing
ADV - Auger V Bit
ADT - Auger Tungsten Carbide Bit
RR - Rock Roller
WB - Washbore

Support
C - Casing

Penetration
No resistance ranging to refusal

Water
Level (Date)
Inflow
Partial Loss
Complete Loss

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
SPT - Standard Penetration Test
PP - Pocket Penetrometer

Classification Symbols and Soil Descriptions
Based on Unified Soil Classification System

Moisture Condition
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Consistency/Relative Density
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Fr - Friable
VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

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Engineering Log - Borehole

Project No.: 2024-109

Client:	HPG Project Lauderdale Pty Ltd	Commenced:	20/6/2024
Project Name:	Proposed Class 2 Residential Building	Completed:	20/6/2024
Hole Location:	Refer to Figure 1	Logged By:	JL
Hole Position:		Checked By:	TMC

Drill Model and Mounting:		Inclination:	-90°	RL Surface:	12.80 m		
Hole Diameter:	100 mm	Bearing:		Datum:	AHD	Operator:	AC

Method	Penetration	Support	Water	Samples Tests Remarks	Recovery	RL (m)	Depth (m)	Graphic Log	Group Symbol	Material Description Fraction, Colour, Structure, Bedding, Plasticity, Sensitivity, Additional	Moisture Condition	Consistency Relative Density	Pocket Penetrometer UCS (kPa) 100 200 300 400 500	Structure and Additional Observations
HA			Groundwater not encountered						-	TOPSOIL: Silty SAND, fine grained, dark brown, trace rootlets and clay.	M	VL		0.00-0.20: TOPSOIL
						12.3	0.5		-	FILL: Silty SAND, fine to medium grained, dark brown, trace galsses and gravels.	M	VL - L		0.20-1.60: FILL
						11.8	1.0			From 0.5m, trace sandstone gravels.		MD - VD		
						11.3	1.5							
						10.8	2.0			Hole Terminated at 1.60 m Hand Auger Refusal on cobbles.				

Method
AS - Auger Screwing
ADV - Auger V Bit
ADT - Auger Tungsten Carbide Bit
RR - Rock Roller
WB - Washbore

Support
C - Casing

Penetration
No resistance ranging to refusal

Water
Level (Date)
Inflow
Partial Loss
Complete Loss

Samples and Tests
U - Undisturbed Sample
D - Disturbed Sample
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Classification Symbols and Soil Descriptions
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S - Soft
F - Firm
VSt - Very Stiff
H - Hard
Fr - Friable
VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

DYNAMIC PENETROMETER TEST SHEET

CLIENT: HPG Project Lauderdale Pty Ltd | **DATE:** 20/06/204
PROJECT: Proposed New Class 2 Residential Building | **PROJECT No.:** 2024-109
LOCATION: 5 Lauderdale Avenue, Fairlight, NSW | **SHEET:** 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP5A	DCP5B	DCP5C	DCP5D	DCP6	
0.00 - 0.10	-	1	-	1	-	-	-	-	1	
0.10 - 0.20	-	0	-	0	1	1	-	-	1	
0.20 - 0.30	1	2	-	1	0	0	-	-	0	
0.30 - 0.40	0	2	-	1	1	1	-	-	2	
0.40 - 0.50	1	3	1	3	2	2	-	-	0	
0.50 - 0.60	2	1	1	1	3	HR	-	-	2	
0.60 - 0.70	1	1	3	2	HR		-	-	0	
0.70 - 0.80	3	1	2	1			20	-	4	
0.80 - 0.90	4	7	6(B)	4			6	-	10	
0.90 - 1.00	19 (B)	10 (B)	at 0.88m	8 (B)			8	-	26 (B)	
1.00 - 1.10	at 1.0m	at 0.92m		at 1.0m			10	-	at 1.0m	
1.10 - 1.20							9	-		
1.20 - 1.30							11	-		
1.30 - 1.40							12	-		
1.40 - 1.50							8	-		
1.50 - 1.60							15	-		
1.60 - 1.70							14	1		
1.70 - 1.80							6	1		
1.80 - 1.90							6	1		
1.90 - 2.00							8	0		
2.00 - 2.10							7	1		
2.10 - 2.20							7	1		
2.20 - 2.30							5	1		
2.30 - 2.40							12 (B)	5 (B)		
2.40 - 2.50							at 2.40m	at 2.40m		
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

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

REMARKS:
- **--** No test undertaken at this level due to prior excavation of soils
- **(B)** Test hammer bouncing upon refusal on solid object
- **(HR)** Hard Refusal on bedrock or hard material with no more penetration
- **(PR)** Pratical Refusal: continuous 3 times with ≥15 Blows/100mm or single time with ≥25 Blows

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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 (earth slide <3.0m³) from soils due to the bulk excavation of the new basement within the centre of the site		Excavation up to 8.0m depth through fill, natural soils and bedrock within the centre of the site	a) Excavation directly beneath the dwelling, impact 30% b) Main building at about 2.0m to the excavation, impact 10% c) Building at about 2.5m to the excavation, impact 10% d) Footpath adjacent to the excavation, impact 20%		a) Person in the building 20hr/day average; b) Person in the dwelling 20hr/day average; c) Person in the dwelling 20hr/day average d) Person on the footpath 0.1hr/day average;	a) Possible to not evacuate b) Likely to not evacuate c) Likely to not evacuate d)Possible to not evacuate	a) Person in the building, damage only b) Person in the building, damage only d) Person in the building, damage only e) Person in open space, impact only	
			Likely	**Prob. of Impact**	**Impacted**				
		a) The new residential building of the site at 5 Lauderdale Avenue	0.01	0.90	0.30	0.8333	0.50	0.05	**5.63E-05**
		b) Main Dwelling at 3B Lauderdale Avenue.	0.01	0.15	0.10	0.8333	0.75	0.05	**4.69E-06**
		c) Main Dwelling at 7 Lauderdale Avenue.	0.01	0.15	0.10	0.8333	0.75	0.05	**4.69E-06**
		d) Footpath along southern side of Lauderdale Avenue.	0.01	0.15	0.20	0.0417	0.50	0.10	**6.25E-07**
b	Landslip (Rock slide/topple <10.0m³) due to instability in unsupported bedrock excavation		Excavation up to 8.0m depth with bedrock excavation of up to about 7.0m	a) Excavation directly beneath the dwelling, impact 80% b) Main building at about 2.0m to the excavation, impact 30% c) Building at about 2.5m to the excavation, impact 20% d) Footpath adjacent to the excavation, impact 50%		a) Person in the building 20hr/day average; b) Person in the dwelling 20hr/day average; c) Person in the dwelling 20hr/day average d) Person on the footpath 0.1hr/day average;	a) Likely to not evacuate b) Likely to not evacuate c) Likely to not evacuate d)Unlikely to not evacuate	a) Person in the building, building is inundated with debris and the person buried b) Person in the building, damage only d) Person in the building, damage only e) Person in open space, impact only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) The new residential building of the site at 5 Lauderdale Avenue	0.001	0.80	0.80	0.8333	0.75	1.00	**4.00E-04**
		b) Main Dwelling at 3B Lauderdale Avenue.	0.001	0.20	0.30	0.8333	0.75	0.05	**1.88E-06**
		c) Main Dwelling at 7 Lauderdale Avenue.	0.001	0.20	0.20	0.8333	0.75	0.05	**1.25E-06**
		d) Footpath along southern side of Lauderdale Avenue.	0.001	0.20	0.50	0.0417	0.25	0.10	**1.04E-07**

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <3.0m³) from soils due to the bulk excavation of the new basement within the centre of the site	a) The new residential building of the site at 5 Lauderdale Avenue	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Moderate damage to some of structure or significant part of site, requires large stabilising works **or** MINOR damage to neighbouring property.	Moderate
		b) Main Dwelling at 3B Lauderdale Avenue.	Likely	Event will probably occur under adverse 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.	Moderate
		c) Main Dwelling at 7 Lauderdale Avenue.	Likely	Event will probably occur under adverse circumstances over the design life.	Medium	Limited Damage to part of structure or site requires some stabilisation **or** INSIGNIFICANT damage to neighbouring properties.	Moderate
		d) Footpath along southern side of Lauderdale Avenue.	Likely	Event will probably occur under adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required **or** no impact to neighbouring properties.	Very Low
B	Landslip (Rock slide/topple <10.0m³) due to instability in unsupported bedrock excavation	a) The new residential building of the site at 5 Lauderdale Avenue	Possible	The event could occur under adverse conditions over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site **or** MEDIUM damage to neighbouring properties.	High
		b) Main Dwelling at 3B Lauderdale 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.	Moderate
		c) Main Dwelling at 7 Lauderdale Avenue.	Possible	The event could occur under adverse conditions over the design life.	Medium	Limited Damage to part of structure or site requires some stabilisation **or** INSIGNIFICANT damage to neighbouring properties.	Moderate
		d) Footpath along southern side of Lauderdale 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.	Moderate

* 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%.
* Cost of site development estimated at $5,000,000

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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.

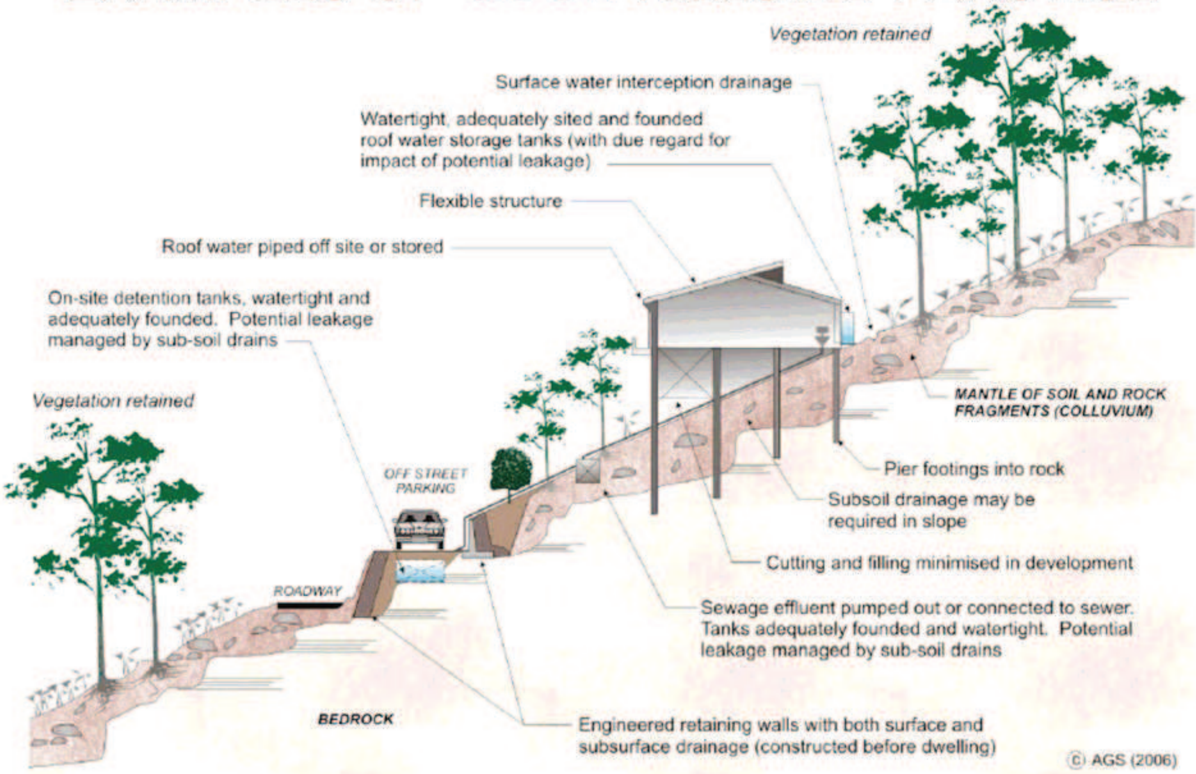
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

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