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GEOTECHNICAL REPORT

for

PROPOSED ALTERATIONS AND ADDITIONS TO EXISTING BUILDING

at

19 DARLING POINT ROAD, DARLING POINT, NSW

Prepared For

Ascham School

Project No.: 2022-273

December, 2023

Document Revision Record

Issue No	Date	Details of Revisions
0	14th December 2022	Original issue
1	12th December 2023	Revisions to final

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APPENDICES

CROZIER
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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS TO EXISTING BUILDING AT 19 DARLING POINT ROAD, DARLING POINT, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions to an existing building at 19 Darling Point Road, Darling Point, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Tribe Studio Architects Pty Ltd on behalf of the client Ascham School.

The site contains an existing four-storey residential flat building located centrally within the lot, together with a detached single storey brick building (laundry) located near the south-eastern corner. Three garages are located along the Darling Point Road frontage.

It is understood that the proposed works comprise internal modifications, the extension of the rear of the building including the construction of a new lift and landscaping works at the rear of the site and construction of a new upper level over the extended structure. It is understood that bulk excavation will be required down to approximately ≤2.50m depth for the proposed extension (≤2.50m from the side boundaries) and down to approximately ≤4.0m depth for the proposed new lift (≤10m from the side boundaries). Proposed works will also include an approximately 4.0m depth excavation for an OSD tank to the front of the site near existing garages.

Woollahra Councils 'Guidelines for Preparation of Geotechnical and Hydrogeological Reports' Annexure 3, September 2002 outlines the requirements for Development Application reporting where excavation to > 1.0m depth is proposed. To provide the report for Council an investigation is required that will include assessment of sub-surface conditions, soil strengths, and groundwater levels.

The investigation and reporting were undertaken as per the Proposal P22-563, Dated: 31st October 2022.

The investigation comprised:

a) Detailed geotechnical mapping of the site and adjacent properties including photographic record of site conditions by an experienced Geotechnical Engineer.

b) DBYD and onsite clearing of test locations by a registered service location contractor.
c) Concrete coring at one location to drill a borehole (BH3) and remediation.
d) Drilling of five boreholes using hand tools along with Dynamic Penetrometer (DCP) tests to investigate the subsurface geology including soil conditions/parameters.

The following plans and diagrams were supplied by the Architect and relied upon for fee proposal preparation, assessment and reporting.

- Architectural Drawings – Tribe Studio Architects, Project No.: 1007, Drawing No.: DA-00-001 – DA-02-000, DA-02-001, DA-06-008 - DA-06-011, DA-06-012, DA-06-013, DA-06-020 – DA-10-002, DA-11-008, DA-11-011 – DA-11-012, DA-11-013, DA-21-010 , DA-21-012, DA-31-001, DA-31-002 - DA-31-003, DA-82-000, DA-82-002, DA-82-003 – DA-84-001, DA-95-001, Dated: 15/11/2023.
- Survey Drawing – Rygate Surveyors, Ref No.: 76389, Sheet 1 - 4 of 4, Dated: 04/06/2014 and 11/3/2022.

2. PROPOSED WORK:

It is understood that the proposed works comprise the rear extension of the existing four storey building to the east. The works also includes the construction of a new lift within the building and landscaping works at the rear of the site.

It is understood that bulk excavation down to ≤2.50m depth is required that will be ≤2.50m from the side boundaries for the rear extension of the building. Bulk excavation down to approximately ≤4.0m depth will be required for the proposed lift that will be ≤10.0m from the side boundaries and adjacent to the existing building onsite. Proposed works will also include an approximately 4.0m depth excavation for an OSD tank within the front of the site near the existing garages.

Detail of the proposed excavation and distance to the neighbouring structures is supplied in Section 5.3.2 of this report.

3. SITE FEATURES:

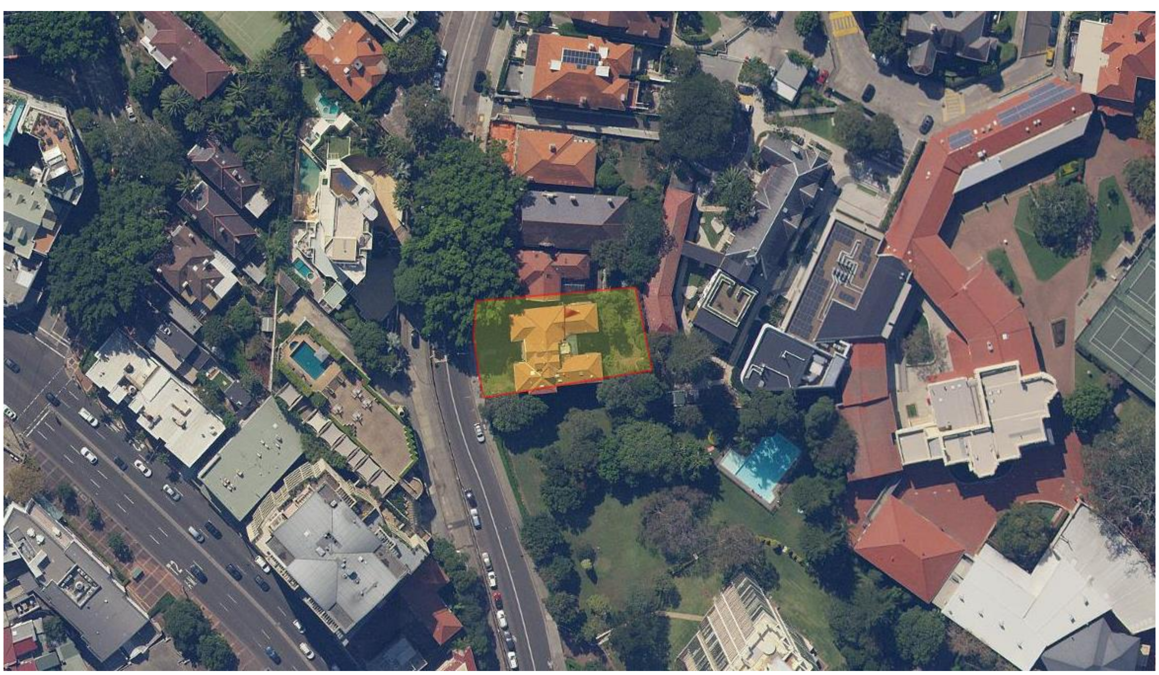
3.1. Description:

The site is a rectangular shaped block located on the higher eastern side of Darling Point Road and on the western side of the Ascham School within gently west dipping topography. The site is 3.0m higher than the road reserve with the western front of the site raised/supported by a mortared sandstone retaining wall.

The site has a front west boundary sum of 24.385m, a side north boundary of 40.165m, a side south boundary of 42.745m and a rear eastern boundary of 21.375m, as referenced from the provided survey plan.

Ground Surface Level (GSL) within the site reduce from a high of approximately RL 64.94m within north-easter corner of the site to a low of approximately RL 40.52m adjacent at the south-western corner of the site with the road reserve at RL 36.67m.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph-1. General Views of the site at the time of investigation are provided in Photograph-2 to Photograph-4.



Photograph-1: Aerial photo of site and surrounds

Photograph-2: Front view of the site from Darling Point Road. View looking east.

Photograph-3: View from the rear of the site. View looking west.

Photograph-4: Rear of the site. View looking south-east.

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Quaternary sand (Qhd) located approximately 50m south from exposure of the underlying Hawkesbury Sandstone (Rh) which is of Triassic Age.

The Quaternage Age medium to fine grained "marine" sands with podsols (Qhd) have been deposited over the Hawkesbury Sandstone (Rh) bedrock. Podsols form in sand under leached horizons with a profile usually consisting of a leached light-colour surface layer and subsoil containing clay and oxides of iron, varying in colour from red to yellowish red. These podsols can contain cemented gravels, laterite and piezolites.

The rock unit (Rh) typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminate. Morphological features often associated with the weathering of Hawkesbury Sandstone are the formation of near flat ridge tops with steep angular side slopes that consist of sandstone terraces and cliffs in part covered with sandy colluvium. The terraced areas often contain thin sandy clay to clayey sand residual soil profiles with intervening rock (ledge) outcrops.

An extract of the Sydney geological series sheet is provided below. It should be noted that the locations of the geological boundaries should be considered approximate only.

Extract 1: Site location in reference to Sydney 1:100 000 Geological Series Sheet

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and limited inspection of adjacent properties on the 1st December 2022 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing structures and ground conditions.

It also included the drilling of five auger boreholes (BH1 to BH5) to investigate sub-surface geology. A hand auger was used as access to required test locations within the site was limited and access for a conventional drilling rig was unavailable.

DCP testing was carried out from ground surface adjacent to the boreholes in accordance with AS1289.6.3.3 – 1997, “Determination of the penetration resistance of a soil – Perth sand penetrometer” to estimate near surface soil conditions.

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

4.2. Field Observations

Darling Point Road is formed with a gently south dipping bitumen pavement with low concrete gutter and kerbs along the side and the western side is supported by a concrete retaining wall. Between the road pavement and site is a concrete pathway and concrete crossover driveway which allows access to two front lock up garages located at the western front of the site. Ground movement, cracking or underlying geotechnical issues was not observed within the road reserve which appeared in good condition.

The western front of the site is approximately 3m higher than the street and contains two near level front grass lawns (above the site-garages) with an entry staircase and pathway through the centre. The lawns are partly supported along the western side by an approximately 3m high mortared sandstone block retaining wall. Along the southern boundary is another concrete staircase which allows access to the site. The front lawns and walls showed no signs of excessive ground movement, however the retaining walls contained minor cracking and loss of mortar at some locations (Photograph-5 and Photograph-6).

Photograph-5:Loss of mortar/cracking in retaining wall. View looking north.

Photograph-6:Loss of mortar/cracking in retaining wall. View looking south.

The center of the site is broadly occupied by a three to four storey ‘U shaped’ brick building that extends north and south to approximately 1.50m from the side boundaries. A grass and concrete pathways pass to the northern and southern sides of the building, respectively. The site building appears to be at least 70 years old and it appeared in good condition with no signs of excessive cracking or underlying geotechnical issues. Similarly the side pathways appeared in good condition with no excessive deflection or underlying geotechnical issues.

The rear of the site (Photograph-4) contains a concrete pathway at the rear of the building then a low (≤1.0m) concrete retaining wall at the base of a steep garden slope which rises to a higher grass lawn. A single storey brick shed is located within the southern side of the rear lawn, whilst a concrete retaining wall to ≤2.0m height is located along the east boundary supporting the neighbouring property above the level of the site. Signs of ground movement or underlying geotechnical issues was not observed within the rear of the site.

The neighbouring property to the north (No. 23 Darling Point Road) is similar to the site, it contains a street level lock up garage at the front with a grass lawn above. The centre of the propery is broadly occupied by a two storey brick building which contains a timber pergola to the south-east. Access to the rear of the property appears possible via a pathway adjacent to the common boundary and the rear of the property also contains a grass lawn. The property dwelling's roof and pergola's roof extends south to the common boundary. The neighbouring property contains a similar GSL to the site along the common boundary. The property building appers to be at least 70 years old and it appeared in good condition. Signs of cracking or excessive ground movement adjacent to the common boundary or underlying geotechnical issues was not observed.

The neighbouring property to the south (No.3-No.17 Darling Point Road) is relatively large and contains a multi-storey unit building within the centre and a swimming pool within the northern side of the property. The remainder of the property is broadly occupied by open grass lawns. The swimming pool and unit building are located approximately 25m and 50m from the common boundary. The GSL of the property appers to be generally 0.50m higher than the site along the common boundary and up to 3.0m higher than site at the western end of the common boundary where the site is supported by a mortared sandstone block wall. Signs of cracking, excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and all structures appeared in good condition.

The neighbouring property to the east (No.188 New South Head Road – Ascham School) is broadly occupied by multi-storey buildings with vehicular entrance from Darling Point Road and Octagon Road located within the northern side of the property. The closest property structure to the site comprises a two storey brick building that is located approximately ≤1.50m from the common boundary. The neighbouring property is approximately ≤2m higher than the site and is supported by a ≤2m high concrete retaining wall. The closest property structure appears to be at least 50 years old and appeared in good condition, similarly the common boundary wall appeared in good condition with no signs of cracking, tiling or underlying geotecnial issues.

4.3. Field Testing:

The boreholes (BH1 to BH5) were drilled using a hand auger within the centre to rear of the site. The boreholes were discontinued within sand at varying depths between 2.00m (BH4) and 3.10m (BH1).

Dynamic Penetrometer (DCP) tests were carried out from the GSL adjacent to the boreholes and they were discontinued within the sand unit at varying depths between 2.30m (DCP3) and 2.90m (DCP5).

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

- **FILL** – this layer was encountered to a maximum of 0.75m. It was classified as very loose, brown, fine to medium grained, moist, silty sand with gravels and plant roots.
- **SAND –** this unit was encountered below the fill down to the maximum drilled depth of 3.10m. It was generally classified as pale brown/grey, moist, silty sand and medium dense, becoming dense below varying depths between 1.30m (DCP4) and 2.20m (DCP2).

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

5. COMMENTS:

5.1 Geotechnical Assessment:

The site investigation identified sandy fill to a maximum depth of 0.75m (BH2), underlain by generally medium dense sand becoming dense/very dense below varying depths between 1.30m and 2.40m to the maximum drilled depth of 3.10m depth. Bedrock, a free-standing water table or significant seepage were not encountered in the investigation to the maximum tested depth of 3.10m.

The field observations identified cracking and minor bulging at the western front wall and whilst this only appears related to age deterioration and soil creep. It is recommended that a structural engineer assess the competency of the wall and provide remedial recommendations if necessary. CGC can assist for any geotechnical investigation into the front wall's footings/founding conditions, if deemed necessary by the Structural Engineer.

Based on the proposed excavation depths and geological conditions encountered, it is anticipated that the excavation will extend through fill/sand down to the Basement Excavation Level (BEL). Therefore, the excavation can be done by hand or using conventional earth moving equipment.

Based on the proposed rear excavation the safe batter slopes recommended in Section 5.3.2 of this report are not achieved towards the northern and southern boundaries. The use of a contiguous pile wall or similar are viable options for permanent support. All retaining structures must be constructed as per *Earth-retaining structures AS 4678-2002* and as per Section 5.3.3.

The use of driven piles or similar likely to create ground vibrations during their installation are not recommended within the site.

Extreme care must be exercised not to undermine the existing site building's footings during the excavation works. The existing building's footings within the excavation zone of influence (ZOI, defined by a lines from the bottom of the excavation projecting upward at an angle of 30° to the ground surface) should be exposed by the builder and if founded within the ZOI, then will require underpinning extending to below the excavation ZOI down to competent material (recommended sand of at least medium dense).

It is recommended that all new footings be extended through the fill and be founded within sand of similar density (of at least medium dense). Preliminary bearing capacity suitable for the site is provided in Section 5.3.1 of this report.

Additional geotechnical testing is recommended (e.g. additional DCP test) to approximately 4.0m to 5.0m depth at the rear of the site prior to the final structural design.

Where sand compaction is required, it is recommended that saturation and then compaction of the soil via a plate compactor or similar (and maintenance of a moist condition by placement of a membrane) should be utilised to increase sand density, along with geotechnical testing post compaction to verify that the required allowable bearing capacity has been achieved. Larger vibration rollers are not recommended.

An experienced professional Structural Engineer should assess any potential increase of loading to existing footing from the new works. Where the load to an existing footing has increased, then further testing of the founding soil may be required.

It is anticipated that the proposed works will generate minimal to no vibrations to the neighbouring properties provided driven support systems are not utilized. Also, the natural ground water level will not be disturbed, hence monitoring or a contingency plan for ground vibrations and groundwater changes will not be required.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on neighbouring properties provided the recommendations of this report are implemented in the design and construction phases. The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tool investigation due to access limitations. This test equipment provides limited data from small isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

5.2. Site Specific Risk Assessment:

Based on our site investigation and review of the proposed works we have identified the following credible geological/geotechnical hazard which need to be considered in relation to the existing site and the proposed works. The hazard is:

A. Landslip (earth slide $5m^3$-$10m^3$) from soils due to Site excavation.

Hazard A was estimated to have a **Risk to Life** of **1.41×10^{-7}** for a single person, while the **Risk to Property** was considered to be **'Low'**.

There is a 'Low' Risk to Property for Hazard A, the assessments were based on excavations with no support or underpinning or planning. Provided the recommendations of this report are implemented including installation of retaining 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. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:

Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings on sand.
Type of Footing	Strip/Pad or Piers.
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	- Loose Sand: 150kPa - Medium Dense Sand: 200kPa - Dense sand: 300kPa

Sub-grade material and Maximum Ultimate End Bearing Capacity for a single pile footing:

<u>**Pile Diameter = 0.45m**</u>

Depth (m)/RL(m)	Qp (Ultimate End bearing capacity) (kPa)	Average Side Friction for pile footing (fs)	Qu (Ultimate load-carrying capacity of the pile per pile) (kN)
2.0/40.0	570	25kPa	90
3.0/39.0	1600	25kPa	270
4.0/38.0	2200	35kPa	370

Note: Friction resistance values down to 1.0m depth is considered to be minimal to nil, as there is potential for fill down to 1.0m depth

Basic geotechnical strength reduction factor for ultimate capacities as per Piling Design and installation AS 2159 – 2009	ϕ_{gb} = 0.48 *
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e – Shallow Soil Site

Remarks:

*Pending further investigation, level of construction control and redundancy in system

All permanent structure footings should be founded off sand of similar density to reduce the potential for differential settlement unless designed for by the structural engineer.

All new footings must be inspected and tested by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity. The footings must be kept in a moist stable and undisturbed to ensure the foundation conditions remain in a suitable state. This is mandatory to allow them to be 'certified' at the end of the project.

Sand density over large areas (i.e. large pad) is very difficult to maintain. Therefore, where proposed reduced allowable bearing pressures should be used and CGC should be contacted for suitable values.
Should a bored pile foundation/retention solution be adopted it is considered that based on the borehole sidewall instability encountered during the investigation, the sand soils underlying the site will likely require the use of temporary steel liners and tremmie placement of concrete or contiguous flight auger (CFA) methods.

5.3.2. Excavation:

Property Separation

The tables below shows the properties potentially affected by the proposed excavation and the separation distances to the shared property boundary and structure.

Table 1: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No.23 Darling Point Road	Dwelling, pergola, pathway and rear lawn	2.50	2.50	Lawn, pergola and dwelling directly adjacent to the boundary
South	No.3-17 Darling Point Road	Lawn, pool and building	2.50	2.50	Lawn directly adjacent to the boundary, pool 25m from the boundary and building 50m from the boundary
West	Darling Point Road	OSD Tank	4.0	1.0	Potential services within Darling Point Road
East	No.188 New South Head Road	Lawn and building	2.50	7.50	Lawn is directly adjacent to the common boundary and building is another 1.50m from the boundary

Type of Material to be Excavated	Fill ≤ 0.75m depth (BH2).
	Sand ≥ 0.75m depth through to the base of the Bulk Excavation Level (BEL).

Guidelines for batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill/Sand/Silty Sand	1.5:1.0	2.0:1.0

Remarks:

Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Topsoil/Sandy Soils	Excavator with bucket.
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Not applicable, unless the use of vibration generating equipment/support systems (not recommended on this site).	
Full time vibration Monitoring Required	Not applicable	
Geotechnical Inspection Requirement during construction	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Prior to/during construction of the proposed retaining structure to confirm design and construction conditions are achieved/maintained • During installation of excavation support. • For assessment of batter slopes. • All excavated footings.	
Dilapidation Surveys Requiremen	Recommended for structures with 7.0m of proposed excavation perimeter, this will reduce the potential for spurious claims of damage.	

5.3.3. Retaining Structures:	
Required	New retaining structures will be required as part of the proposed development. All excavation will need retaining structures unless permanent batters can be formed. It is recommended that support prior to bulk excavation be implemented to the north and south of the rear extension where the safe batter slopes cannot be formed.
Types	Where permanent support prior to excavation is required, the construction of a contiguous soldier pile is a viable option. Where minimal lateral deflection is required, lateral support (if needed) could be provided by internal bracing or propping or similar. Where support post excavation is required, the construction of a steel reinforced concrete/concrete block wall is suitable where temporary support or stable batters can be implemented.

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

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Sand Fill/Loose Sand	18	$\phi' = 28°$	0.35	0.52	N/A
Sand-Medium Dense	18	$\phi' = 30°$	0.30	0.45	N/A
Sand-Dense to Very Dense	20	$\phi' = 35°$	0.27	0.43	3.69

Remarks:

In suggesting these parameters it is assumed that the retaining walls will be fully drained due to sandy conditions 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 hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	No
Site Location and Topography		Within gently west dipping topography at mid-slope level
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Possible, requires insitu testing of soil permeability

Remarks:

All new structure gutters, down pipes and stormwater intercept trenches should be connected to a sotmwaer system design by a Hydraulic Engineer which discharges to the Council's stormwater system off site. Onsite Stormwater Disposal may be available on site, pending Infiltration test. CGC can assist with further testing.

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review the structural design drawings, including the retaining structure/batter slope design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect the installation of excavation support measures and batter slopes.
3. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure prior to the placement of steel or concrete.

Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

6. PRELIMINARY IMPLEMENTATION PLAN:

Excavation support is required for the proposed excavation works to protect the common boundary, neighbouring structures and site-dwelling structures.

6.1. Monitoring Program:

6.1.1. Settlement and Deflection:

Based on the proposed works, it is considered likely that settlement or deflection of soils/structures will occur external to the excavation if poor support systems are implemented. Provided an adequately engineer designed retention system is implemented prior to bulk excavation of the sandy soils underlying the site then settlement/deflection outside the excavation perimeter is likely to be negligible.

To ensure wall movements are within anticipated design tolerances, an accurate survey monitoring program of the retaining walls (north and south sides of the rear extension excavation) and site dwelling structures (adjacent to the lift excavation), should be put in place for the duration of the excavation works. It is also recommended that survey points are installed on adjacent site and neighbouring structures including boundary walls within a 2.0H: 1.0V zone of influence of excavation, and measured prior to any bulk excavation, to allow assessment and early detection of movement should latent, unforeseen ground conditions be encountered. These measurements should be completed/undertaken by a registered surveyor to $\leq$2mm accuracy and should be re-measured following completion of the support wall system.

All survey re-measurement results should be assessed immediately by the surveyor and/or site foreman and referred to the geotechnical and structural engineers where any deflection/variation of ≥5mm from the original value is identified.

6.1.2. Ground Vibrations:

Driven excavation support methods are not recommended at the site due to the potential for high vibration levels from these types of support which is likely to result in compaction of loose sandy soils in adjacent areas including below existing shallow footings.

The bulk excavation will comprise of sand and therefore it is expected that this excavation will not produce significant ground vibrations and full time vibration monitoring is not required.

6.1.3. Ground Water:

Seepage was not observed to a maximum investigation investigated depth of 3.10m. The presence of a groundwater table is not expected at least shallower than the proposed BEL. The proposed works will therefore not intersect or be impacted by groundwater. Therefore groundwater monitoring will not be required.

6.2. Contingency Plan:

6.2.1. Settlement and Deflection

Survey measurements of settlement and deflection should be assessed immediately by the site foreman and referred to the geotechnical engineer and structural engineer where any deflection/variation of ≥5mm from the original measured values is identified. If this occurs, excavation should cease and additional support/retention measures may need to be implemented to limit any further settlement/deflection or subsequent damage. The geotechnical engineer and structural engineer should be immediately contacted regarding advice for such measures.

7. CONCLUSION:

The field investigation identified the presence of fill, underlain by generally medium dense sand becoming dense/very dense with depth to the maximum investigated depth of 3.10m (BH1). Bedrock, a free-standing water table or significant seepage were not encountered in the investigation.

It is anticipated that the excavation will extend through fill and sand to the Base Excavation Level (BEL). Therefore, the use of rock breaking equipment will not be required and the works can be done by hand or by conventional earthmoving equipment.

Based on the excavation depth and the distance to the north and south boundaries, the safe batter slopes recommended are not achievable. Therefore, the construction of support prior to excavation will be required.

It is recommended that all new footings be extended through the fill and be founded onto/within natural sand of at least medium dense.

The risks associated with the proposed development are and can be maintained within 'Acceptable' levels with negligible impact to neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

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8. REFERENCES:

1. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
2. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin.
3. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
4. Australian Standard AS 2870 – 2017, Residential Slabs and Footings

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

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

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

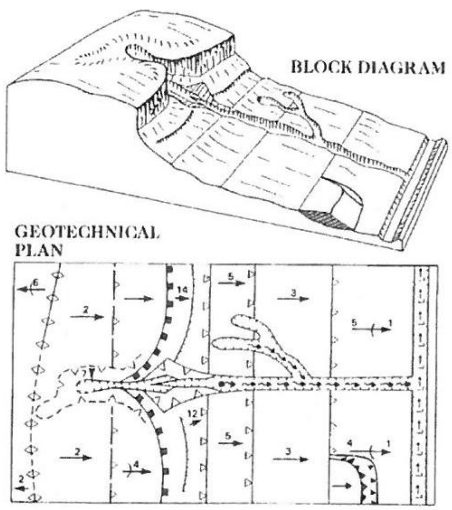
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

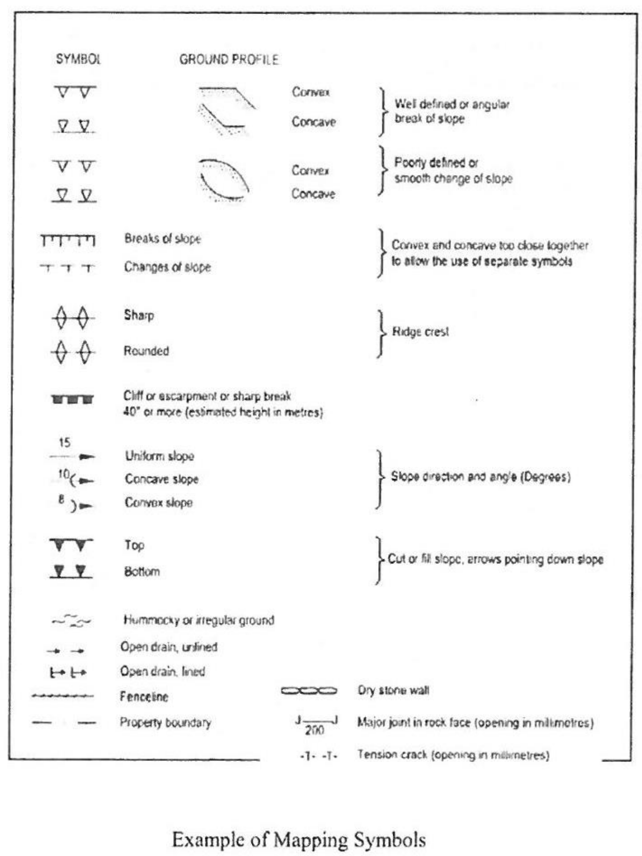
Reproduction of Information for Contractual Purposes

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

Site Inspection

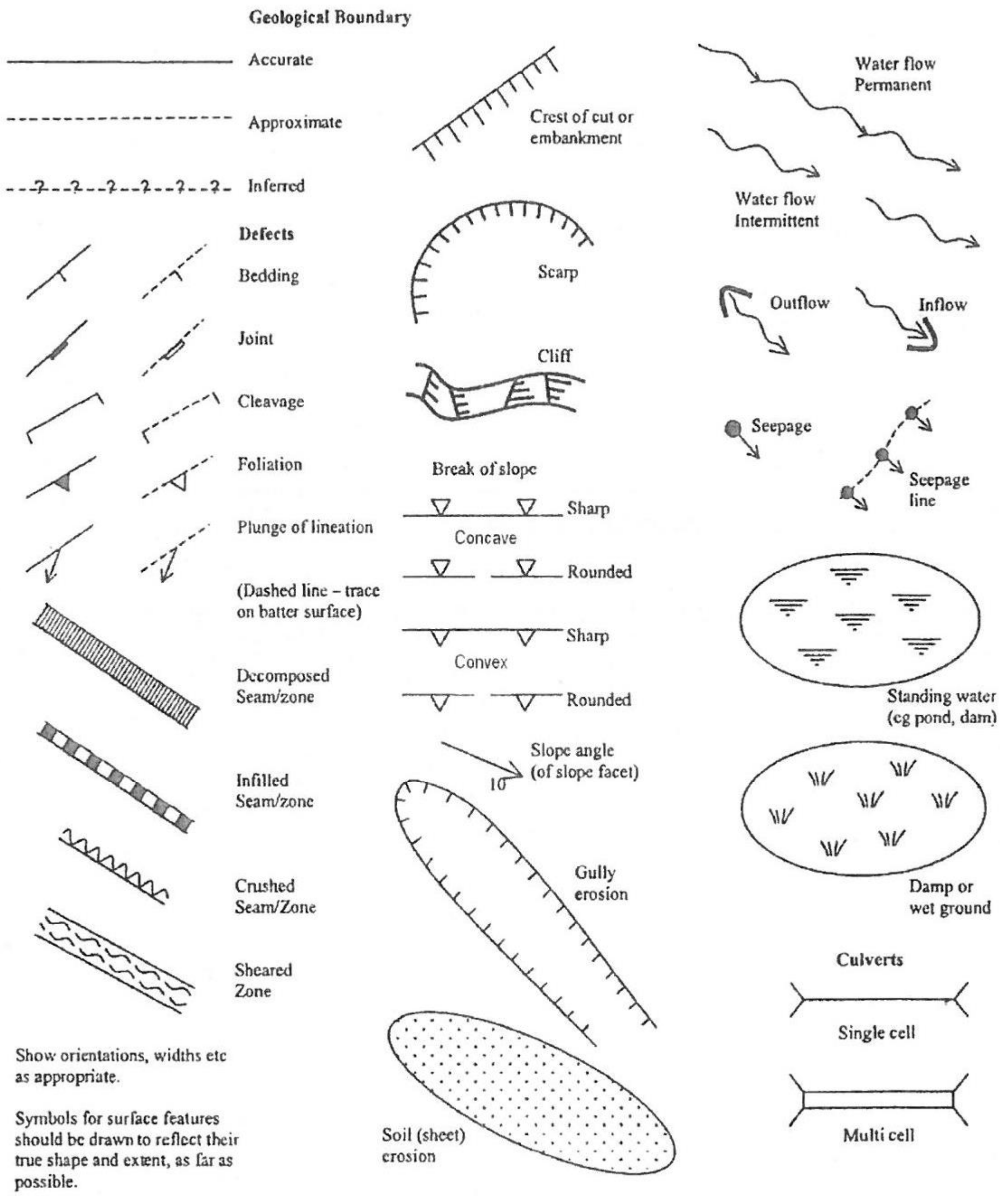
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





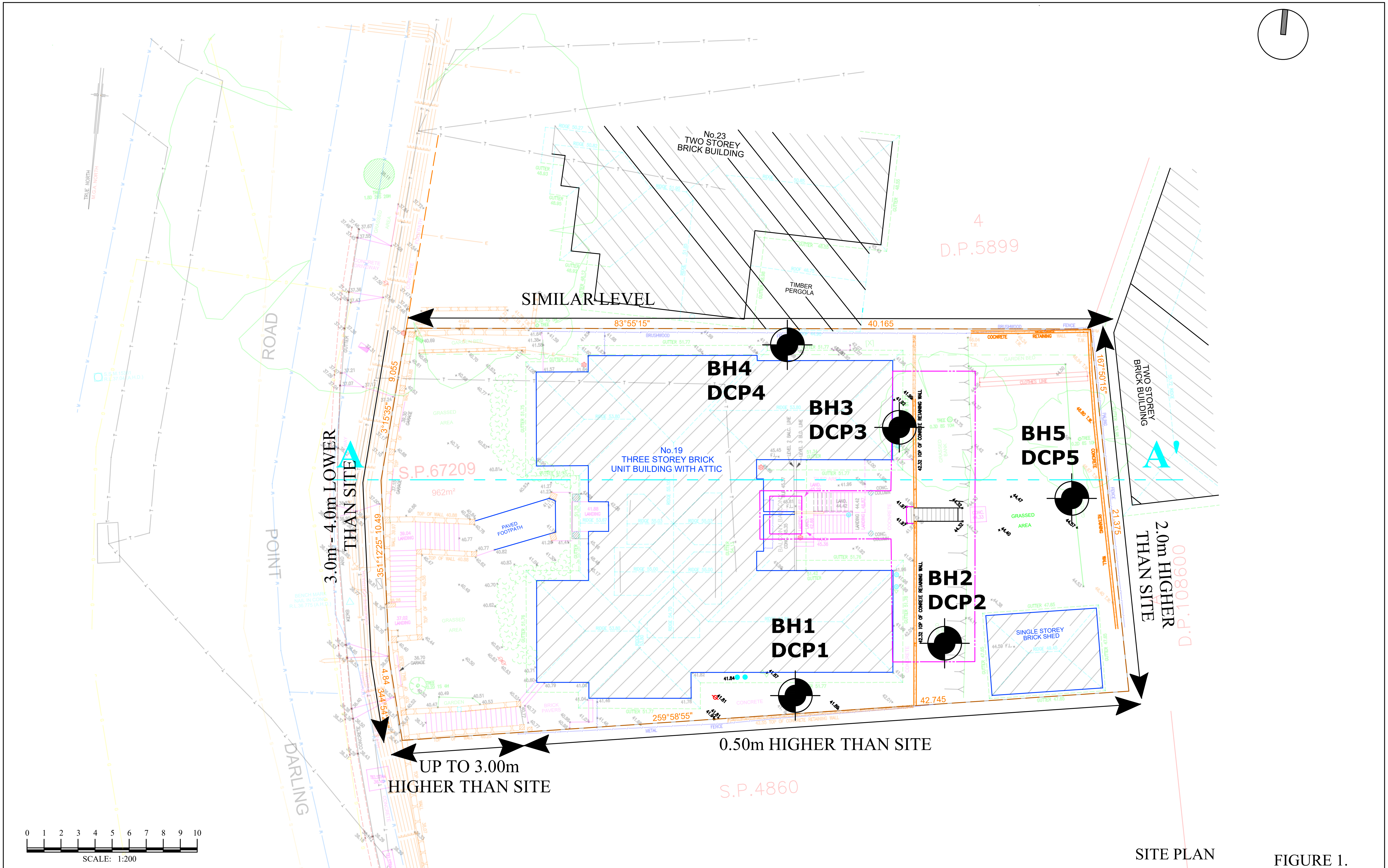
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN

FIGURE 1.

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100

ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883

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

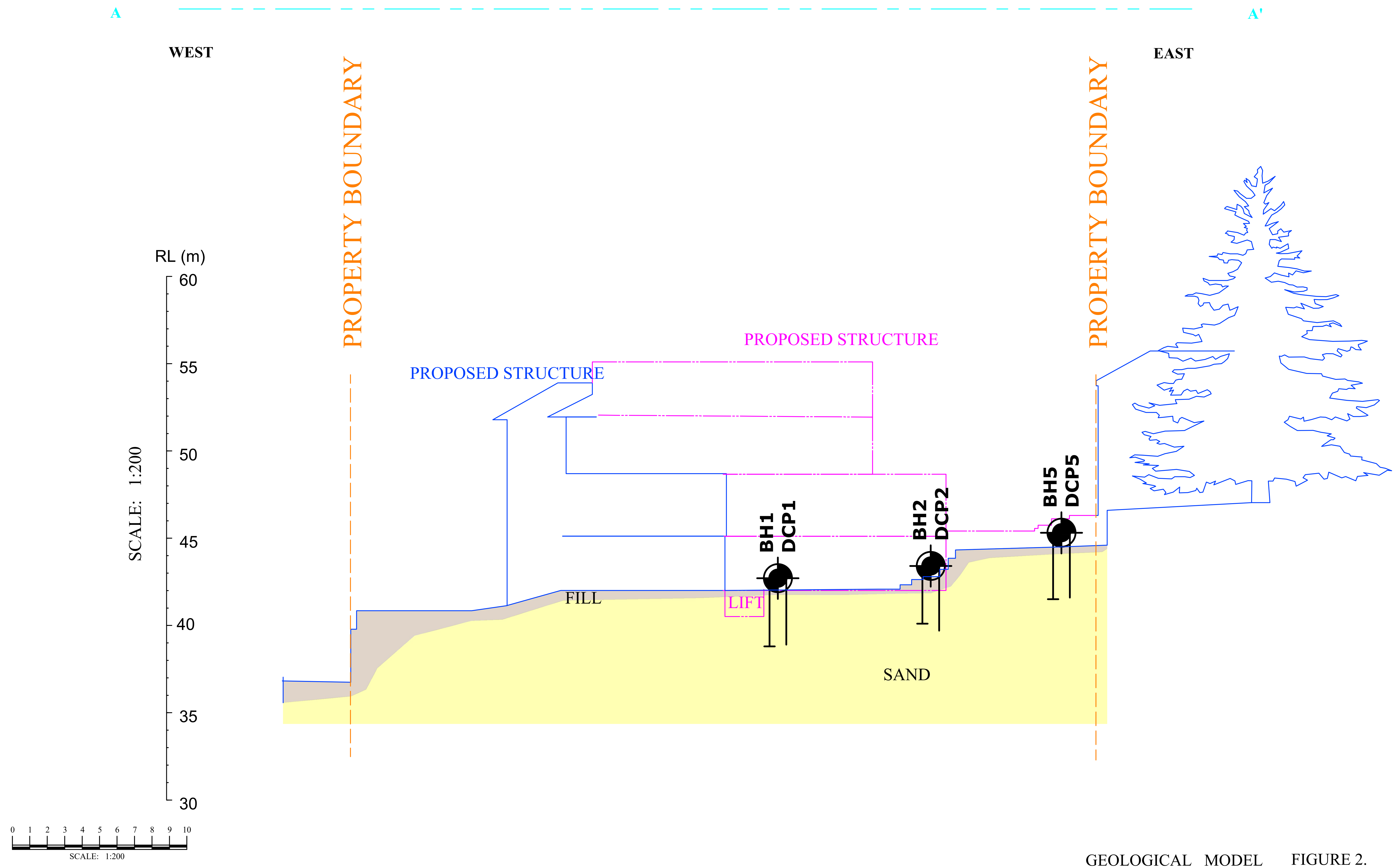
LEGEND

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

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	5/12/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2022-273

PREPARED FOR:
ASCHAM SCHOOL

ADDRESS:
No. 19 DARLING POINT ROAD
DARLING POINT



GEOLOGICAL MODEL FIGURE 2.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
(blue line)	EXISTING STRUCTURE
(magenta line)	PROPOSED STRUCTURE
(brown fill)	FILL
A — A'	CROSS-SECTION REFERENCE LINE
(orange dashed line)	PROPERTY BOUNDARY
(yellow fill)	SAND

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	5/12/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2022-273

PREPARED FOR:
ASCHAM SCHOOL

ADDRESS:
No. 19 DARLING POINT ROAD
DARLING POINT

BOREHOLE LOG

CLIENT: Ascham School **DATE:** 1/12/2022 **BORE No.:** 1

PROJECT: Alterations and additions **PROJECT No.:** 2022-273 **SHEET:** 1 of 1

LOCATION: 19 Darling Point Road, Darling Point, NSW **SURFACE LEVEL RL(m):** 41.90

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00						
0.05		CONCRETE SLAB				
0.15		TOPSOIL/FILL: Very loose, grey, fine to medium grained, moist, silty sand				
	SM	SAND: Very loose, orange red, fine to medium grained, moist, silty sand with cemented sand (podzol)				
0.30		… medium dense				
0.50						
0.65		… becoming pale brown				
1.00						
1.80		... dense				
1.90		… very dense				
2.00						
3.00						
3.10		END OF BOREHOLE at 3.10m depth within sand				

RIG: None

METHOD: Hand Augers

GROUND WATER OBSERVATIONS:

REMARKS:

DRILLER: AC

LOGGED: PS

CHECKED: TMC

BOREHOLE LOG

CLIENT: Ascham School **DATE:** 1/12/2022 **BORE No.:** 2

PROJECT: Alterations and additions **PROJECT No.:** 2022-273 **SHEET:** 1 of 1

LOCATION: 19 Darling Point Road, Darling Point, NSW **SURFACE LEVEL RL(m):** 42.6

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSOIL/FILL: Very loose, brown, fine to medium grained, moist, silty sand with some gravels and roots				
0.75	SM	SAND: Medium dense, pale brown, fine to medium grained, moist, sand				
1.00						
2.00						
2.20		… dense				
2.50		END OF BOREHOLE at 2.50m depth within sand				
3.00						

RIG: None DRILLER: AC

METHOD: Hand Augers LOGGED: PS

GROUND WATER OBSERVATIONS:

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Ascham School **DATE:** 1/12/2022 **BORE No.:** 3

PROJECT: Alterations and additions **PROJECT No.:** 2022-273 **SHEET:** 1 of 1

LOCATION: 19 Darling Point Road, Darling Point, NSW **SURFACE LEVEL RL(m):** 42.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00						
0.05		CONCRETE SLAB				
0.15		SLAG (Well compacted)				
0.20		TOPSOIL/FILL: Very loose, grey, fine to medium grained, moist, silty sand				
0.50	SM	SAND: Medium dense, pale brown, fine to medium grained, moist, silty sand				
1.00						
1.40		… becoming orange and dense				
1.60		… very dense				
2.00						
2.30		…bands of grey sand				
3.00		END OF BOREHOLE at 3.00m depth within sand				

RIG: None DRILLER: AC

METHOD: Hand Augers LOGGED: PS

GROUND WATER OBSERVATIONS:

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Ascham School **DATE:** 1/12/2022 **BORE No.:** 4

PROJECT: Alterations and additions **PROJECT No.:** 2022-273 **SHEET:** 1 of 1

LOCATION: 19 Darling Point Road, Darling Point, NSW **SURFACE LEVEL:** 42.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSOIL/FILL: Very loose, brown, fine to medium grained, moist, silty sand with some gravels and roots				
0.50	SM	SAND: Medium dense, pale brown, fine to medium grained, moist, silty sand				
1.00						
1.20		… becoming dense, pale brown/grey				
1.90		… becoming very dense, orange				
2.00		END OF BOREHOLE at 2.00m depth within sand				

RIG: None DRILLER: AC

METHOD: Hand Augers LOGGED: PS

GROUND WATER OBSERVATIONS:

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Ascham School **DATE:** 1/12/2022 **BORE No.:** 5

PROJECT: Alterations and additions **PROJECT No.:** 2022-273 **SHEET:** 1 of 1

LOCATION: 19 Darling Point Road, Darling Point, NSW **SURFACE LEVEL:** 44.5

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: Very loose, brown, fine to medium grained, moist, silty sand with some gravels and roots				
0.25		… becoming pale brown/brown				
0.40	SM	SAND: Medium dense, pale brown, fine to medium grained, moist, sand				
1.00						
1.50		… becoming pale grey				
1.90						
2.00						
2.50						
2.60		… becoming orange red				
2.90		… becoming very dense and medium to coarse grained				
3.00		END OF BOREHOLE at 3.00m depth within sand				

RIG: None DRILLER: AC

METHOD: Hand Augers LOGGED: PS

GROUND WATER OBSERVATIONS:

REMARKS: CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Ascham School
PROJECT: Alterations and additions
LOCATION: 19 Darling Point Road, Darling Point, NSW
DATE: 1/12/2022
PROJECT No.: 2022-273
SHEET: 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP3 RL 42.0m	DCP4	DCP5					
0.00 - 0.10	1	0	1	2	2					
0.10 - 0.20	1	1	1	1	1					
0.20 - 0.30	2	1	1	1	1					
0.30 - 0.40	3	1	2	2	2					
0.40 - 0.50	5	2	3	3	1					
0.50 - 0.60	4	2	3	3	2					
0.60 - 0.70	5	2	3	4	2					
0.70 - 0.80	4	1	5	4	2					
0.80 - 0.90	4	3	5	5	3					
0.90 - 1.00	4	3	6	7	3					
1.00 - 1.10	4	3	5	6	3					
1.10 - 1.20	3	4	4	7	4					
1.20 - 1.30	3	4	4	7	4					
1.30 - 1.40	4	4	4	8	4					
1.40 - 1.50	4	4	7	9	5					
1.50 - 1.60	6	5	8	9	4					
1.60 - 1.70	6	5	13	8	5					
1.70 - 1.80	7	5	13	8	5					
1.80 - 1.90	8	5	13	9	6					
1.90 - 2.00	10	5	12	10	7					
2.00 - 2.10	10	5	14	10	7					
2.10 - 2.20	10	6	16	12	8					
2.20 - 2.30	11	7	17	13	7					
2.30 - 2.40	12	7		13	9					
2.40 - 2.50	11	9		16	7					
2.50 - 2.60	12	9		16	9					
2.60 - 2.70	12	10		16	9					
2.70 - 2.80	12	11			10					
2.80 - 2.90	13	12			11					
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. F3.3, PERTH SAND PENETROMETER

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <5.0m³-10.0m³) from soils due to the rear excavation		Excavation generally ≤2.5m depth	a) Lawn located ≤ 2.50m from the excavation, impact 10% a) Lawn located ≤ 2.50m from the excavation, impact 5%		a) Person in the lawn 2hr/day avge. b) Person in the lawn 2hr/day avge.	a) Likely to not evacuate b) Likely to not evacuate	a) Person possibly impacted by fall b) Person possibly impacted by fall	
			Possible	**Prob. of Impact**	**Impacted**				
		a) No.23 Darling Poin Road (Lawn)	0.001	0.15	0.10	0.08	0.75	0.15	**1.41E-07**
		b) No.3-No.17 Darling Point Road (Lawn)	0.001	0.15	0.05	0.08	0.75	0.15	**7.03E-08**

* considered for person most at risk, where multiple people occupy area then increased risk levels

* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated

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

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

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide $<5.0m^3$-$10.0m^3$) from soils due to the rear excavation	a) No.23 Darling Poin Road (Lawn)	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) No.3-No.17 Darling Point Road (Lawn)	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low

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

Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

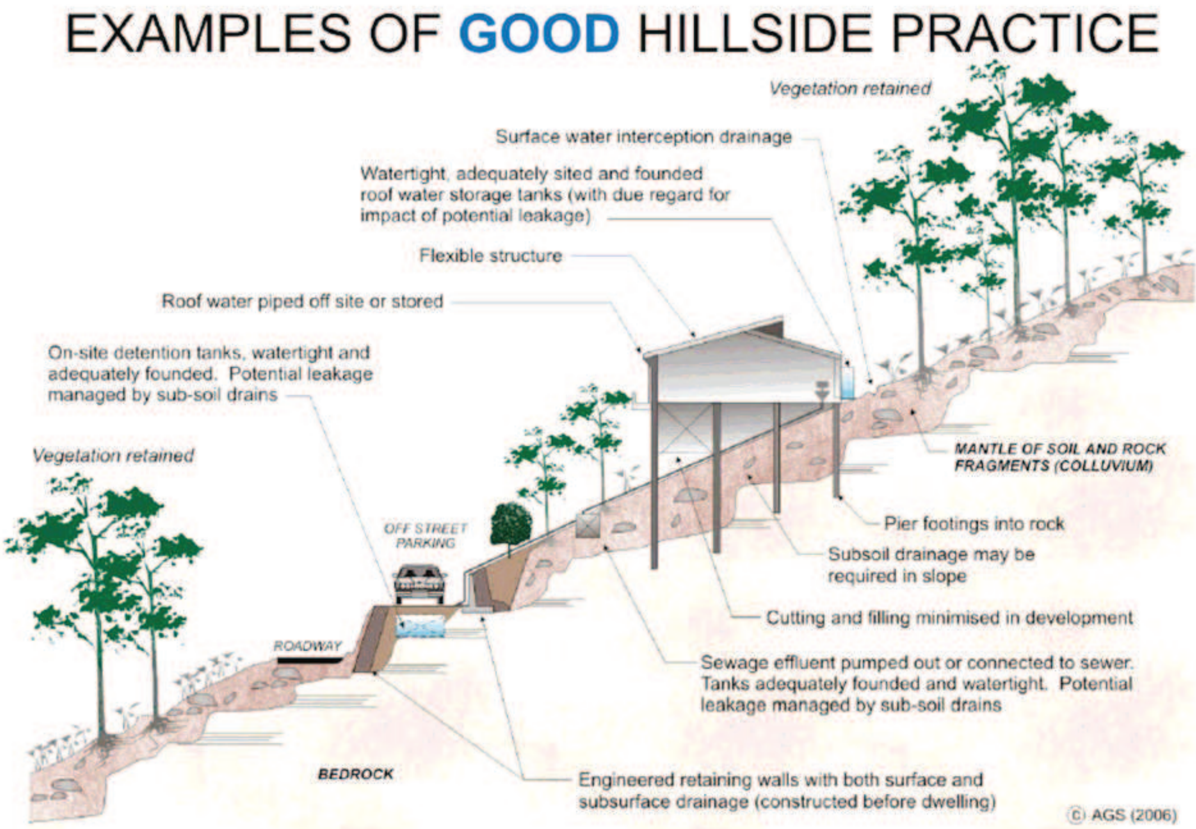
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

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

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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


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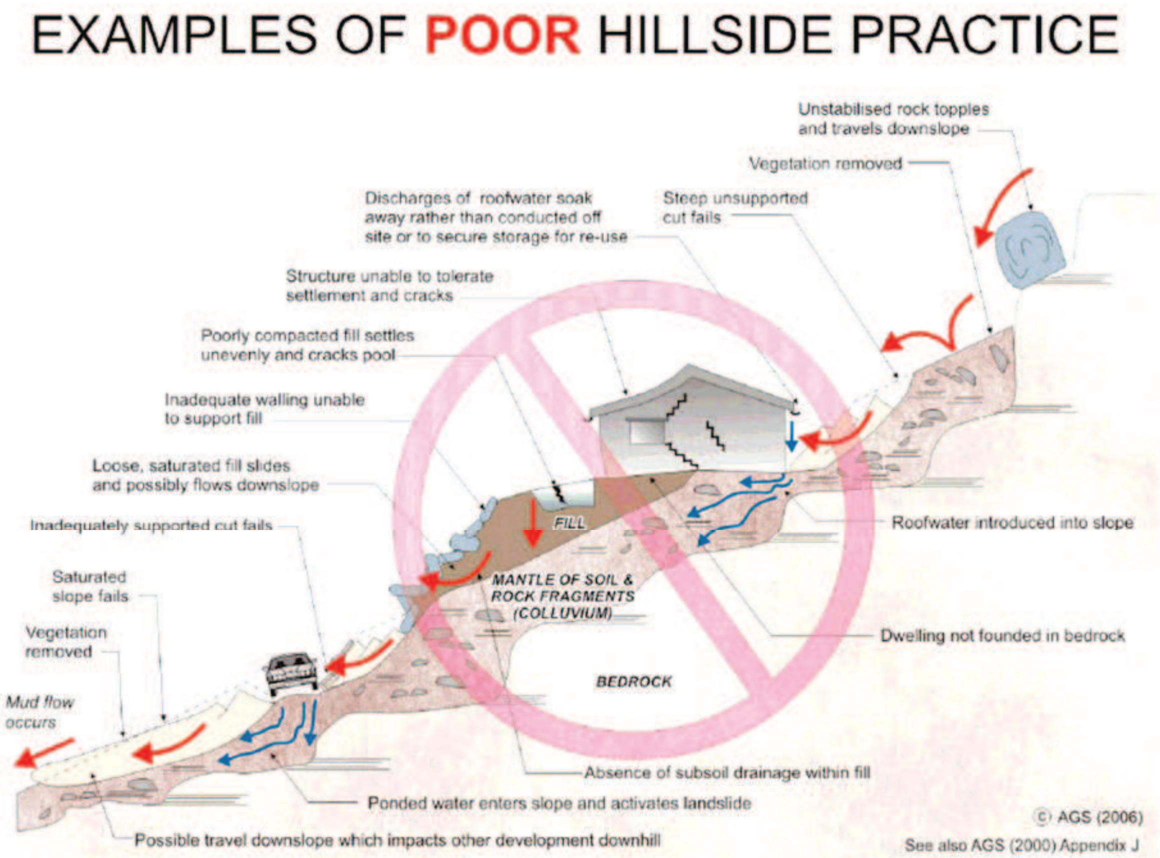
EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles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