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GEOTECHNICAL INVESTIGATION AND ASSESSMENT REPORT

FOR THE PROPOSED NEW DWELLING

AT

38 THE DRIVE, FRESHWATER, NSW

PREPARED FOR

TOBIAS PARTNERS

Project No.: 2024-185
July 2025

Document Revision Record

Issue No	Date	Details of Revisions
0	29th July 2025	Original Issue

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38 THE DRIVE, FRESHWATER, NSW

1. INTRODUCTION:

This report details the results of a limited geotechnical investigation carried out for a proposed new residential dwelling at 38 The Drive, Freshwater, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Tobias Partners for Development Application submission.

It is understood that the proposed works involve the demolition of the existing dwelling/structures within the site and the subsequent construction of a new residential dwelling which will include a front two storey garage/ home office/ rumpus structure and a rear three storey house connected via a tunnel and elevator. The lift is understood to require an excavation of approximately 18.5m in height. The tunnel linking the lift and garage level appears approximately 34 m in length. Excavations for the proposed dwelling will require excavations of approximately 5 m depth and the garage excavation will require excavations of approximately 11 m deep.

Reference to the Warringah Council – Local Environmental Plan 2011, the site is situated within a C landslip risk hazard zone. It is understood that further ground investigation will be undertaken at a later date. Nevertheless, this report includes the findings of this investigation and full risk assessment of the site for both property and life as per the AGS March 2007 publication.

The site investigation and reporting were undertaken as per the Proposal P25-171.1, Dated: 08th May 2025.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the entire site and accessible adjacent land, with identification of observable geotechnical hazards within the site, by a Senior Engineering Geologist including a photographic record,
b) DBYD plans and onsite service location by an accredited contractor.

c) Drilling of two (2no.) hand auger boreholes along with seven (7no.) Dynamic Cone Penetrometer (DCP) testing to investigate the subsurface geology, depth to bedrock and ground water conditions

The following plans and drawings were supplied for the work:

- Architectural Drawings – (Tobias Partners, Name: Preliminary DA Issue for Consultant Coordination, Dated: 22/07/25, Drawings: A1.101 to A1.108, A2.001 to A2.003, A2.101 and A2.102.
- Survey Drawing – CMS Surveyors, Drawing Name: 23724detail, Date of Survey: 20/08/24

2. PREVIOUS WORK

In addition to the above, CGC previously undertook a Geotechnical Feasibility Assessment of the Concept Design at 38 The Drive, Freshwater, dated 17th October 2024.

As part of a prior assessment for the subject site, it was identified that the construction of the new dwelling and garage/accommodation structure (drawings have been updated since this assessment, nevertheless, the findings still remain applicable) would require excavations of up to approximately 10.5 m (garage/accommodation) and 7.5 m depth (dwelling). These excavation depths were noted to be relatively common for the Northern Beaches area and were not considered to pose significant geotechnical constraints to development, largely due to the favourable excavation properties of the underlying Hawkesbury Sandstone.

The previous report highlighted that the upper surface of the sandstone bedrock typically presents the greatest uncertainty in excavation, as it can contain weathered zones, detached blocks, or unstable residual soil. Such features were expected to be assessed and addressed during excavation, with stability concerns deeper in the profile managed through appropriate stabilisation techniques, such as rock bolting or anchoring.

It was also noted that bulk rock excavation may induce ground vibration capable of affecting adjacent structures. However, vibration-related risks are well understood across the region, and a range of established methods, equipment, and monitoring protocols are available to mitigate these impacts. Provided that appropriate geotechnical input and monitoring are adopted, vibration was not considered a constraint to development. The proposed structure setbacks (minimum 1.0 m) were anticipated to further assist in reducing vibration transmission, particularly given the apparent condition and sensitivity of neighbouring structures.

Of particular note in the previous assessment was the proposed tunnel and associated lift shaft. While this was acknowledged as the most atypical aspect of the development, it was not considered to represent a geotechnical constraint. It was recognised that in residential contexts, tunnelling is often completed via open-cut methods followed by structural formation and backfilling. However, due to the depth of the proposed tunnel, a more traditional tunnelling approach may be necessary. Similar tunnels have been completed successfully in comparable conditions within the Sydney region, including a recent example in Mosman. It was anticipated that while the tunnel may be subject to longer approval timeframes due to safety and compliance requirements, construction feasibility in geotechnical terms remained favourable.

The previous report concluded that a detailed subsurface investigation would be required to further inform design and construction, including the drilling of at least three to four cored boreholes to depths exceeding proposed excavation levels. However, this was not considered critical at the Development Application stage due to the relative common nature of the excavation scales proposed.

3. PROPOSED DEVELOPMENT:

Based on the architectural drawings provided (Tobias Partners, Name: Preliminary DA Issue for Consultant Coordination, Dated: 22/07/25) it appears the ground level for the garage is proposed at RL24.0m with the floor levels of the house development ranging between RL39.0 (Plant room) and RL49.5m (Level 7 (level 3 of the dwelling)).

The new dwelling will require excavations of up to approximately 5.0 m deep, whilst excavations of approximately 11.0m will be required to accommodate the proposed garage.

The proposed tunnel connecting the garage to the residence appears to have a base level of approximately RL 24.00 m and extends roughly 34 m in length, terminating at a vertical lift shaft. The shaft is approximately 18.5 m high and is intended to provide access to Level 5 (ground floor of the proposed residence). At this stage, it is unclear whether the tunnel will be constructed via open-cut excavation and backfilling or through conventional tunneling techniques.

The tables below summarise the maximum excavations required and the distance away from the respective boundaries:

Dwelling

Distance from northern boundary and depth of excavation	Distance from Southern boundary and depth of excavation	Distance from Western boundary and depth of excavation
- The excavation edge (northwestern part of proposed dwelling) is approximately 1.35m away from the northern boundary and will be up to approximately 5m deep. - The excavation edge (northeastern part of proposed dwelling) is approximately 0.9m away from the northern boundary and will be up to approximately 3.5m deep.	- The excavation edge (southeast of the proposed dwelling) is approximately 0.9m away from the southern boundary and will be up to approximately 3m deep (though the topography reduces the other side of the boundary). - The excavation edge (southern part of the proposed dwelling) is approximately 3.05m away from the site boundary and will be up to approximately 5m deep.	- The excavation edge (western part of the proposed dwelling) is approximately 1.1m from the western boundary and will be up to approximately 5m deep. However, due to non-uniform site boundaries, in the northwestern corner of the proposed dwelling, it is proposed to excavate up to the boundary by approximately 2m deep.

Garage

Distance from northern boundary and depth of excavation	Distance from Eastern boundary and depth of excavation	Distance from Southern boundary and depth of excavation
- The excavation edge is approximately 1.15m away from the northern boundary and will be up to approximately 9.5m deep.	- The excavation will extend up to the eastern site boundary; however, it will comprise a gradual slope, with excavation depths ranging from approximately 1.0 m to 2.5 m, tapering to zero at the boundary.	- The excavation edge is approximately 0.9m away from the southern boundary and will be up to approximately 11m deep.

4. SITE FEATURES:

4.1. Description:

The subject site is an irregularly shaped parcel of land situated on the upper (western) side of the junction between Dick Street and The Drive.

An aerial image illustrating the site and its surrounding context is presented below as Photograph 1, sourced from the NSW Government SIX Maps spatial data platform.

Topographically, the site is characterised by a steep to very steep incline that rises westward, terminating at a plateau along the upper (western) portion of the site. The eastern portion of the site accommodates a single-storey garage structure, while a two-storey residential dwelling is located on the upper slopes in the west, partially built into the hillside.

Access across the steep terrain is provided via a series of zig-zagging stairs and walkways extending up from the eastern boundary. Additionally, a travellator structure is located along the northern boundary to facilitate movement up the slope.

The western garden area comprises a grass-covered and concrete patio area. In the northwestern corner, the site is bounded by a low brick retaining wall approximately 0.8 m in height, which retains the neighbouring property's garden area.

Outcropping sandstone bedrock is visible across much of the site. A detailed discussion of the exposed geological conditions is provided in Section 4.3.

4.2. Site Boundaries and Adjoining Properties

The subject site is bordered by six properties, described as follows:

- North – 6 Coast View Place (majority of northern boundary):
  This property extends along most of the northern boundary and comprises a two- to three-storey dwelling stepped into the hillside, located toward the northwest corner. A secondary dwelling (granny flat) and a swimming pool are situated approximately halfway down the slope. The main dwelling is located approximately 1.35 m from the boundary, while the granny flat and pool are positioned approximately 2.05 m from the boundary.

- Southeast – 36 The Drive:
  This property comprises a two- to three-storey dwelling excavated into a plateau within the hillside. The plateau is separated from the subject site by a near-vertical sandstone exposure, ranging from approximately 1 m high in the east to 5 m high in the west. The rear yard contains a patioed area that transitions into the exposed sandstone face. Several sandstone floaters were observed positioned precariously along the edge of the slope. The dwelling is set back approximately 0.6 m from the boundary.

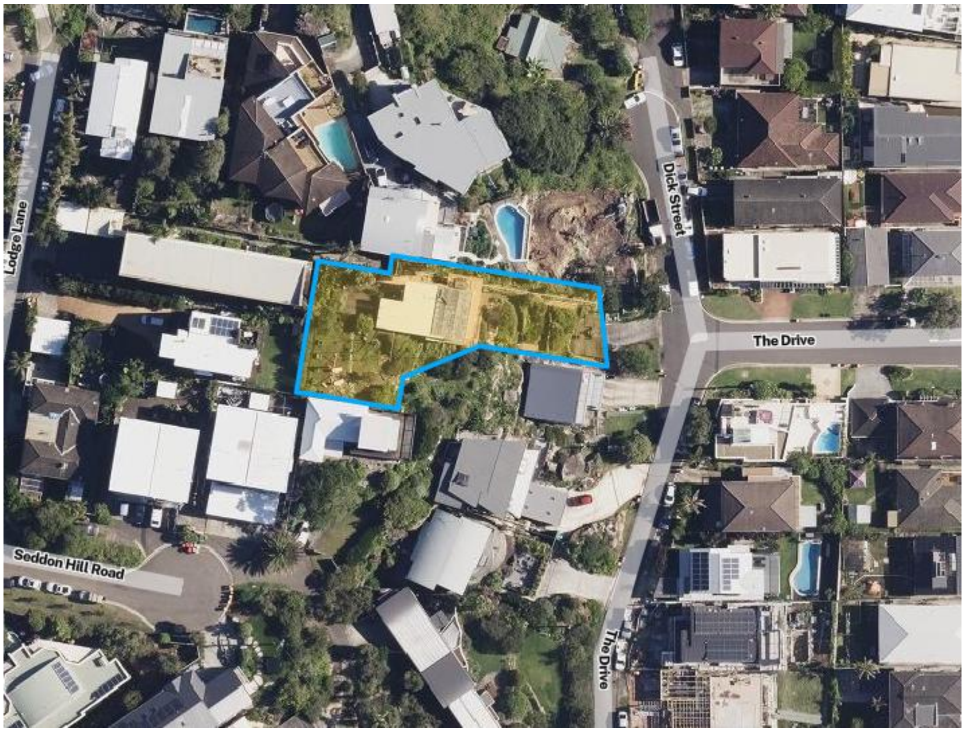
- South – 1 Seddon Hill Road:
  This property consists of a two- to three-storey dwelling stepped into the hillside. The dwelling was largely obscured from view by a timber boundary fence. The property slopes downward from west to east, with a level difference of approximately 2 m. The western portion of the site lies at a similar elevation to 38 The Drive. A swimming pool is identified in the topographical survey but was not visible during the site inspection. The dwelling is set back approximately 1 m from the site boundary.

- Southwest Corner – 3 Seddon Hill Road:
  This property features a two- to three-storey dwelling stepped into the hillside, presenting as three storeys to the southern (front) elevation and two storeys to the northern (rear) elevation. The rear garden is at a similar elevation to that of 38 The Drive. The dwelling is set back approximately 5 m from the boundary.

- West-Southwest – 9 Lodge Lane:
  This property comprises a two-storey dwelling, which was obscured by thick hedging during the site inspection. Based on the topographical survey and publicly available imagery, the property is inferred to lie at a similar elevation to 38 The Drive. A swimming pool is understood to be located near the shared boundary, though it could not be directly observed. The dwelling is set back approximately 8 m from the site boundary, while the pool is located approximately 1–2 m from the boundary.

- West and Part of North – 11 Lodge Lane:
  This property consists of a two- to three-storey apartment block stepped into the slope, with three storeys to the western side and two storeys to the eastern side. While levels could not be confirmed during the site visit, the topography suggests the western portion of the property is at a similar elevation to the subject site. The property also wraps around to form part of the northern boundary.

Along this section, a 1 m high brick retaining wall is present, with 36 The Drive located on the elevated side.



Photograph: 1 – Aerial photo of site and surrounds.

4.3. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Hawkesbury Sandstone (Rh) which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. This rock unit was identified in surface exposures within and adjacent to the site.

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. The outline of the cliff areas are often rectilinear in plan view, controlled by large bed thickness and wide spaced near vertical joint patterns. The dominant defect orientations are south-east and north-east. Many cliff areas are undercut by differential weathering along sub-horizontal to gently west dipping bedding defects or weaker sandstone/siltstone/shale horizons. Slopes are often steep (15º to 23°) and are randomly covered by sandstone boulders.

The sandstone bedrock was observed to be horizontally bedded, with individual bed thicknesses ranging from 0.4 m to 1.0 m. Where vertically exposed, frequent overhangs of up to 2.0 m (average approximately 0.5 m) were noted, likely resulting from preferential erosion along bedding plane defects. Dense vegetation obscured much of the exposed bedrock, limiting the extent of direct observation.

5. FIELD WORK:

5.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site on the 24th June 2025 by a Senior Engineering Geologist. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of rock outcrops, soil slopes, existing structures and neighbouring properties.

It also included the drilling of two boreholes (BH01 & BH02) to investigate sub-surface geology. A hand auger was used as access to the test locations for a conventional drilling rig was unavailable.

DCP testing was carried out from ground surface in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer" to estimate near surface soil conditions and depth to bedrock, at seven (7no.) locations.

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. Geological models/sections are provided as Figure: 2 and Figure: 3, Appendix: 2.

5.2. Field Testing:

The boreholes (BH01 and BH02) were drilled using a hand auger at select locations within the site with refusal encountered at depths of 0.25 m and 0.40 m, respectively. The Auger boreholes refused on sandstone bedrock of at least low strength.

DCP tests were carried out from the ground surface adjacent to the boreholes and at an additional select locations, with refusal encountered at depths of between 0.20 m and 0.60 m.

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

- **TOPSOIL:** A surficial layer of clayey sand topsoil was encountered at all borehole locations, extending to depths of approximately 0.20 m.

- **FILL:** Underlying the topsoil, a layer of loose sand with trace gravel was encountered in all boreholes, extending to depths between 0.25 m and 0.35 m. The gravel component consisted of both natural sandstone fragments and anthropogenic material. This is anticipated to be up to approximately 0.80m behind some of the relatively small retaining walls.

- **SAND WITH GRAVEL:** In Borehole BH02, a zone of loose sand with gravel was encountered below the fill layer, extending to a depth of approximately 0.40 m. This material was located directly above the underlying sandstone bedrock.

- **SANDSTONE BEDROCK:** Sandstone bedrock was encountered in all boreholes and inferred from all DCP tests at shallow depths ranging between 0.20 m and 0.60 m. The sandstone, as observed in borehole and surface exposures, is interpreted to be of low strength in its upper weathered profile. The sandstone bedrock was observed to be horizontally bedded, with individual bed thicknesses ranging from 0.4 m to 1.0 m. Where vertically exposed, frequent overhangs of up to 2.0 m (average approximately 0.5 m) were noted. Dense vegetation obscured much of the exposed bedrock, limiting the extent of direct observation.

There were no indications of significant seepage or a groundwater table in any of the boreholes during drilling or observed on the DCP rods on retrieval.

6. COMMENTS:

6.1. Geotechnical Assessment:

Our exploratory hole locations were constrained by the existing residential structure, steeply inclined slopes, hardstand surfaces, and the garage located at the front of the site. The investigation identified a thin surficial layer of topsoil comprising clayey sand to a depth of approximately 0.20 m, underlain by a loose sandy fill layer with trace gravel, extending to depths of between 0.25 m and 0.35 m. In BH02, a further layer of sand with gravel (interpreted to be Residual soil) was encountered to a depth of approximately 0.40 m. Sandstone bedrock of the Hawkesbury Sandstone Formation was encountered at shallow depths across all borehole locations, ranging between 0.20 m and 0.60 m. The bedrock was interpreted to be of low strength in the upper profile. No groundwater was encountered during the investigation, nor was any seepage within the exposed sandstone outcrops.

Wider Development

Excavations of up to approximately 5 m deep will be required for the proposed dwelling, and up to approximately 11 m for the proposed garage. These works are expected to be carried out generally within at least low-strength sandstone of the Hawkesbury Sandstone Formation. Based on current site observations and subsurface investigations, the excavations may be temporarily formed with steep to near-vertical batter angles, provided the rock mass remains intact, unfractured, and free of significant structural defects. Subject to further investigation and inspection during excavation, these may remain permanently unsupported; however, a conservative approach involving retention systems should be adopted in the meantime.

Although no critical defects were identified during the investigation, localised instances of undercutting within exposed cliff lines were observed. Where these features are to be impacted by the proposed excavation/ vibrations, overhanging rock blocks should be supported by columns, dentitions, rock anchoring, or removed/ cut back. The extent of these measures should be confirmed during initial excavation works/ following vegetation clearance. Ongoing geotechnical supervision during excavation is recommended to verify ground conditions and ensure safety.

Where isolated near-surface soil deposits occur, these should be battered at a permanent angle of approximately 30°. In cases where deeper soil deposits are present, retaining walls or other support systems may be required, or alternatively, this material may be removed (subject to its location). In the northern portion of the site, where the proposed ‘void’ is located, excavation will extend to the western site boundary. In this area, it will not be possible to batter the excavation face back at a 30-degree angle due to boundary restrictions. Therefore, it is recommended that a pre-excavation soldier piled wall be installed to retain the excavation, in order to achieve sufficient active resistance, the piled wall will likely need to extend to below the excavation level.

All new foundations should be designed to extend through any fill, residual soils, or weathered rock, and bear upon competent sandstone bedrock of at least low strength. Based on the borehole data, DCP results and site observations, competent bedrock is anticipated at depths of between approximately 0.20 m and 0.60 m. Given this, traditional strip or pier footings are considered suitable at this site and should be designed for an allowable bearing capacity of 1000 kPa on this low strength sandstone.

Tunnel/ Lift Shaft

It is currently unclear how the proposed tunnel and lift shaft will be constructed, as there are two common methodologies typically employed:

- Excavation and backfill: Excavation from the surface to the base level of the tunnel, followed by construction at formation level and reinstatement of backfill above; or

- Traditional tunnelling methods: Confined underground excavation progressing horizontally from one end, typically supported progressively as excavation advances.

As part of the proposed development, a tunnel approximately 34 m in length is planned to connect the garage structure at the front of the site (RL 24.0 m) to a vertical lift shaft located beneath the dwelling, extending approximately 18.5 m down/up to provide access to the upper floor levels. As such it is anticipated that a tunneling methodology will be preferred.

At this stage, Crozier Geotechnical Consultants (CGC) has not undertaken detailed investigation specifically targeting the tunnel or lift shaft elements. CGC does not provide specialist tunnelling design services and recommends that a qualified tunnelling engineer be engaged to undertake detailed design and provide construction advice specific to the chosen methodology.

Although site-specific data remains limited, and should be confirmed through further ground investigation works, it is noted that tunnelling through the Hawkesbury Sandstone Formation has been successfully completed throughout the Northern Beaches and wider Sydney region for both residential and infrastructure-scale projects. The bedrock unit underlying the site is generally suitable for tunnelling, provided localised conditions such as fracturing, weathering, and minor seepage are adequately assessed and managed.

Published literature, including Wong (2013), Enever (1999), and Pells (2002), has documented the presence of high north–south in situ horizontal stresses in the Hawkesbury Sandstone, which can induce ground movement around bulk excavations. Measured deflections of between 0.5 mm and 2.0 mm per metre of excavation depth (D) have been reported, typically diminishing to negligible levels beyond 1.5–2.0 D from the excavation boundary. Though this is not expected to influence any neighboring buildings due to separation distances.

The tunnel and lift shaft are expected to be excavated entirely within the sandstone, with the possibility of shale beds. It is assumed that as excavations progress downwards, the strength of the sandstone increases. In absence of any cored boreholes, a conservative approach should be adopted and it should be assumed that low strength sandstone will be encountered to the base of the excavation. Near-vertical excavation faces are expected to remain temporarily stable, provided that significant structural defects are not encountered.

While groundwater was not observed during the investigation, minor seepage is expected along bedding planes or defects. Gravity drainage is likely to be sufficient during excavation, though waterproofing (e.g. tanking) may be required to prevent localised inflows or seepage into the dwelling, garage or tunnel over the long term. These aspects should be confirmed through additional investigation.

Excavation is expected to be undertaken using medium-sized (15 to 20t) excavators fitted with rock hammers and saw or grinder attachments (based upon excavation depths), though this will need to be confirmed depending on construction/ tunnelling methodology. In view of the proposed cuts/ excavations full-time vibration monitoring is recommended to minimise disturbance, provide assurance to surrounding property owners, and assist in managing any claims of damage.

Given the preliminary nature of the concept design, further detailed subsurface investigation is recommended:

- Three to four cored boreholes along the proposed tunnel alignment and at the lift shaft location, drilled to at least 5 m below proposed excavation levels;
- Detailed logging and laboratory testing of the recovered core to assess rock strength, weathering, defect spacing and groundwater conditions;
- Assessment of groundwater seepage and hydrostatic pressures, including consideration for dewatering or drainage design (if required).

This additional information will support the assessment of construction methodology (cut-and-backfill versus traditional tunnelling), assist in the design of any required excavation support, and inform the staging and monitoring requirements for safe implementation. Ongoing geotechnical involvement, particularly during initial excavation stages and shaft/tunnel breakthroughs, is strongly recommended to verify ground conditions and confirm that the excavation proceeds safely. It should be reiterated that a tunneling specialist should be engaged.

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. Small rock slides (<5m3) from around rock excavation perimeters.
B. Large rock slides (<20m3) from around rock excavation perimeters.

A qualitative assessment of risk to life and property related to these hazards is presented in **Tables 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 **6.00×10^{-3}** for any person in the main dwelling, while the Risk to Property was considered to be '**Very High**'. Hazard B was estimated to have a Risk to Life of up to **6.00×10^{-4}** for a single person in the dwelling onsite, while the Risk to Property was considered to be '**High**'. Hazard C was estimated to have a Risk to Life of up to **1.05×10^{-4}** for a single person in the dwelling onsite, while the Risk to Property was considered to be '**Low**'.

Based on the results in **Table B**, The Risk to Property for the site or surrounding properties is evaluated to be up to '**very high**', and therefore requires investigation, planning and implementation of treatment options to reduce the risk to low. The assessments were based on excavations with no support or planning, using ground conditions anticipated in adjacent properties. Provided the recommendations of this report are implemented, including further investigation, detailed geotechnical monitoring and the installation of engineered support systems, the likelihood of any failure becomes 'Unlikely' and as such the consequences reduce with risk becoming 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.

6.3. Design & Construction Recommendations:

6.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings on bedrock
Type of Footing	Strip/Pad or Slab at base of excavation
Sub-grade material and Maximum Allowable Bearing Capacity	Low strength sandstone: 1000kPa
Site sub-soil classification as per *Structural design*	B_e – Rock Site

actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia	
Remarks: All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify the bearing capacities provided above are achieved and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

6.3.2. Excavation:	
Depth of Excavation	Up to 5 m deep for main dwelling
	Up to 18.5 m deep for tunnel excavations.
	Up to 11 m deep for the new garage.
Type of Material to be Excavated	Fill (very minor)
	Residual soils (very minor)
	Low strength sandstone
VLS = Very low strength, LS = Low strength, MS = Medium strength, HS = High strength	

Guidelines for batter slopes for general information are tabulated below:

Material	Safe Batter Slope (degrees)	
	Short Term/ Temporary	Long Term/ Permanent
Fill	36	30
Residual soils	45	30
Low strength bedrock	Vertical*	70 – potentially vertical*

*Dependent on assessment by engineering geologist

Remarks:

Seepage at the bedrock surface or along defects in the soil/rock can occasionally 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. It is anticipated that these support measures will form either a strutted wall, or gravity/ cantilevered retaining wall.

Equipment for Excavation	Fill/ Residual soil	Excavator with bucket
	LS bedrock	Rock hammer and saw

Remarks:

Based on previous testing of ground vibrations created by various rock excavation equipment within low strength bedrock, to maintain a vibration level below 5mm/s PPV the below hammer weights and buffer distances are required:	

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
≥1 tonne	Up to 20.00m

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 commencement of rock excavation works, where ≥250kg 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 and the saw cuts will provide a slight increase in buffer distance for use of rock hammers.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Neighbouring residential dwellings = 5mm/s
Vibration Calibration Tests Required	If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed.
Full time vibration Monitoring Required	Yes, based upon the anticipated size of the excavations and proximity to adjacent structures.
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • At 1.50m depth intervals of unsupported excavation. • At 1.50m depth/ length intervals within the tunnel/ lift. • At completion of the excavation. • Where ground conditions are exposed that differ to those expected.
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 15m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client

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

6.3.3. Retaining Structures:	
Required	In the absence of cored borehole data, the depth at which the Hawkesbury Sandstone transitions from low to medium or high strength remains uncertain. As such, a conservative approach should be adopted for preliminary design, assuming low strength sandstone extends to the full depth of excavation. Therefore, an allowance for new retaining structures/ strutting should be made. In temporary conditions, the low-strength sandstone may be excavated vertically, provided the material remains intact and free from significant, adversely oriented defects. This approach would facilitate the construction of retaining walls, whether cantilevered or gravity-type. For unsupported permanent excavations, it is recommended that faces be formed at a shallower inclination, approximately 70°, to reduce the risk of instability. This will be confirmed in additional investigation. As previously noted, in the northwest corner of the site, at the location of the proposed 'void', excavation is proposed to extend to the site boundary. Due to the presence of fill or soil in this area, and limitations imposed by the proximity to the boundary, an unsupported open-cut excavation is not currently feasible. Accordingly, it is recommended that a soldier piled wall with concrete panel infills be constructed in this area. The piles will likely need to extend below the base of the excavation to resist lateral earth pressures; however, this will require analysis by the structural engineer. Should further investigation reveal that the sandstone is relatively defect-free and transitions into medium or high strength material at shallow depths, it is

	likely that excavation batters could remain stable at steeper inclinations (vertical), and in some cases permanently unsupported. This is subject to verification during additional ground investigation and excavation observations. Where tunnel excavations are proposed, the selection and design of appropriate temporary and permanent support systems must be undertaken by a qualified tunnelling specialist based on actual ground conditions encountered. Support may include combinations of shotcrete, rock bolts, dowels or other reinforcement systems as dictated by the rock mass quality and defect conditions. An additional ground investigation comprised of rotary cored boreholes is required to characterise the rock mass strength, groundwater regime, weathering profile, and defects to support final retaining wall and tunnel support design.
Types	Post excavation construction – concrete walls, anticipated to be gravity or cantilevered, or strutted excavated faces. 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)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill/ Residual soils	18	$\phi' = 28°$	0.35	0.52	N/A
LS bedrock	23	$\phi' = 38°$	0.10	0.15	300 kPa

Remarks:

In suggesting these parameters, it is assumed that the retaining walls will be fully drained. 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. If required, it is suggested that post excavation retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

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

6.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		None identified, assumed that seepage will be present.
Excavation likely to intersect	Water Table	Possible
	Seepage	Minor expected, on defects and at soil/rock interface
Site Location and Topography		High western side of the intersection of Dick Street and The Drive. The site slopes steeply to very steeply down to the east.
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not possible via absorption to inclination of the site, requires discharge to street.
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site.		

6.4. 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. Undertake additional ground investigation.
2. Review the structural drawings, including the retaining structure/batter slope design and construction methodology, and stormwater system plans for compliance with the recommendations of this report, this will be required for CC to meet condition 11.
3. Conduct excavation inspections as per the recommendations of Section 4.3.2 in this report

4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect the completed development to ensure all retention and stormwater systems are complete and connected and that construction activity has not created any new landslip hazards.

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 complete the certification if it has not been called to site to undertake the required inspections.

7. CONCLUSION:

Based on the architectural drawings provided (Tobias Partners, Name: Preliminary DA Issue for Consultant Coordination, Dated: 22/07/25) and our site investigation, the proposed development will require significant excavation works to accommodate a new multi-storey dwelling, garage structure, tunnel connection, and lift shaft. Excavations of up to approximately 5 m are proposed for the house, up to 11 m for the garage structure at the front of the site, and up to 18.5 m for the lift shaft extending 34 m in length towards the garage.

The site is underlain by Hawkesbury Sandstone, which was encountered in all boreholes at shallow depth (0.20–0.60 m) and is outcropping in the site and adjoining properties. The material was interpreted as low strength sandstone, with localised horizontal bedding defects noted. Minor soil and fill layers (up to 0.40 m thick) were identified in the near surface, though no groundwater table was encountered, and only minor seepage is anticipated along defects. The site is considered generally suitable for the proposed development from a geotechnical perspective, subject to appropriate controls during excavation and construction.

In the absence of cored boreholes, the depth at which the sandstone transitions to medium or high strength remains unknown. For design purposes, conservative assumptions should be made (i.e. low strength to full excavation depth), with vertical or near-vertical temporary excavation faces adopted where feasible, and shallower (e.g. 70°) batters considered where defected rock is encountered or unsupported rock is proposed. In the location of the proposed ‘void’ in the northwest of the site, due to the excavation extending to the site boundary, excavation support will be required, it is currently considered that soldier piled wall with concrete infill will be suitable.

The proposed tunnel and lift shaft are technically feasible in this geological setting. Although CGC does not provide specialist tunnelling design services, tunnelling through Hawkesbury Sandstone has been widely and successfully completed throughout the Sydney region. Published literature highlights the potential for excavation-induced ground movement due to high horizontal stresses within the sandstone. Final tunnel support and staging must be designed by a specialist tunnelling engineer.

Given the scale and complexity of excavation, and the current limited subsurface data, it is strongly recommended that further geotechnical investigation be undertaken. This should include cored boreholes along the tunnel alignment and lift shaft location to better define rock mass strength, defect spacing, and groundwater conditions. These data will support detailed foundation design, excavation support, and tunnelling methodology selection.

Appropriate vibration controls should be in place during rock excavation, including the use of low-energy plant and, real-time vibration monitoring to mitigate stakeholder concerns and reduce the risk of damage claims. Ongoing geotechnical supervision is recommended throughout excavation to verify actual conditions and guide any required adjustments to the construction methodology.

In summary, the site is considered suitable for the proposed development, including the deep excavation and tunnelling components, provided that additional investigation is undertaken.

Prepared By:

Josh Watts
Senior Engineering Geologist

Reviewed By:

Troy Crozier
Principal
MIEAust, CPEng, NER

8. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Colquhoun, G.P. Hughes, K.S. Deyssing, L. Ballard, J.C. Phillips G. Troedson, A.L. Folkes C.B. Fitzherbert J.A. The Geological Survey of New South Wales (GSNSW) Seamless Geology Project Version 2.4, May 2024. State of New South Wales and Department of Regional NSW 2024. Accessed via MinView
3. 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.
4. E. Hoek & J.W. Bray 1981, "Rock Slope Engineering" By The Institution of Mining and Metallurgy, London.
5. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin.

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: -

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

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.

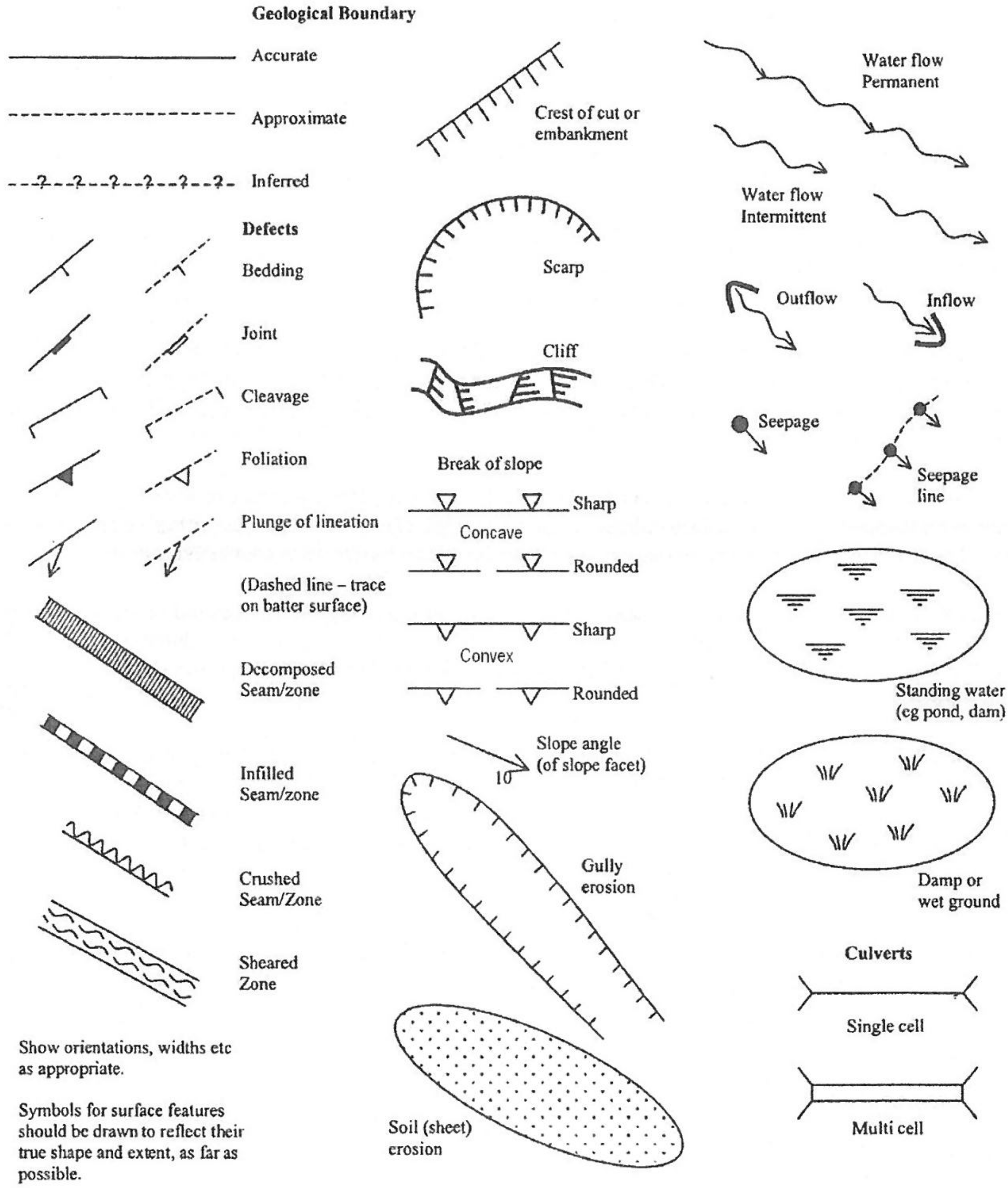
Reproduction of Information for Contractual Purposes

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.

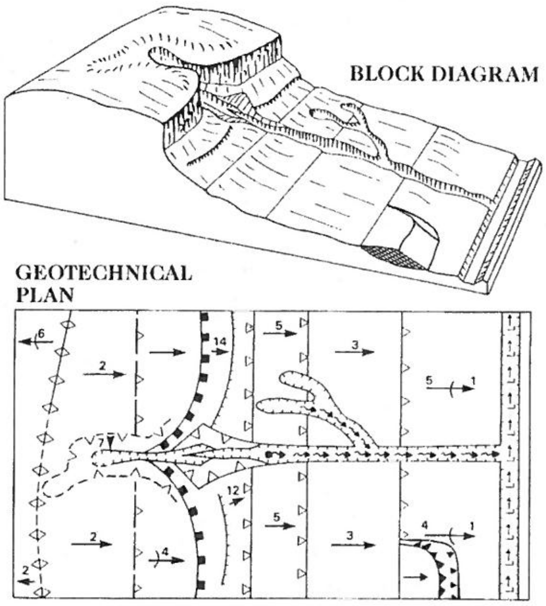
Site Inspection

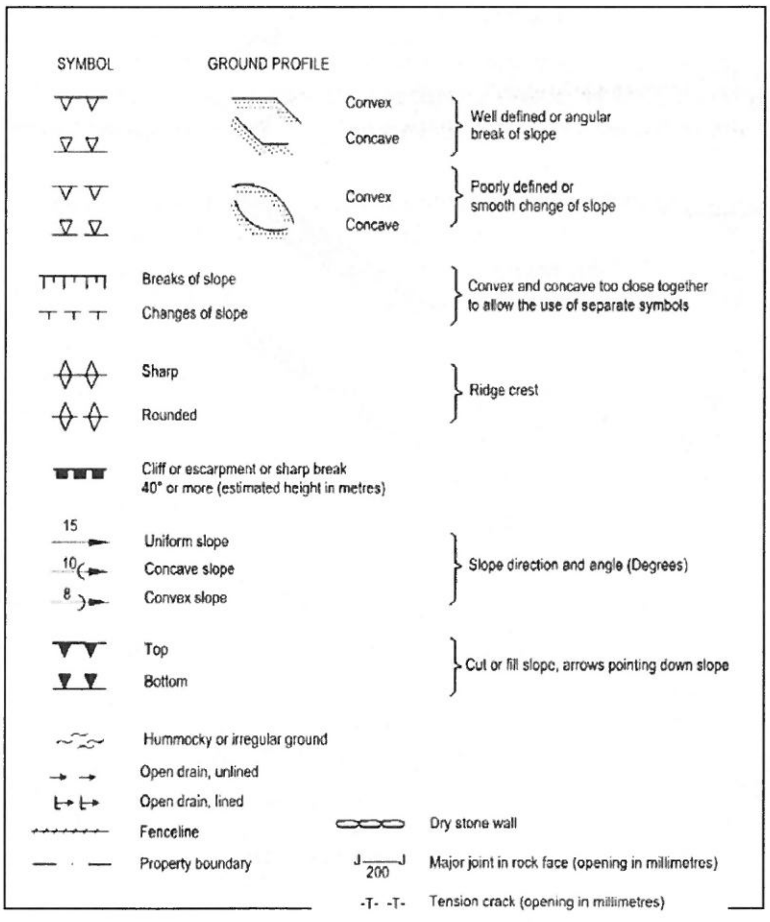
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



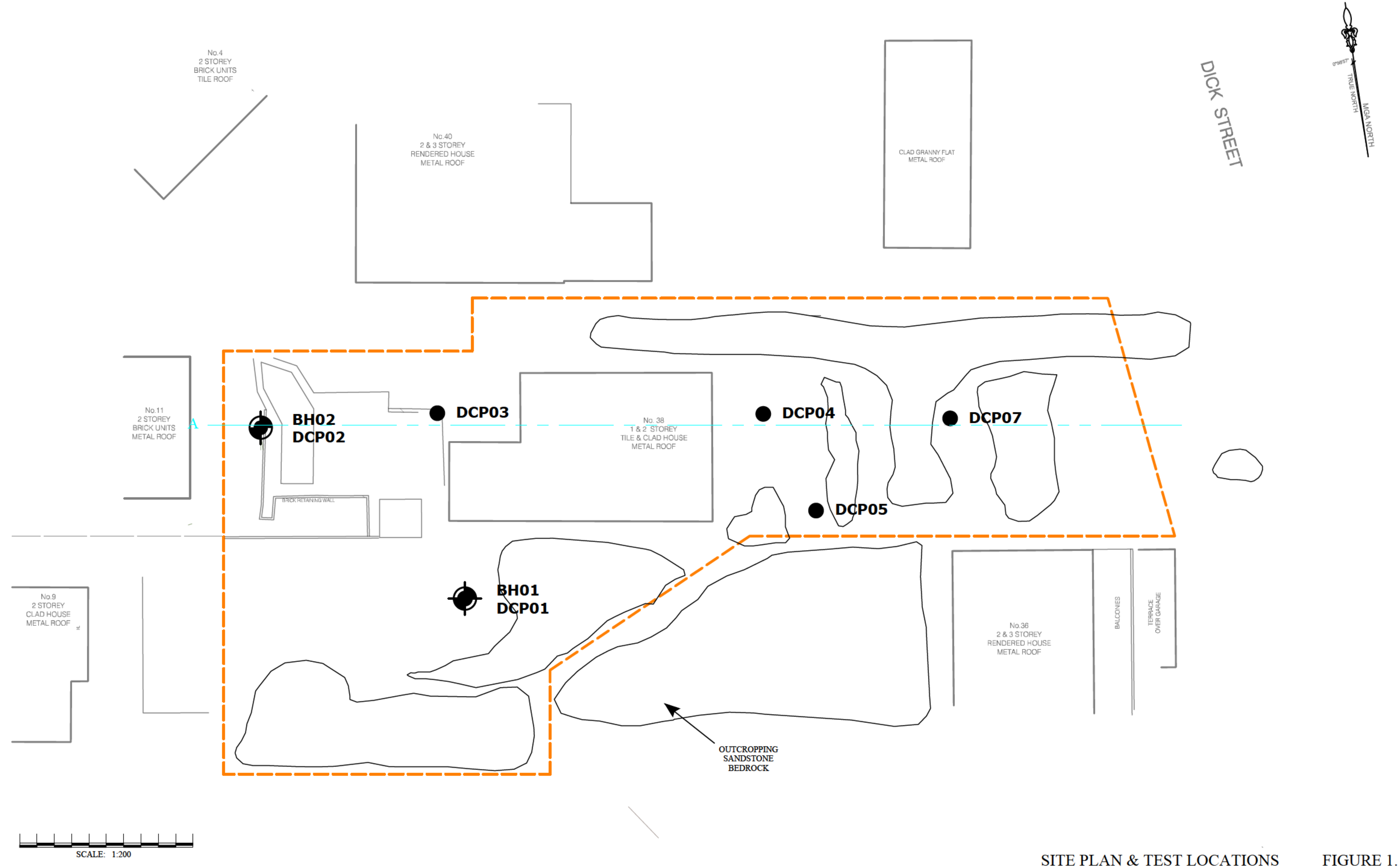
Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).





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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
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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

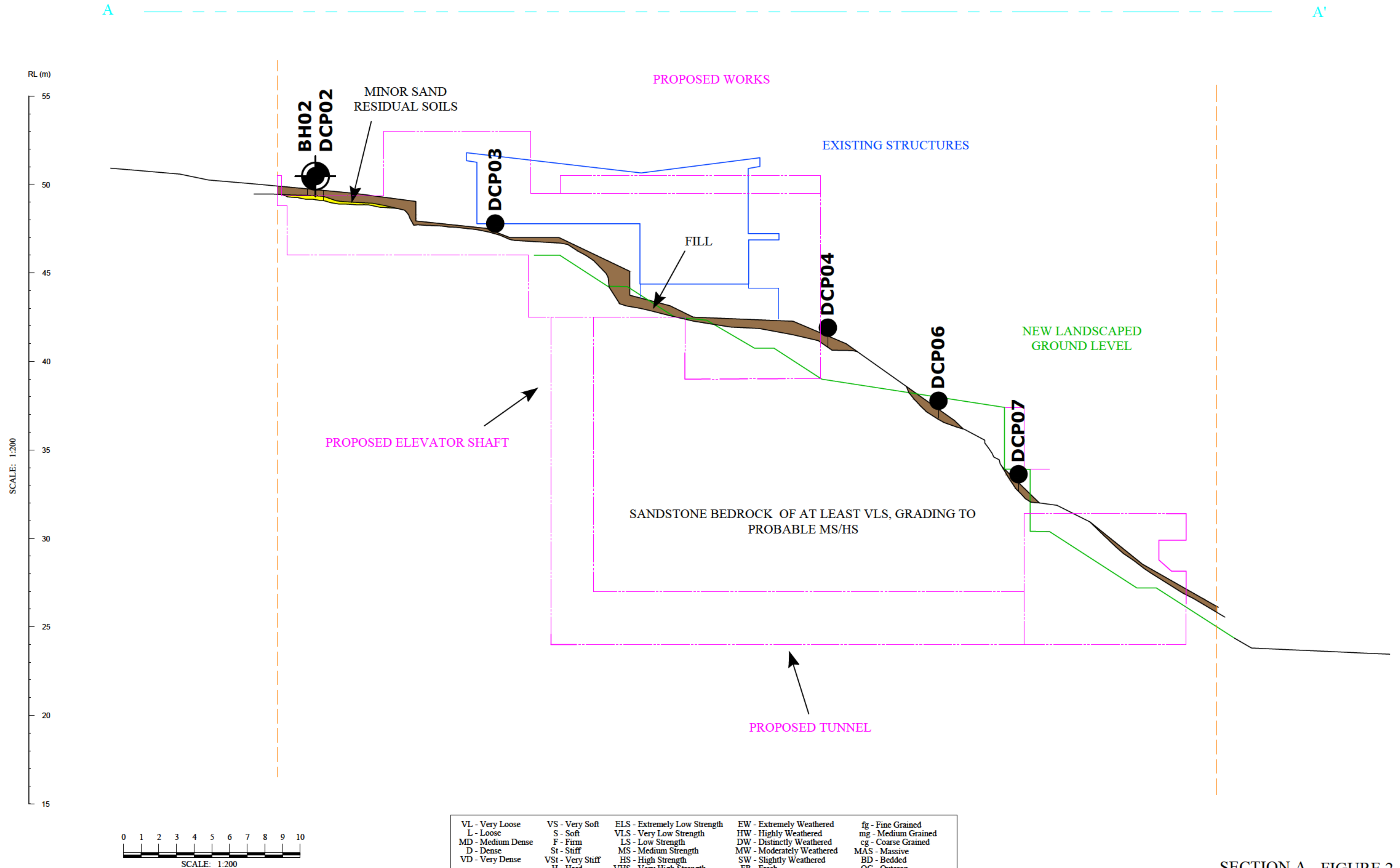
LEGEND

OUTCROPPING SANDSTONE BEDROCK	BH / DCP: AUGER / DYNAMIC CONE PENETROMETER LOCATION	BOUNDARY	A — A' SECTION LINE	DCP: DYNAMIC CONE PENETROMETER

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	07/2025
APPROVED BY:	TMC
DRAWN BY:	JC
PROJECT:	2024-185

PREPARED FOR:
NADIM & MIRANDA DABAN

ADDRESS:
38 THE DRIVE, FRESHWATER



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

SECTION A FIGURE 2

CROZIER
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LEGEND

- EXISTING STRUCTURES
- PROPOSED WORKS
- DCP — DYNAMIC CONE PENETROMETER
- SANDY RESIDUAL SOILS
- SANDSTONE BEDROCK
- PROPOSED LANDSCAPED SURFACE LEVEL
- BOUNDARY
- BH / DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
- A — A' SECTION LINE
- TOPSOIL/FILL

SCALE:	1:200 @ A3
DRAWING:	FIGURE 2
DATE:	06/2025
APPROVED BY:	TMC
DRAWN BY:	JC
PROJECT:	2024-185

PREPARED FOR:
NADIM & MIRANDA DABAN

ADDRESS:
38 THE DRIVE, FRESHWATER

BOREHOLE LOG

CLIENT: **DATE:** 24/06/2025 **BORE No.:** BH01

PROJECT: 38 The Drive **PROJECT No.:** 2024-185 **SHEET:** 1 of 1

LOCATION: Freshwater **SURFACE LEVEL:** TS

Depth (m) 0.00	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
0.20	TS	TOPSOIL: Clayey SAND. Loose, brown, moist. Sand is fine to coarse				
0.25	F	FILL: Sand with trace of gravel Loose, grey to brown, dry. Sand is fine to coarse Gravel is angular to subangular fine to medium sandstone and metal.				
		Borehole refused on sandstone bedrock at 0.25m.				

RIG: N/A DRILLER: AC

METHOD: Hand Auger LOGGED: JW

GROUND WATER OBSERVATIONS: N/A

REMARKS: CHECKED: JW

BOREHOLE LOG

CLIENT:

DATE: 25/06/2025

BORE No.: BH02

PROJECT: **38 The Drive**

PROJECT No.: 24/06/2025

SHEET: 1 of 1

LOCATION: **Freshwater**

SURFACE LEVEL: TS

Depth (m) 0.00	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
0.20	TS	TOPSOIL: Clayey SAND. Loose, brown, moist. Sand is fine to coarse				
0.35	F	FILL: Sand with trace of gravel Loose, grey to brown, dry. Sand is fine to coarse Gravel is angular to subangular fine to medium sandstone and metal.				
0.40	S	SAND with gravel. Loose, grey to brown, dry. Sand is fine to coarse. Gravel is angular to subangular fine to medium sandstone. (RESIDUAL SOIL)				
		Borehole refused on sandstone bedrock at 0.40m				

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: JW

GROUND WATER OBSERVATIONS: N/A

REMARKS:

CHECKED: JW

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Henry and Michelle Motteram | **DATE:** 24/06/2025
PROJECT: 23 Wisdom Road | **PROJECT No.:** 2024-185
LOCATION: Greenwich | **SHEET:** 1 of 1

	Test Location									
Depth (m)	DCP01	DCP02	DCP03	DCP04	DCP05	DCP07				
0.00 - 0.10	-	1	-	1	1	-				
0.10 - 0.20	1	2	6 - B	3	-	-				
0.20 - 0.30	7	3		3	6	-				
0.30 - 0.40	8	3		2	4	-				
0.40 - 0.50	8	5		2	6 - B	7 - B				
0.50 - 0.60	16 - B	15 - B		7 - B						
0.60 - 0.70										
0.70 - 0.80										
0.80 - 0.90										
0.90 - 1.00										
1.00 - 1.10										
1.10 - 1.20										
1.20 - 1.30										
1.30 - 1.40										
1.40 - 1.50										
1.50 - 1.60										
1.60 - 1.70										
1.70 - 1.80										
1.80 - 1.90										
1.90 - 2.00										
2.00 - 2.10										
2.10 - 2.20										
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
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: **(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	Small rock slides (<5m3) from around rock excavation perimeters. - Garage		More dilating heavily fractured bedrock tends to be within the upper zone.	a) Excava ions onsite - 18.5m maximum excavation b) Garage excavation of 11m approx 0.9m from boundary. c) Garage excavation of 11m approx 1.53m from building. d) Garage excavation of 9.5m approx 3.2m from building. e) Garage excavation of 9.5m approx 1.15m from boundary. f) Dwelling excava ion of 3.5m approx 2.25m from the building. g) Dwelling excavation of 3.5m approx 0 9m from boundary. h) Dwelling excavation of 5m approx 4.08m to the building. i) Dwelling excavation of 5m approx 3.05m to the boundary. j) Dwelling excavation of 5m approx 8.2m to the building. k) Dwelling excavation of 5m approx 3.05m to he boundary. l) Dwelling excavation of 5m approx 9.00m to the building. m) Dwelling excavation of 5m approx 1.10m to the boundary. n) Dwelling excavation of 5m approx 2.00m to the building. o) Dwelling excavation of 5m approx 1.10m to the boundary. p) Dwelling excavation of 5m approx 0.9m to the boundary. q) Dwelling excavation of 5m approx 4.80m to the building.		a) Person in the building 18hr/day average. b) Person in garden 1 hour a day. c) Person in the building 18hr/day average. d) Person in the building 18hr/day average. e) Person in garden 1 hour a day. f) Person in the building 18hr/day average. g) Person in garden 1 hour a day. h) Person in the building 18hr/day average. i) Person in garden 1 hour a day. j) Person in the building 18hr/day average. k) Person in garden 1 hour a day. l) Person in the building 18hr/day average. m) Person in garden 1 hour a day. n) Person in the building 18hr/day average. o) Person in garden 1 hour a day. p) Person in garden 1 hour a day. q) Person in the building 18hr/day average.	a) Likely to not evacuate. b) Unlikely to not evacuate. c) Likely to not evacuate. d) Likely to not evacuate. e) Unlikely to not evacuate. f) Likely to not evacuate. g) Unlikely to not evacuate. h) Likely to not evacuate. i) Unlikely to not evacuate. j) Likely to not evacuate. k) Unlikely to not evacuate. l) Likely to not evacuate. m) Unlikely to not evacuate. n) Likely to not evacuate. o) Unlikely to not evacuate. p) Unlikely to not evacuate. q) Likely to not evacuate.	a) Person in house, potentially buried b) Person in open space,very unlikley to be buried. c) Person in house, potentially buried d) Person in house, potentially buried e) Person in open space,very unlikley to be buried. f) Person in house, potentially buried g) Person in open space,very unlikley to be buried. h) Person in house, potentially buried i) Person in open space,very unlikley to be buried. j) Person in house, potentially buried. k) Person in open space,very unlikley to be buried. l) Person in house, potentially buried m) Person in open space,very unlikley to be buried. n) Person in house, potentially buried. o) Person in open space,very unlikley to be buried. p) Person in open space,very unlikley to be buried. q) Person in house, potentially buried.	
			Possible	Prob. of Impact	Impacted				
		a) The site dwelling and garage	0.01	1.00	0.80	0.7500	1.00	1.00	6.00E-03
		b) Garden/ patio of No. 36 The Drive Residential Property (southern neighbour)	0.01	0.75	0.05	0.0417	0.25	0.50	1.95E-06
		c) Dwelling of No. 36 The Drive (southern neighbour)	0.01	0.50	0.30	0.7500	1.00	0.75	8.44E-04
		d) Granny Flat of No. 6 Coast View Place. (northern neighbour)	0.01	0.30	0.30	0.7500	1.00	0.75	5.06E-04
		e) Garden of granny flat No. 6 Coast View Plac (northern neighbour)	0.01	0.50	0.05	0.0417	0.25	0.50	1.30E-06
		f) Dwelling of No. 6 Coast View Plac (northern neighbour)	0.01	0.50	0.10	0.7500	1.00	0.75	2.81E-04
		g) Garden/ patio of No. 6 Coast View Plac (northern neighbour)	0.01	0.50	0.05	0.0417	0.25	0.25	6.51E-07
		h) Dwelling of no.1 Seddon Hill Road (southern neighbour)	0.01	0.10	0.05	0.7500	1.00	0.75	2.81E-05
		i) Garden of no.1 Seddon Hill Road (southern neighbour)	0.01	0.20	0.05	0.0417	0.25	0.25	2.60E-07
		j) Dwelling of no. 3 Seddon Hill Road (southern neighbour)	0.01	0.00	0.00	0.7500	1.00	0.75	5.63E-10
		k) Garden of no.3 Seddon Hill Road (southern neighbour)	0.01	0.10	0.05	0.0417	0.25	0.25	1.30E-07
		l) Dwelling of no. 9 Lodge Lane (western neighbour)	0.01	0.00	0.00	0.7500	1.00	0.75	5.63E-11
		m) Garden of no.9 Lodge Lane (western neighbour)	0.01	0.50	0.05	0.0417	0.25	0.25	6.51E-07
		n) Dwelling of no.11 Lodge Lane (western neighbour)	0.01	0.40	0.15	0.7500	1.00	0.75	3.38E-04
		o) Garden of no.11 Lodge Lane (western neighbour)	0.01	0.50	0.05	0.0417	0.25	0.25	6.51E-07
		p) Garden of no.11 Lodge Lane (western neighour)	0.01	0.75	0.05	0.0417	0.25	0.25	9.77E-07
		q) Dwelling of No. 40 The Drive (northern neighbour)	0.01	0.05	0.01	0.7500	1.00	0.75	2.81E-06

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
B	Large rock slides (<20m3) from around rock excavation perimeters. - Garage		More dilating heavily fractured bedrock tends to be within the upper zone.	a) Excava ions onsite - 18.5m maximum excavation b) Garage excavation of 11m approx 0.9m from boundary. c) Garage excavation of 11m approx 1.53m from building. d) Garage excavation of 9.5m approx 3.2m from building. e) Garage excavation of 9.5m approx 1.15m from boundary. f) Dwelling excava ion of 3.5m approx 2.25m from the building. g) Dwelling excavation of 3.5m approx 0 9m from boundary. h) Dwelling excavation of 5m approx 4.08m to the building. i) Dwelling excavation of 5m approx 3.05m to the boundary. j) Dwelling excavation of 5m approx 8.2m to the building. k) Dwelling excavation of 5m approx 3.05m to he boundary. l) Dwelling excavation of 5m approx 9.00m to the building. m) Dwelling excavation of 5m approx 1.10m to the boundary. n) Dwelling excavation of 5m approx 2.00m to the building. o) Dwelling excavation of 5m approx 1.10m to the boundary. p) Dwelling excavation of 5m approx 0.9m to the boundary. q) Dwelling excavation of 5m approx 4.80m to the building.		a) Person in the building 18hr/day average. b) Person in garden 1 hour a day. c) Person in the building 18hr/day average. d) Person in the building 18hr/day average. e) Person in garden 1 hour a day. f) Person in the building 18hr/day average. g) Person in garden 1 hour a day. h) Person in the building 18hr/day average. i) Person in garden 1 hour a day. j) Person in the building 18hr/day average. k) Person in garden 1 hour a day. l) Person in the building 18hr/day average. m) Person in garden 1 hour a day. n) Person in the building 18hr/day average. o) Person in garden 1 hour a day. p) Person in garden 1 hour a day. q) Person in the building 18hr/day average.	a) Likely to not evacuate. b) Unlikely to not evacuate. c) Likely to not evacuate. d) Likely to not evacuate. e) Unlikely to not evacuate. f) Likely to not evacuate. g) Unlikely to not evacuate. h) Likely to not evacuate. i) Unlikely to not evacuate. j) Likely to not evacuate. k) Unlikely to not evacuate. l) Likely to not evacuate. m) Unlikely to not evacuate. n) Likely to not evacuate. o) Unlikely to not evacuate. p) Unlikely to not evacuate. q) Likely to not evacuate.	a) Person in house, potentially buried b) Person in open space,very unlikey to be buried. c) Person in house, potentially buried d) Person in house, potentially buried e) Person in open space,very unlikey to be buried. f) Person in house, potentially buried g) Person in open space,very unlikey to be buried. h) Person in house, potentially buried i) Person in open space,very unlikey to be buried. j) Person in house, potentially buried. k) Person in open space,very unlikey to be buried. l) Person in house, potentially buried m) Person in open space,very unlikey to be buried. n) Person in house, potentially buried. o) Person in open space,very unlikey to be buried. p) Person in open space,very unlikey to be buried. q) Person in house, potentially buried.	
			Possible	Prob. of Impact	Impacted				
		a) The site dwelling and garage	0.001	1.00	0.80	0.7500	1.00	1.00	6.00E-04
		b) Garden/ patio of No. 36 The Drive Residential Property (southern neighbour)	0.001	0.75	0.05	0.0417	0.25	0.50	1.95E-07
		c) Dwelling of No. 36 The Drive (southern neighbour)	0.001	0.75	0.40	0.7500	1.00	0.75	1.69E-04
		d) Granny Flat of No. 6 Coast View Place. (northern neighbour)	0.001	0.75	0.40	0.7500	1.00	0.75	1.69E-04
		e) Garden of granny flat No. 6 Coast View Plac (northern neighbour)	0.001	0.75	0.05	0.0417	0.25	0.50	1.95E-07
		f) Dwelling of No. 6 Coast View Plac (northern neighbour)	0.0001	0.75	0.20	0.7500	1.00	0.75	8.44E-06
		g) Garden/ patio of No. 6 Coast View Plac (northern neighbour)	0.0001	0.60	0.05	0.0417	0.25	0.25	7.81E-09
		h) Dwelling of no.1 Seddon Hill Road (southern neighbour)	0.0001	0.20	0.15	0.7500	1.00	0.75	1.69E-06
		i) Garden of no.1 Seddon Hill Road (southern neighbour)	0.0001	0.30	0.05	0.0417	0.25	0.25	3.91E-09
		j) Dwelling of no. 3 Seddon Hill Road (southern neighbour)	0.0001	0.10	0.10	0.7500	1.00	0.75	5.63E-07
		k) Garden of no.3 Seddon Hill Road (southern neighbour)	0.0001	0.20	0.05	0.0417	0.25	0.25	2.60E-09
		l) Dwelling of no. 9 Lodge Lane (western neighbour)	0.0001	0.10	0.10	0.7500	1.00	0.75	5.63E-07
		m) Garden of no.9 Lodge Lane (western neighbour)	0.0001	0.60	0.05	0.0417	0.25	0.25	7.81E-09
		n) Dwelling of no.11 Lodge Lane (western neighbour)	0.0001	0.50	0.15	0.7500	1.00	0.75	4.22E-06
		o) Garden of no.11 Lodge Lane (western neighbour)	0.0001	0.65	0.05	0.0417	0.25	0.25	8.46E-09
		p) Garden of no.11 Lodge Lane (western neighour)	0.0001	0.75	0.05	0.0417	0.25	0.25	9.77E-09
		q) Dwelling of No. 40 The Drive (northern neighbour)	0.0001	0.20	0.15	0.7500	1.00	0.75	1.69E-06

<u>TABLE : B</u>

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Small rock slides (<5m3) from around rock excavation perimeters.	a) The site dwelling and garage	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.	Very High
		b) Garden/ patio of No. 36 The Drive Residential Property (southern neighbour)	Possible	The event could occur under adverse conditions 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
		c) Dwelling of No. 36 The Drive (southern neighbour)	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
		d) Granny Flat of No. 6 Coast View Place. (northern neighbour)	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
		e) Garden of granny flat No. 6 Coast View Place (northern neighbour)	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
		f) Dwelling of No. 6 Coast View Place (northern neighbour)	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
		g) Garden/ patio of No. 6 Coast View Place (northern neighbour)	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
		h) Dwelling of no.1 Seddon Hill Road (southern neighbour)	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
		i) Garden of no.1 Seddon Hill Road (southern neighbour)	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
		j) Dwelling of no. 3 Seddon Hill Road (southern neighbour)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very low
		k) Garden of no.3 Seddon Hill Road (southern neighbour)	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
		l) Dwelling of no. 9 Lodge Lane (western neighbour)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very low
		m) Garden of no.9 Lodge Lane (western neighbour)	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
		n) Dwelling of no.11 Lodge Lane (western neighbour)	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
		o) Garden of no.11 Lodge Lane (western neighbour)	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
		p) Garden of no.11 Lodge Lane (western neighour)	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
		q) Dwelling of No. 40 The Drive (northern neighbour)	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

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
B	Large rock slides (<20m3) from around rock excavation perimeters.	a) The site dwelling and garage	Unlikely	The event might occur under very adverse circumstances 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) Garden/ patio of No. 36 The Drive Residential Property (southern neighbour)	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
		c) Dwelling of No. 36 The Drive (southern neighbour)	Unlikely	The event might occur under very adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Moderate
		d) Granny Flat of No. 6 Coast View Place. (northern neighbour)	Unlikely	The event might occur under very adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Moderate
		e) Garden of granny flat No. 6 Coast View Place (northern neighbour)	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
		f) Dwelling of No. 6 Coast View Place (northern neighbour)	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
		g) Garden/ patio of No. 6 Coast View Place (northern neighbour)	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
		h) Dwelling of no.1 Seddon Hill Road (southern neighbour)	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
		i) Garden of no.1 Seddon Hill Road (southern neighbour)	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
		j) Dwelling of no. 3 Seddon Hill Road (southern neighbour)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very low
		k) Garden of no.3 Seddon Hill Road (southern neighbour)	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
		l) Dwelling of no. 9 Lodge Lane (western neighbour)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Very low
		m) Garden of no.9 Lodge Lane (western neighbour)	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
		n) Dwelling of no.11 Lodge Lane (western neighbour)	Unlikely	The event might occur under very 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.	Low
		o) Garden of no.11 Lodge Lane (western neighbour)	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
		p) Garden of no.11 Lodge Lane (western neighour)	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
		q) Dwelling of No. 40 The Drive (northern neighbour)	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

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.

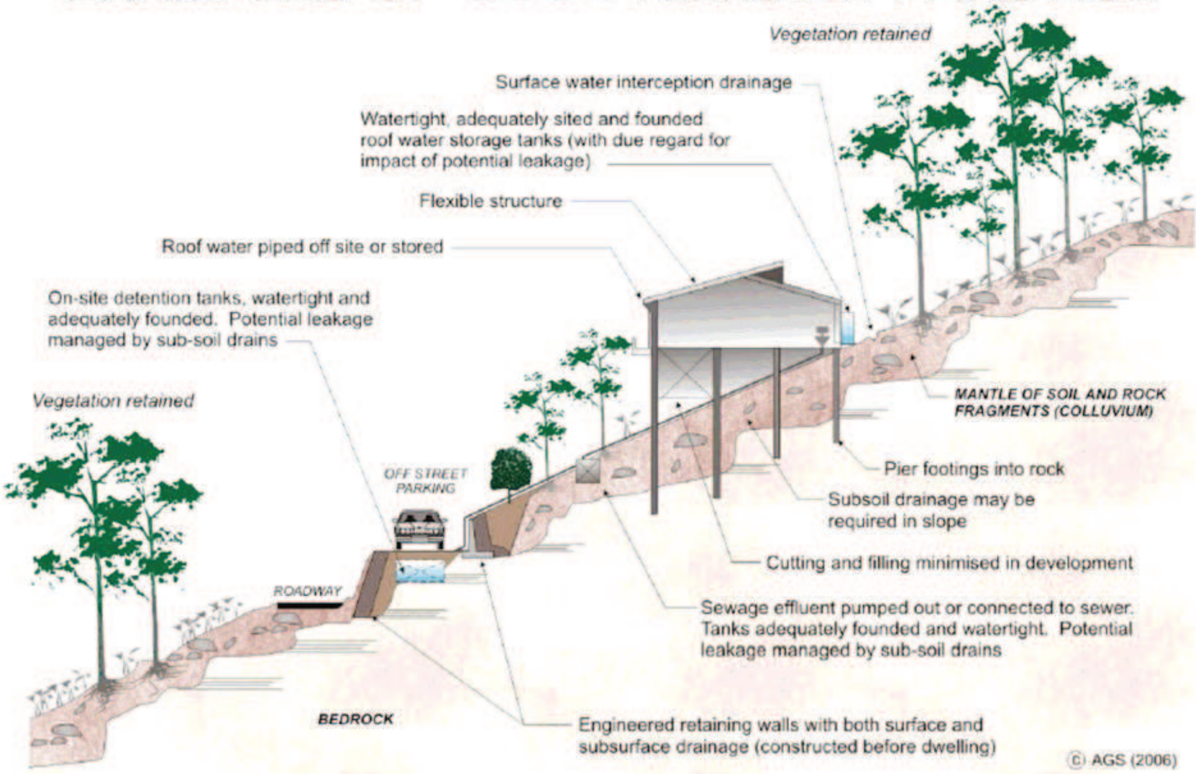
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

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