Douglas Partners

Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on Geotechnical Assessment

Proposed New Residence
50 Minkara Road, Bayview

Prepared for
Ms Miranda Wong

Project 86886.00
October 2019

Douglas Partners
Geotechnics / Environment / Groundwater

Document History

Document details

Project No.	86886.00	Document No.	R.001.Rev0
Document title	Report on Geotechnical Assessment Proposed New Residence		
Site address	50 Minkara Road, Bayview		
Report prepared for	Ms Miranda Wong		
File name	86886.00.R.001.Rev0		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Fiona MacGregor	Bruce McPherson	14 October 2019

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1		Robert Harrison

The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
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bsi. ISO 9001 Quality Management
FS 604853

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1. Introduction

This report presents the results of a geotechnical assessment undertaken for a proposed new residence at 50 Minkara Road, Bayview. The investigation was commissioned by Robert Harrison, architect, on behalf of the owner Ms Miranda Wong, and was undertaken in accordance with Douglas Partners' proposal 190773 dated 25 July 2019.

It is understood that the existing house and carport on the site will be demolished and a new two level residence, a garage and a swimming pool are to be constructed in the same area as the existing house.

The aim of the assessment was to assess the stability of the site and provide geotechnical recommendations for design and construction of the proposed development. The assessment comprised a walk-over inspection of the site by a senior engineering geologist on 3 September 2019.

A previous assessment of the site was undertaken by Douglas Partners Pty Ltd (DP) in 2004 prior to alterations and additions to the existing house. That assessment included mapping of the site by a senior engineering geologist, dynamic penetrometer testing and the drilling of two hand augered boreholes to enable permeability testing. The results of the previous field work have been used in this assessment, together with the recent inspection.

2. Site Description and Regional Geology

The site comprises an approximately trapezoidal shaped allotment located on the lower eastern side of Minkara Road, Bayview. It has a frontage to Minkara Road of about 23 m, an average length of about 110 m and an eastern boundary length of about 60 m. The site area is about 4400 m^2.

The site slopes very steeply towards the east, falling from an elevation of about RL 116 m along the western boundary to about RL 86 m about two-thirds of the way down the slope, at the top of a steep cliff face. The lower third of the site has not been surveyed due to its inaccessibility.

For descriptive purposes the site can be divided into three sections, namely:

- the upper western 50 m of the site;
- the central 40 m of the site; and
- the lower, eastern section of the site, which is located below a shear cliff face about 6 m to 8 m in height.

The upper and central sections of the site are separated by a bouldery cliff line of about 2.5 m to 4 m in height. The upper section slopes at an average angle of 12 degrees while the central section is steeper with an average slope of about 24 degrees.

The lower cliff line is vertical, of about 6 m to 8 m height and continues laterally into the properties to the north and south. A visual inspection of the lower section of the site, observed from the top of the cliff, suggests that the lower section slopes towards the east at an angle of about 25 degrees and comprises colluvium or slopewash soils with some large sandstone floaters which were previously part of the cliff.

Sandstone bedrock outcrop is present at the break of slope between the upper and central sections, in the excavation on the southern and western sides of the garage and in the major cliff line towards the eastern boundary. There are a number of smaller sandstone boulders and floaters within the soils of the upper section of the site and some large sandstone floaters and detached joint blocks associated with the break between the upper and central sections, as well as on the slope of the central section of the site.

The existing residence has been constructed on the eastern part of the upper section of the site and generally appears to be founded on sandstone bedrock.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Hawkesbury Sandstone, with the contact with the underlying Newport Formation (Narrabeen Group) rocks just to the east and downslope of the site. Both rock formations are of Triassic Age. The Hawkesbury Sandstone typically comprises medium to very coarse grained quartz sandstone with minor shale and laminite lenses. The underlying Newport Formation rocks typically comprise interbedded laminite, shale and lithic sandstone.

Additional details of the site are shown in Photos 1 to 6 in Appendix A.

3. Field Work Methods

The original field work on the site was undertaken in June 2004 and comprised:

- A site walkover inspection and mapping by a senior engineering geologist;
- Nine dynamic cone penetrometer (DCP) tests taken to refusal at depths of between 0.3 m and 1.0 m;
- Two hand auger bores drilled at locations 3 and 5 to depths of 0.5 m and 1.05 m to allow in-situ testing of permeability; and
- Two constant head permeability tests undertaken in Bores 3 and 5.

The locations of the tests, site features, and areas of rock outcrop or rock floaters are shown on Drawing 1 in Appendix B.

The surface levels at the test locations, relative to Australian Height Datum (m AHD), were interpolated from the provided site survey plan. These levels are shown on the DCP test results sheet and the borehole logs given in Appendix C.

In September 2019 a second walkover inspection was undertaken by a senior engineering geologist. That inspection confirmed that there had been no substantial changes to the geotechnical features of the property since the previous inspection.

4. Field Work Results

The results of the dynamic penetrometer tests, the borehole logs and the permeability test results are given in Appendix C. These results have been used in combination with the site inspection to prepare an interpreted geological profile down the site, which is given as Drawing 2 in Appendix B.

4.1 Site Observations

The principal observations made during the site inspection are given below:

- the site slopes steeply to the east, down from the Minkara Road boundary;
- the upper western section has an average slope angle of about 12 degrees. There are some sandstone floaters or boulders present within a silty sand soil. Borehole 3 was drilled in this area and intersected silty gravelly sand to a depth of 0.5 m, before refusing on sandstone (either bedrock or a boulder);
- a septic tank is located within the upper section of the site;
- there has been some settlement of the concrete driveway slab but no significant separation between adjoining sections;
- the existing residence is of brick and tile construction and is located on the eastern portion of the upper section immediately above a break in slope where sandstone bedrock outcrops;
- sandstone is present in the excavation on the southern and western sides of the garage, as well as in cut along the western side of the residence in the undercroft area below the suspended driveway/carport slab. The existing residence appears to be sound with no significant cracking observed;
- minor cracking of the lower level, concrete verandah slab was observed parallel to the eastern wall of the residence. The north eastern corner of the brick wall supporting the slab appears to be founded on a detached sandstone bedding slab immediately overlying sandstone bedrock whilst the southern end of the brick wall appears to found on either filling or colluvium above the level of bedrock. Some settlement beneath the outer edge of the concrete slab was noted;
- sandstone outcrop is present in the break of slope between the upper and central sections. The bedrock comprises medium and high strength sandstone with some open and sand filled joints observed in the outcrop with the following approximate orientations:
  - strike 010 to 030 degrees (relative to magnetic north) with dips of 90 degrees ± 15 degrees
  - strike 090 to 110 degrees with dips of 90 degrees ± 15 degrees;
- sandstone floaters are present both on the outcrop (as detached blocks or slabs) and on the slope of the central section of the site below;
- some seepage was noted along a bedding plane within the sandstone outcrop (Photo 4);
- the central section of the site slopes at an average angle of about 24 degrees and there are numerous sandstone floaters on and within the soil profile;
- across the central section there are signs of past rough access tracks which are overgrown. In some steeper sections there are indications of soil creep and minor erosion;
- at the north-east corner of the central section surface runoff from No. 52 Minkara Road enters the site and then discharges directly over the cliff line onto the lower part of the site;

- there was no evidence of major landslip or gross instability observed on the upper western or central sections of the site;
- the presence of a large number of floaters and detached blocks on the slope suggests that there have been some rock falls from the cliffs and some blocks of rock have become detached from the underlying bedrock over geological time, that is, over millions of years; and
- there was some evidence of minor downslope soil creep on the steeper central section of the site and the sandy colluvial soils are probably highly erodible.

4.2 Boreholes and Penetrometer Tests

Two boreholes (Bores 3 and 5) were drilled using hand equipment at the locations shown on Drawing 1. The subsurface conditions encountered in the boreholes are given in detail in Appendix C, together with the results of the dynamic penetrometer tests. The interpreted geological conditions on the site are summarised below.

The upper, western section of the site is underlain by sandy colluvial soils with numerous sandstone floaters overlying medium strength sandstone and highly weathered siltstone, as exposed in the excavated cut behind the garage. Boreholes drilled on the neighbouring property confirmed bedrock, across the equivalent part of the site, at depths of 0.5 m to 1.5 m.

Across the central section of the site the soil profile comprised silty sand and sandy clay colluvial soils. Again a cored borehole at a similar elevation on a neighbouring property found sandy soil and sandstone boulders to 1.5 m to 2.5 m depth and then in situ bedrock.

The inferred geotechnical model of the subsurface conditions across the site is illustrated on Drawing 2 and comprises silty sand and sandy clay colluvium with numerous included boulders and floaters, possibly over a thin layer of residual soil, then mostly sandstone bedrock with some highly weathered siltstone layers.

Groundwater was not encountered in any of the tests but some seepage was observed on the cliff face between the upper and central sections of the site.

4.3 Permeability Tests

Tests were undertaken in Bores 3 and 5 to assess the soil permeability using the Constant Head method outlined in Appendix G of AS1547:2012 (On-site domestic waste water management).

The test result sheets are attached in Appendix C and indicate soil permeabilities (or hydraulic conductivities) of:

- 1.8 m per day at Bore 3 - in the silty gravelly sand colluvial soils on the upper part of the site, and
- 0.2 m per day at Bore 5 - in the sandy clay colluvial soils on the central part of the site.

5. Proposed Development

The proposed development involves demolition of the existing house and carport and construction of a new two level residence, garage and swimming pool in essentially the same location as the existing house.

The proposed development will require minor additional excavation below the back of the proposed house (to a maximum depth of 1.5 m), construction of some low retaining walls, and construction of a new swimming pool and viewing deck, potentially extending out over the break in slope between the upper and central sections of the site.

Further details of the proposed works are given on the architectural drawings prepared for the property by Robert Harrison.

6. Comments

6.1 Interpreted Geological Model

The interpreted geological profile down the site is shown on Drawing 2 in Appendix B.

Basically, the site is underlain by shallow sandstone bedrock, with some minor siltstone bands, which is likely to step down the site in a series of small cliffs or near-vertical steps with colluvial soil and boulders accumulating on the benches between the cliffs.

The bedrock is likely to be mainly medium to high strength sandstone with some lower strength bands. Many large detached blocks of sandstone (floaters) were observed on the ground surface.

The main joints in the sandstone are likely to be two sets of near vertical joints oriented just east of north and just south of east, that is, approximately parallel to the site boundaries.

Seepage was observed out of one of the rock faces and it is likely that there is always some gradual seepage along the top of the rock and along bedding planes within the rock. This seepage is likely to increase after periods of heavy rainfall. The permanent groundwater table is likely to be a subdued reflection of the surface topography, probably located well below the proposed excavation levels.

6.2 Design Constraints

Most of the site is underlain by Hawkesbury Sandstone bedrock at relatively shallow depth. Naturally occurring slopes on Hawkesbury Sandstone typically step down a series of cliffs and benches. These slopes usually have a low risk of instability, with the main mechanism for slope failures being undercutting of the cliff faces by weathering along softer seams and blocks of rock becoming detached from the bedrock and eventually sliding or rolling down the slope.

It is recommended that all the proposed structures be founded on the sandstone bedrock.

In some areas it may be difficult to determine whether the sandstone is in-situ bedrock or a very large detached block. Depending on the size and position of the block it may be possible to found some structures on very large stable blocks using lower bearing pressures rather than removing them, but in such a case there is a risk that the detached block will move slightly under load which could lead to cracking of the structure. In general floaters or detached blocks will need to be trimmed where they overlap excavations or footings locations.

While the site is mostly underlain by shallow Hawkesbury Sandstone, it is located on a very steep slope with downslope properties to the east underlain by the Narrabeen Formation, which tends to be more susceptible to slope instability and usually has thicker layers of colluvial soils which are only marginally stable. Accordingly, control of stormwater run-off from the site is very important in reducing the risk of instability on the downslope properties.

6.3 Footings

Sandstone bedrock is at shallow depths below the proposed new house and it is recommended that the new house be uniformly founded on sandstone bedrock. While it is expected that most of the sandstone will be medium or high strength, there may be some layers which are only low strength, which will have a reduced bearing capacity.

Footings founded on sandstone bedrock may be designed using the values given in Table 1. For bored piles, if required, shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the shaft adhesion values for compression in Table 1.

Table 1: Recommended Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure (Serviceability)		Maximum Ultimate Pressure		Young's Modulus E (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
Low Strength Sandstone	1000	100	4,000	250	100
Medium Strength Sandstone	3500	350	20,000	800	350

While most of the sandstone bedrock is expected to be medium strength or better, there may be some lower strength layers present. It is suggested therefore that the footings are initially designed using the lower bearing pressure and if inspections indicate that individual footings are founded on higher strength sandstone then the footing sizes may potentially be reduced.

Footings (i.e. pads or piles) founded on the edge or within the zone of influence of vertical rock excavations or cliff faces, should be inspected to assess whether there are adversely oriented joints affecting the footings.

Generally the allowable bearing pressure for footings founded near the edge of vertical rock faces on medium to high and high strength sandstone (or stronger) should be limited to about 1,000 kPa. If adverse jointing is apparent in the rock face below the footings, then stabilisation using rock bolts or anchors and underpinning may be required. Alternatively, the footings may be taken down to below the zone of influence of the vertical face, in which case there would be no need to reduce the bearing pressure.

Foundations proportioned on the basis of the allowable bearing pressures in Table 1 would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent footings expected to be less than half of this value.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters.

6.4 Excavations

It is expected that the additional excavation required for the rear of the lower part of the house will be through medium and high strength sandstone.

Competent Hawkesbury Sandstone can stand vertically unsupported, provided that there are no adversely oriented joints or faults forming unstable wedges on excavation faces. Any overlying soils, however, will need to be supported or battered to a safe angle. Any excavation faces through sandstone that are greater than 1.5m high will need to be inspected during construction by a geotechnical engineer to identify any adverse features that require stabilisation.

For excavation faces within the site and not along site boundaries, suggested temporary and permanent batter slopes for unsupported excavations are shown in Table 2. If surcharge loads are applied near the crest of the slope then further specific geotechnical review and probably flatter batters or stabilisation using rock bolts or soil nails may be required.

Table 2: Recommended Batter Slopes for Exposed material

Material	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
Soils	1.5 : 1	2 : 1**
VL - L Sandstone	0.5 : 1*	1 : 1*
M-H & H Sandstone	Vertical*	Vertical*

Note: VL = Very Low Strength, L = Low Strength, M = Medium Strength, H = High Strength

* Subject to jointing assessment by experienced Geotechnical Engineer/Engineering Geologist

** Permanent batters in soil may need to be reduced to 3H: 1V to facilitate maintenance of grassed slopes, if required

Given that the typical main joint sets within Hawkesbury Sandstone in the Sydney region are likely to be oriented at acute angles to the proposed excavation faces, there may be some narrow wedges formed where these near vertical joints intersect the excavation faces and some rock bolts may be required to stabilise these wedges. This potential requirement, however, can only be assessed by inspection during excavation.

6.5 Retaining Walls

Engineer designed retaining walls should be founded on sandstone bedrock. The design of the retaining walls may be based on a triangular earth pressure distribution using earth pressure coefficients provided in Table 3. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_0) values should be used where the wall movement needs to be reduced (i.e. adjacent to existing structures or utilities).

Table 3: Recommended Design Parameters for Retaining Walls

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (K_a)	At Rest (K_o)		
Soils	20	0.3	0.45	0	25
VL-L Sandstone	22	0.2	0.3	15	29
M-H & H Sandstone	24	0*	0*	30	30

VL = Very Low Strength, L = Low Strength, M = Medium Strength, H = High Strength

* Subject to jointing assessment by experienced Geotechnical Engineer/Engineering Geologist

The design of the retaining walls should allow for all surcharge loads, including building footings, inclined slopes behind the wall, and construction related activities. The retaining walls should also incorporate free draining backfill material and appropriate subsoil drainage to prevent water pressure building up behind the wall.

Passive resistance for footings or piles in rock below the base of the bulk excavation may be based on the values provided in Table 4. The top 0.5 m below the bulk excavation level should be ignored due to possible disturbance and over-excavation.

Table 4: Recommended Passive Resistance Values

Material Description	Maximum Allowable Passive Pressure (kPa)	Maximum Ultimate Passive Pressure (kPa)
VL-L Sandstone	200	700
M-H & H Sandstone	1300	4000

VL = Very Low Strength, L = Low Strength, M = Medium Strength, H = High Strength

6.6 Groundwater

It is expected that during and following periods of wet weather there will be ongoing seepage both along the top of rock and also along bedding planes or other structures in the rock. This seepage is not expected to be associated with a regional groundwater table and will fluctuate with rainfall and climatic conditions.

Appropriate allowance of subsoil drainage should be incorporated into the design and construction to reduce the possible effects of adverse moisture and to ensure the amenity of all below-ground areas.

The design of the drainage measures should allow for future inspection, maintenance and cleaning of drainage lines, as it is common for a red-brown iron hydroxide sludge to build up in drainage systems.

6.7 Stormwater Runoff

It is recommended that all surface and stormwater runoff from both the house and the surrounding land be collected in a properly designed stormwater system and either directed off site into a stormwater pipe or to a location on the site where it can be dispersed gradually in a manner that does not adversely affect the stability of the site or the properties downslope. Such a stormwater disposal system should disperse the water so that the flows are not concentrated.

6.8 AGS Slope Stability Risk Assessment

While inspection of the existing structures on the site indicated no evidence of major slope instability in the area of the proposed new residence, the site is very steeply sloping and there has been a history of slope instability on properties downslope from the site.

The risk of slope instability from hazards on the site affecting both the site and the adjacent sites has been assessed in accordance with the methods of the Australian Geomechanics Society (AGS) "Practice Note Guidelines for Landslide Risk Management, 2007".

Table 5: Potential Hazard Identification

Hazard	Description	Potential Impact	Strategies to minimise occurrence or impact
1	Soil creep on steep slopes affecting new and existing structures, particularly landscaping walls	Unlikely to occur on upper western section of site which is more gently sloping. New house is to be located in this area. Could occur on lower steeper sections of the site but there are no structures planned for this area.	Ensure all new structures are founded on sound bedrock.
2	Settlement or movement of footings due to footings founded on boulders instead of bedrock	Could cause cracking of structures if boulders move slightly	Ensure all new structures are founded on sound bedrock.
3	Collapse of part of excavation face in rock due to adversely oriented joints causing unstable rock wedges	Could injure construction staff and damage any structures located behind excavation face	Ensure excavation faces are inspected by a geotechnical engineer at no more than 1.5 m vertical intervals

Hazard	Description	Potential Impact	Strategies to minimise occurrence or impact
4	Collapse of new retaining walls due to poor construction	Could damage adjacent areas and structures or people next to the walls.	Ensure all retaining walls supporting or adjacent to the new house and other structures are founded on bedrock and designed with adequate drainage
5	Rocks falling from existing cliff faces or cliff face collapse due to weathering and undermining of rock	Could undermine the eastern side of the new house and swimming pool. Unlikely to roll or slide as far as the houses on the downslope properties due to the distance and the likely tabular shape of most blocks.	Ensure footings for new house and other structures are supported on bedrock which is not undermined or overhanging eroded seams.
6	Soil slump failures caused by saturation of soils on steep slopes	Unlikely to impact new house which is located on the upper more gently sloping section of the site. Could impact properties downslope by saturating the soils and triggering slope failures.	Collect stormwater runoff and either direct into a stormwater pipe or into a disposal system which reduces the risk of concentrated flows. This should improve the existing condition and reduce the likelihood of slope failure

A qualitative assessment of likelihood, consequence and slope instability risk to the existing and proposed structures from the identified hazards after completion of construction (assuming appropriate engineering design and construction works) is summarised in Table 6.

Table 6: Slope Instability Risk Assessment for Risk to Property

Hazard	Likelihood	Consequence	Risk
1 – soil creep on steep slopes impacting landscaping walls	Unlikely - on the upper section of the site	Medium	Low
	Possible - on the lower steeper sections	Insignificant	Very Low
2 – settlement or movement of footings founded on boulders and not on bedrock	Unlikely – provided footings are founded on intact rock and inspected by geotechnical personnel	Medium	Low
3 – collapse of excavation face during construction	Unlikely – subject to inspection of rock faces during excavation and installation of any necessary rock bolts or shotcrete	Minor	Low
4 – collapse of new retaining walls	Rare – if walls are founded on sandstone bedrock inspected by geotechnical personnel and walls are appropriately designed, constructed and maintained	Medium	Low

Hazard	Likelihood	Consequence	Risk
5 – rocks falling from existing cliffs and impacting either house above cliff or houses on neighbouring properties downslope	Unlikely - for proposed new house above cliff – provided new footings are founded on sound bedrock which is not undermined	Medium – for undermining of new structures on site	Low
	Unlikely – for houses downslope due to distance from hazard	Medium	Low
6 – soil slumps caused by saturation of soils	Unlikely - on the upper section of the site	Insignificant	Very Low
	Unlikely - on the lower steeper sections or on adjacent properties provided stormwater runoff is carefully controlled and concentrated flows are avoided	Medium	Low

The key findings of this assessment is that provided all new structures are founded on sound bedrock and stormwater runoff is carefully controlled then the risk to property of the proposed new development is low.

For loss of life, the individual risk can be calculated from:

$R_{(LoL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times V_{(D:T)}$

where:

- $R_{(LoL)}$ is the risk (annual probability of loss of life of an individual);
- $P_{(H)}$ is the annual probability of the hazardous event (e.g. failure of the wall/excavation);
- $P_{(S:H)}$ is the probability of spatial impact by the hazard (e.g. of the failure reaching the residence taking into account the distance for a given event);
- $P_{(T:S)}$ is the temporal probability (e.g. of the area being occupied by an individual) given the spatial impact; and
- $V_{(D:T)}$ is the vulnerability of the individual (probability of loss of life of the individual given the impact).

The assessed individual risk to life (person most at risk) resulting from the identified hazards is summarised in Table 7.

Table 7: Slope Instability Risk Assessment for Risk to Life

Hazard	$P_{(H)}$	Person at risk	$P_{(S:H)}$	$P_{(T:S)}$	$V_{(D:T)}$	Risk $R_{(LoL)}$
1 – soil creep on steep slopes impacting landscaping walls	1.0×10^{-4}	Person on site	1.0	1.8×10^{-2}	1×10^{-4}	1.8×10^{-10}
2 – settlement or movement of footings founded on boulders and not on bedrock	1.0×10^{-4}	Person on site	0.33	0.75	1×10^{-4}	2.5×10^{-9}
3 – collapse of excavation face during construction	1.0×10^{-4}	Person on site	8.0×10^{-2}	0.5	0.2	8.0×10^{-7}
4 – collapse of new retaining walls	1.0×10^{-5}	Person on site	2.5×10^{-1}	1.8×10^{-2}	0.2	9.1×10^{-9}
5 – rocks falling from existing cliffs and impacting either house above cliff or houses on neighbouring properties downslope	1.0×10^{-4}	Person on site	1.5×10^{-1}	0.5	0.1	7.5×10^{-7}
	1.0×10^{-4}	Person in downslope house	1.0×10^{-2}	0.75	0.3	2.2×10^{-7}
6 – soil slumps caused by saturation of soils	1.0×10^{-4}	Person in downslope house	0.1	0.75	0.1	7.5×10^{-7}

When compared to the Landslide Risk Management Guidelines of the AGS, it is considered that the proposed development meets 'Acceptable Risk Management' criteria with respect to both property and life for new developments under current and foreseeable conditions.

Provided the construction is undertaken in accordance with the recommendations contained in this report and using sound engineering and construction practices, the proposed work would not be expected to adversely affect the overall stability of the site or negatively influence the geotechnical hazards identified in Tables 6 and 7.

6.9 Conditions Relating to Design and Construction Monitoring

To comply with Northern Beaches (Pittwater) Council conditions and to enable the completion of Pittwater Forms 2b and 3 (which are required as part of the construction, building and post-construction certificate requirements of the GRMP), it will be necessary for DP to:

- review the geotechnical content of all structural drawings (Form 2b requirement); and

- progressively inspect all new footing excavations and bulk excavations into the slope to confirm compliance to design with respect to allowable bearing pressure and stability, and inspect retaining wall drainage measures (Form 3 requirement).

6.10 Design Life and Requirement for Future Geotechnical Assessments

Douglas Partners Pty Ltd interprets the reference to design life requirements specified within the GRMP to refer to structural elements designed to retain the subject slope and maintain the risk of instability within acceptable limits.

Specific structures that may affect the maintenance of site stability in relation to the proposed development on this site are considered to comprise:

- existing (and any proposed) stormwater surface drains and buried pipes leading to the stormwater disposal system; and
- existing and proposed retaining walls on the site.

In order to attain a structural life of 100 years as required by the Council Policy, it will be necessary for the structural engineer to incorporate appropriate construction detailing and for the property owner to adopt and implement a maintenance and inspection program. A typical program for developments on sloping sites is given in Table 8.

Table 8: Recommended Maintenance and Inspection Program

Structure	Maintenance/Inspection Task	Frequency
Drainage lines	Inspect to ensure lines are flowing and not blocked.	Every 2 years or following each significant rainfall event.
Drainage pits	Inspect to ensure that pits are free of debris and sediment build-up. Clear surface grates of vegetation/litter build-up.	During normal grounds maintenance and following each significant rainfall event.
Retaining walls	Inspect walls for the presence of cracking or rotation from vertical, or as-constructed condition	Every 5 years or following each significant rainfall event.
General slopes	Inspect slopes and batters for indications of movement which may comprise tension cracks, back scarps of freshly exposed soil.	Every 2 years or following each significant rainfall event.

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to a relevant professional (e.g. structural engineer or geotechnical engineer).

7. Limitations

Douglas Partners (DP) has prepared this report for this project at 50 Minkara Road, Bayview in accordance with DP's proposal dated 25 July 2019. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Ms Wong for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations 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.

Copyright

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report 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 or excavation. 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 and test pits 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 or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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 advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

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

About this Report

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report 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 specially edited document. DP 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 and environmental 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.

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50).}$ It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

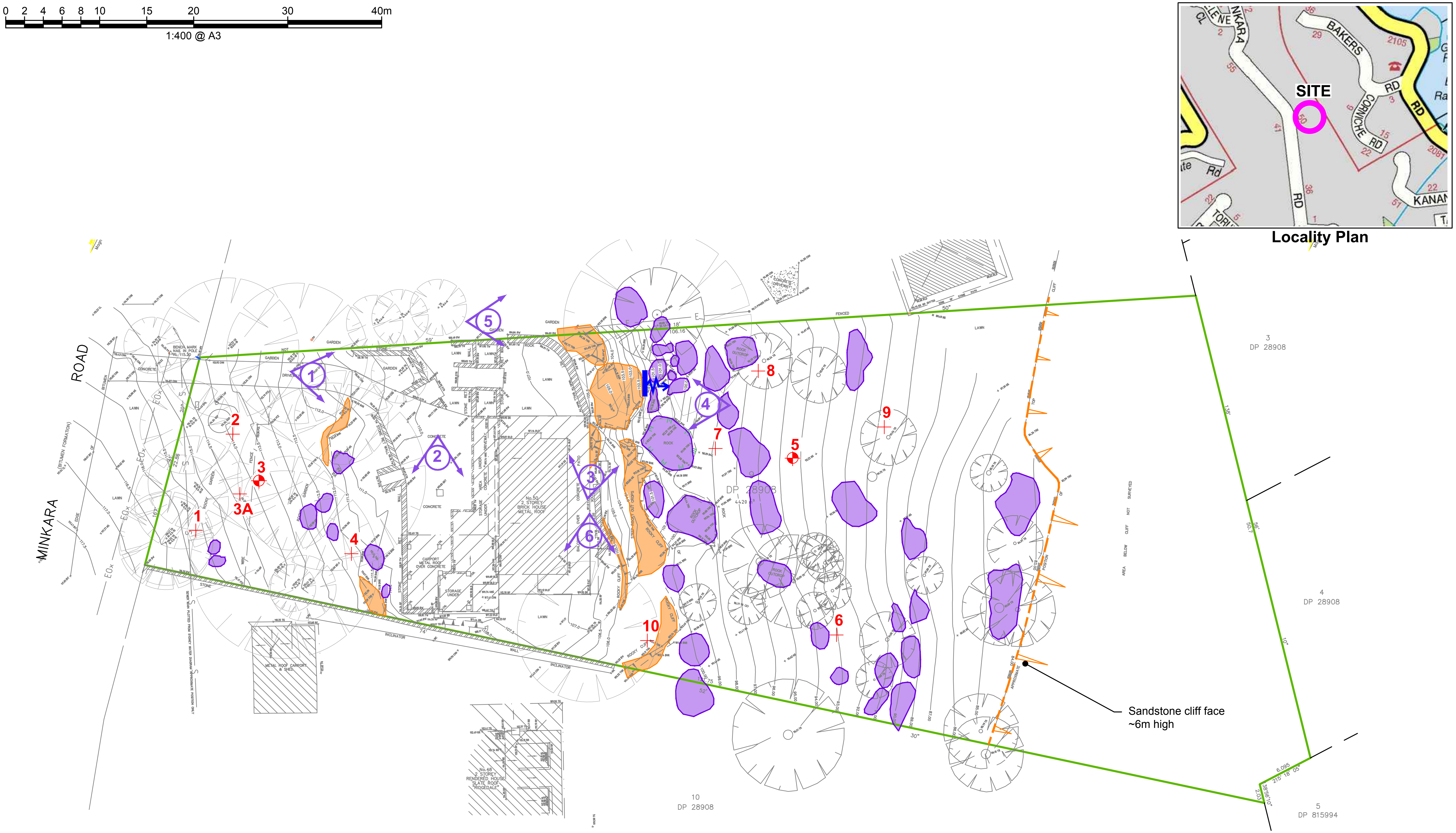
- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Appendix B

Drawings and Site Photographs



NOTE:
1: Base drawing from C.M.S. Surveyors Pty Ltd (Dwg 217Adetail, dated 13.12.2017)
2: Test locations are approximate only and are shown with reference to existing features.

LEGEND

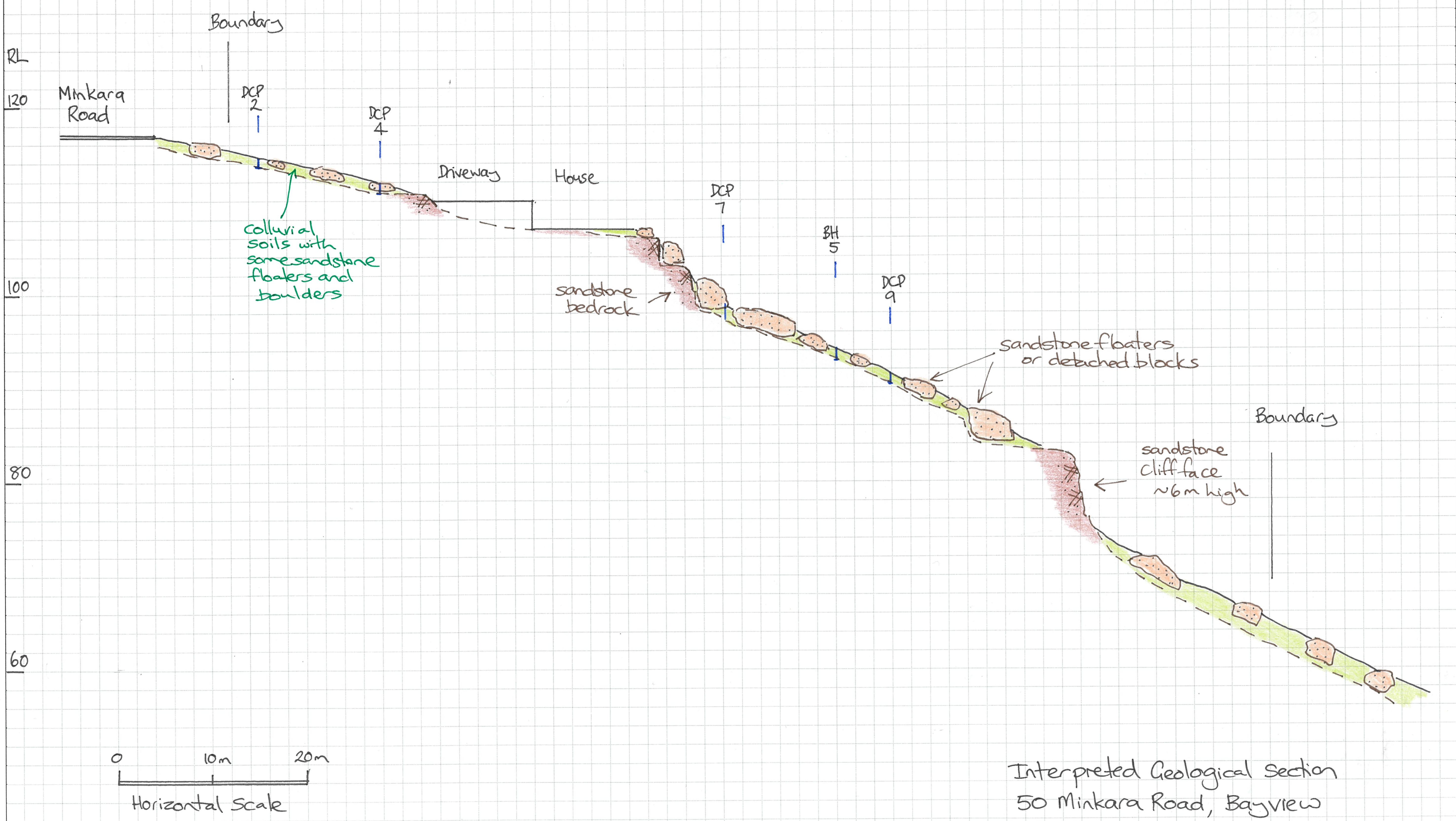
- Borehole location
- Dynamic cone penetrometer test location
- Floater or detached block
- Sandstone outcrop
- Seepage
- 1 Photo number with direction of view

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ms M. Wong		TITLE: **Location of Tests** **Proposed New Residence** **50 Minkara Road, BAYVIEW**	PROJECT No: 86886.00
OFFICE: Sydney	DRAWN BY: PSCH		DRAWING No: 1
SCALE: 1:400 @ A3	DATE: 19.9.2019		REVISION: 0

Douglas Partners
Geotechnics • Environment • Groundwater

PROJECT ______ Project No: ______ Page No: ______ Calcs By: ______ Checked By: FM. Date: 10/10/2019



Interpreted Geological Section
50 Minkara Road, Bayview
Project 86886.00
Drawing 2

FORM: A3 Grid

Photo 1 - view down driveway towards existing house

Photo 2 - view of existing carport area

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photos**	PROJECT:	86886
	Proposed New Residence	Plate No.	1
	50 Minkara Rd, Bayview	REV:	0
	CLIENT: Ms M Wong	DATE:	25-Sep-19

Photo 3 - looking north from eastern side of house (note large floaters)

Photo 4 - looking back up at rocks below north-eastern corner of house

Douglas Partners Geotechnics / Environment / Groundwater	Site Photos Proposed New Residence 50 Minkara Rd, Bayview	PROJECT:	86886
		Plate No.	2
		REV:	0
	CLIENT: Ms M Wong	DATE:	25-Sep-19

Photo 5 - lawn area near north-eastern corner of house

Photo 6 - view south from existing house

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photos**	PROJECT:	86886
	Proposed New Residence	Plate No.	3
	50 Minkara Rd, Bayview	REV:	0
	CLIENT: Ms M Wong	DATE:	25-Sep-19

Appendix C

Field Test Results

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

Results of Dynamic Penetrometer Tests

Client Ms M Wong

Project Proposed New Residence

Location 50 Minkara Road, Bayview

Project No. 86886.00

Date 30/06/04

Page No. 1 of 1

Test Location	1	2	3	3A	4	5	6	7	8	9
RL of Test (AHD)	115.8	113.9	113.8	114.4	112.1	94.4	93.0	99.0	96.8	91.8
Depth (m)	**Penetration Resistance** Blows/150 mm									
0 - 0.15	1	8	2	2	1	5	3	4	1	2
0.15 - 0.30	4	16	18	8	1	11	4	9	5	3
0.30 - 0.45	12	17	25/30mm	5	2	8	3	9	9	2
0.45 - 0.60	16	28/50mm		6	17	7	4	6	12	5
0.60 - 0.75	25			11	28	25	25/140mm	7	8	15
0.75 - 0.90	25/30mm			25	16	25/30mm		9	6	25/50mm
0.90 - 1.05				25/100mm	25/100mm			25/140mm	25/100mm	
1.05 - 1.20										
1.20 - 1.35										
1.35 - 1.50										
1.50 - 1.65										
1.65 - 1.80										
1.80 - 1.95										
1.95 - 2.10										
2.10 - 2.25										
2.25 - 2.40										
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										
3.00 - 3.15										
3.15 - 3.30										
3.30 - 3.45										
3.45 - 3.60										

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Flat End Penetrometer ☐

Tested By MMK
Checked By RKL

Remarks Ref = Refusal, 24/110 indicates 25 blows for 110 mm penetration

BOREHOLE LOG

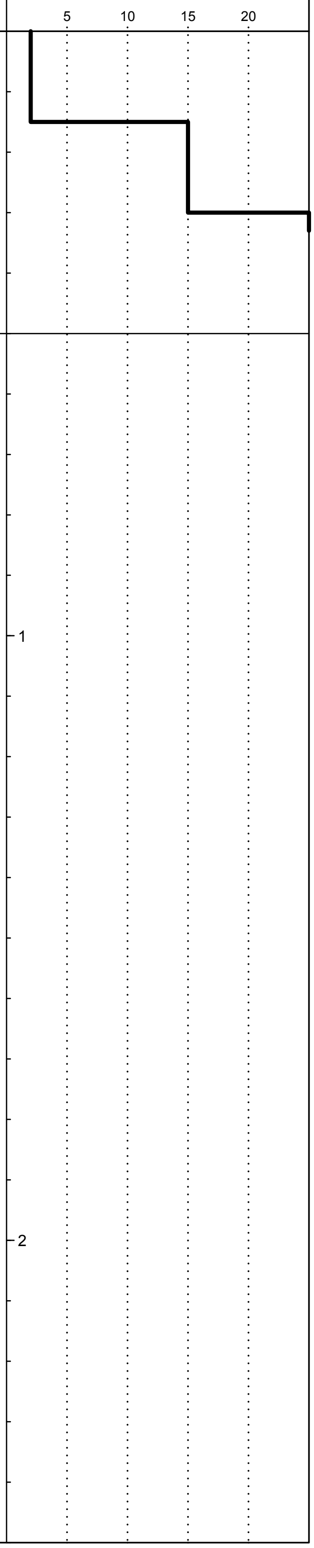
CLIENT: Ms M Wong
PROJECT: Proposed New Residence
LOCATION: 50 Minkara Road, Bayview

SURFACE LEVEL: 113.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 86886.00
DATE: 30/6/2004
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		SILTY GRAVELLY SAND: dark and light grey, silty gravelly sand with some sandstone cobbles and clay. Colluvium		A	0.2			
	0.5	Bore discontinued at 0.5m - refusal on sandstone boulder						
113	1							
112	2							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand Tools **DRILLER:** MMK **LOGGED:** MMK **CASING:** Nil
TYPE OF BORING: Hand auger to 0.5m
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Constant head permeability test carried out at 0.5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

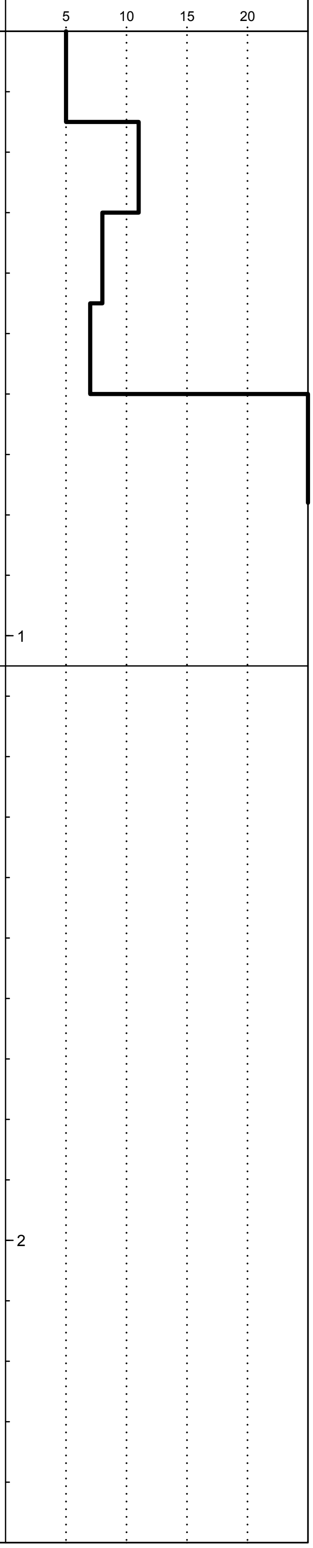
CLIENT: Ms M Wong
PROJECT: Proposed New Residence
LOCATION: 50 Minkara Road, Bayview

SURFACE LEVEL: 94.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 86886.00
DATE: 30/6/2004
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		GRAVELLY SAND: grey, silty gravelly sand with some sandstone cobbles. Colluvium						
94	0.2	SANDY CLAY: stiff, orange brown, sandy clay. Colluvium						
	1.05	Bore discontinued at 1.05m - refusal on sandstone boulder or possibly bedrock						
93								
92								

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand tools **DRILLER:** MMK **LOGGED:** MMK **CASING:** Nil
TYPE OF BORING: Hand auger to 1.05m
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Constant head permeability test carried out at 0.5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

Constant Head Permeameter Test Report [AS1547 App G]

Client:	Ms M Wong	Project No:	86886.00
Project:	Proposed New Residence	Date:	30-Jun-04
Location:	50 Minkara Road, Bayview	Tested by:	MMK

Test Location

Description: Slope above house
Material type: Colluvium - silty gravelly sand
Condition of ground surface before test: Dry
Weather during test: Clear

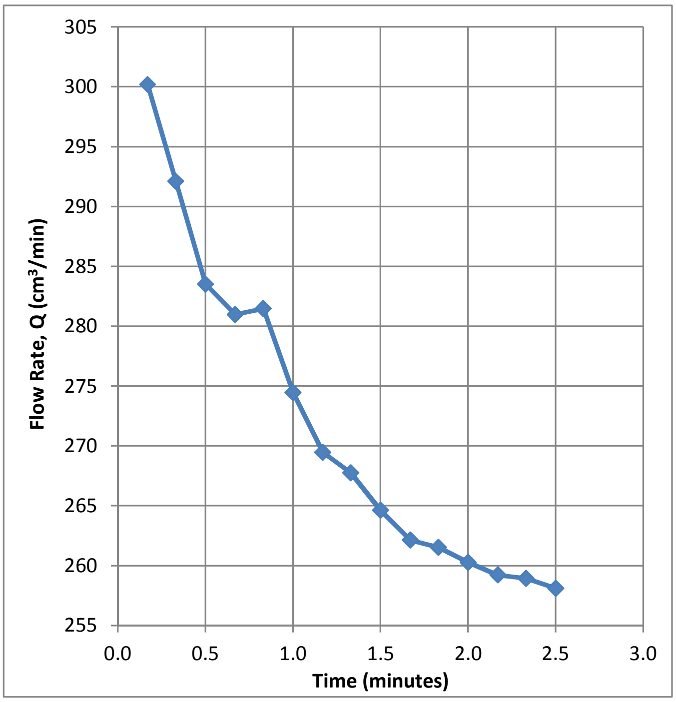
Test No. BH3
Easting: m
Northing m
Surface Level: 113.8 m AHD

Details of Bore Installation

Depth of augered hole	500	mm	Depth to impermeable layer	20	m
Depth of constant water below permeameter	250	mm	Time from filling to start	5	minutes
Diameter of hole	120	mm	Diameter of permeameter	38	mm

Test Results

Time (minutes)	Level below top (mm)	Flow Volume (cm^3)	Rate of Loss [Q] (cm^3/min)
0.00	0	0	
0.17	45	51	300
0.33	85	96	292
0.50	125	142	284
0.67	166	188	281
0.83	206	234	281
1.00	242	274	274
1.17	278	315	269
1.33	314	356	268
1.50	350	397	265
1.67	386	438	262
1.83	422	479	262
2.00	459	521	260
2.17	496	563	259
2.33	532	603	259
2.50	569	645	258
Totals	569	645	258 Overall



Saturated Hydraulic Conductivity - Over total duration of test

k =	**1.23E-01**	cm/min	where $K = 4.4Q[0.5 \sinh^{-1}(H/2r)-\sqrt{[(r/H^2)+0.25]}+r/H]/2\pi H^2$
=	**2.05E-05**	m/sec	ref. AS1547-2012 App G
=	**1.77**	m/day	

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Constant Head Permeameter Test Report [AS1547 App G]

Client:	Ms M Wong	Project No:	86886.00
Project:	Proposed New Residence	Date:	30-Jun-04
Location:	50 Minkara Road, Bayview	Tested by:	MMK

Test Location

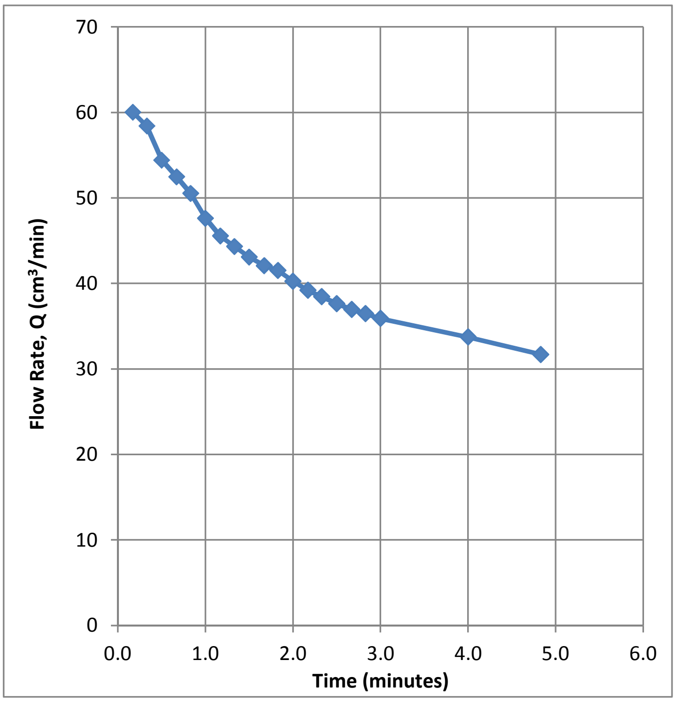
Description:	Slope below house	Test No.	BH5	
Material type:	Colluvium - sandy clay	Easting:		m
Condition of ground surface before test:	Dry	Northing		m
Weather during test:	Clear	Surface Level:	94.4	m AHD

Details of Bore Installation

Depth of augered hole	500	mm	Depth to impermeable layer	20	m
Depth of constant water below permeameter	250	mm	Time from filling to start	5	minutes
Diameter of hole	120	mm	Diameter of permeameter	38	mm

Test Results

Time (minutes)	Level below top (mm)	Flow Volume (cm^3)	Rate of Loss [Q] (cm^3/min)
0.00	0	0	
0.17	9	10	60
0.33	17	19	58
0.50	24	27	54
0.67	31	35	52
0.83	37	42	51
1.00	42	48	48
1.17	47	53	46
1.33	52	59	44
1.50	57	65	43
1.67	62	70	42
1.83	67	76	42
2.00	71	81	40
2.17	75	85	39
2.33	79	90	38
2.50	83	94	38
2.67	87	99	37
2.83	91	103	36
3.00	95	108	36
4.00	119	135	34
4.83	135	153	32
Totals	135	153	32 Overall



Saturated Hydraulic Conductivity - Over total duration of test

k =	**1.51E-02**	cm/min	where $K = 4.4Q[0.5 \sinh^{-1}(H/2r)-\sqrt{(r/H^2)+0.25}+r/H]/2\pi H^2$
=	**2.52E-06**	m/sec	ref. AS1547-2012 App G
=	**0.22**	m/day	

Appendix D

Forms 1 and 1a, and AGS Guidelines

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for Ms Miranda Wong
Name of Applicant
Address of site 50 Minkara Rd, Bayview

Declaration made by geotechnical engineer or engineering geologist ~~or~~ coastal engineer (where applicable) as part of a geotechnical report

I, Fiona MacGregor *on behalf of* Douglas Partners Pty Ltd
(Insert Name) (Trading ~~or~~ Company Name)

on this the 14 october 2019 certify that I am a geotechnical engineer or engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $10million.

I:
Please mark appropriate box

- ∋ ✓ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ∋ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ∋ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ∋ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ∋ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ∋ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Report on Geotechnical Assessment, Proposed New Residence, 50 Minkara Rd, Bayview
Report Date: 14 october 2019
Author: Fiona MacGregor
Author's Company/Organisation: Douglas Partners Pty Ltd

Documentation which relate to or are relied upon in report preparation:

Drawings DA0201, 202, 203, 204, 205, 206, 1111, 2100, 2101, 3002, 3003, 3004, 3005, 3101 and SK006 - all prepared by Robert Harrison, architect + dated June 2019

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature Fiona MacGregor
Name Fiona MacGregor
Chartered Professional Status CPEng, FIEAust
Membership No. 370757
Company Douglas Partners Pty Ltd

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for Ms Miranda Wong
Name of Applicant
Address of site 50 Minkara Road, Bayview

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Report on Geotechnical Assessment, 50 Minkara Rd Bayview,
Report Date: 14 October 2019
Author: Fiona MacGregor
Author's Company/Organisation: Douglas Partners Pty Ltd

Please mark appropriate box

- ✓ Comprehensive site mapping conducted 3 September 2019
(date)
- ✓ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- Subsurface investigation required
  - No Justification ....
  - ✓ Yes Date conducted 2004
- ✓ Geotechnical model developed and reported as an inferred subsurface type-section
- ✓ Geotechnical hazards identified
  - Above the site
  - ✓ On the site
  - ✓ Below the site
  - Beside the site
- ✓ Geotechnical hazards described and reported
- ✓ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ✓ Consequence analysis
  - Frequency analysis
- Risk calculation
- ✓ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ✓ Design Life Adopted:
  - 100 years
  - Other ....
specify
- ✓ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ✓ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

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LANDSLIDE RISK

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is often covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site)
- the likelihood that they will occur
- the damage that could result
- the cost of disruption and repairs and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of repairs and temporary loss of use if a landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2: LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerated", etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1: RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	**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 the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**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 possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. Importantly, the data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us any day. If this were not so, no one would ever be struck by lightning.

Most local councils and planning authorities that stipulate a tolerable risk to property also stipulate a tolerable risk to life. The AGS Practice Note Guideline recommends that 1:100,000 is tolerable in newly developed areas, where works can be carried out as part of the development to limit risk. The tolerable level is raised to 1:10,000 in established areas, where specific landslide hazards may have existed for many years. The distinction is deliberate and intended to prevent the concept of landslide risk management, for its own sake, becoming an unreasonable financial burden on existing communities. Acceptable risk is usually taken to be one tenth of the tolerable risk (1:1,000,000 for new developments and 1:100,000 for established areas) and efforts should be made to attain these where it is practicable and financially realistic to do so.

TABLE 3: RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding , ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other AUSTRALIAN GEOGUIDES:

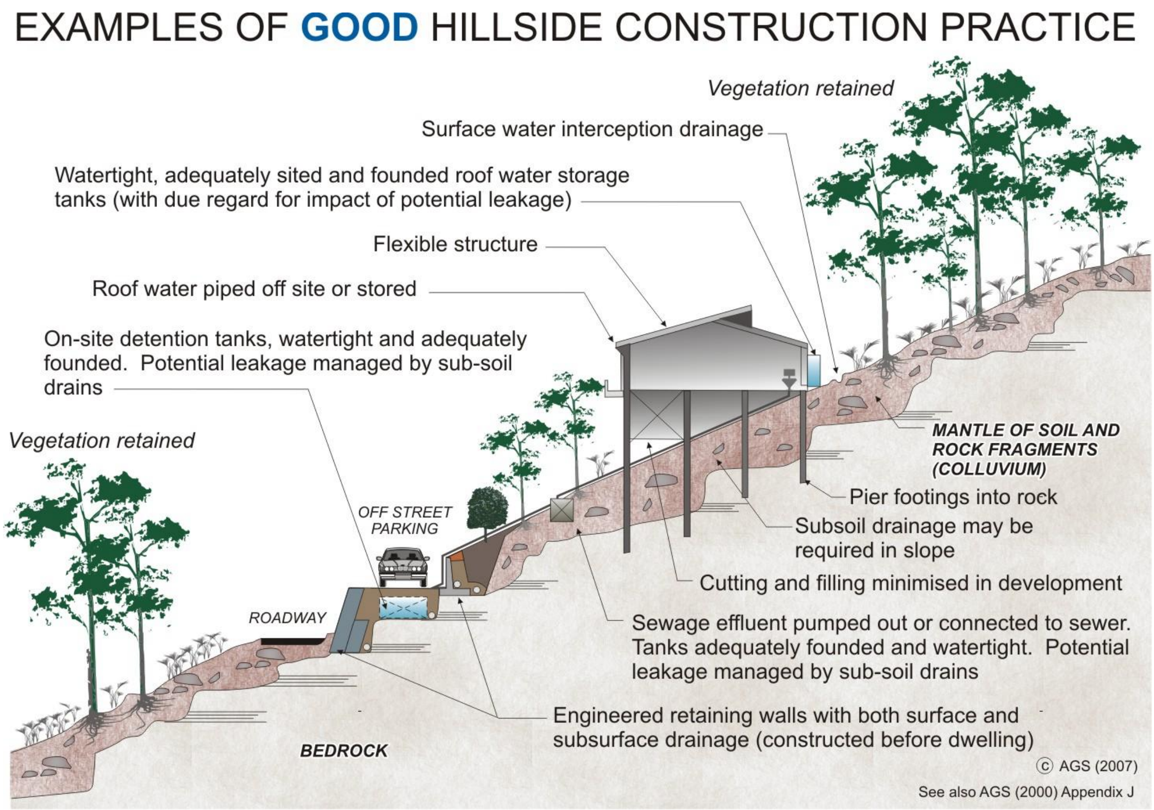
- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
  GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

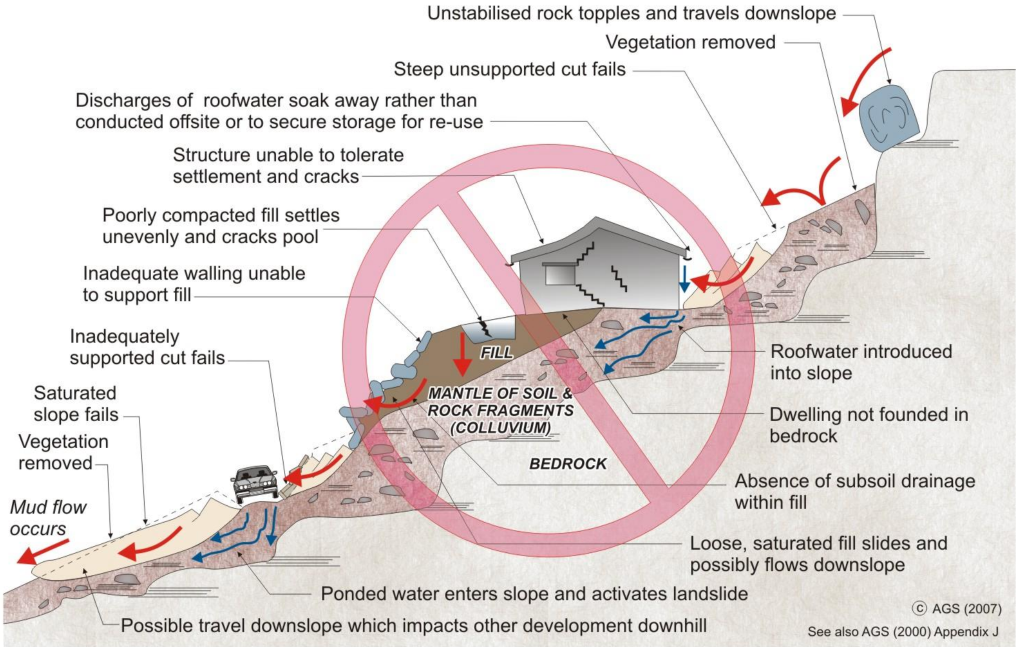
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
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