Douglas Partners
Geotechnics / Environment / Groundwater

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Project 203454.00
28 June 2021
DEM:pc

Mr Peter Madew
521 Barrenjoey Road
BILGOLA BEACH NSW 2107

Geotechnical Assessment, Proposed New Residence
521 Barrenjoey Road, Bilgola Beach

1. Introduction

This report presents the results of a geotechnical assessment by Douglas Partners Pty Ltd (DP), of the site of a proposed new residence at 521 Barrenjoey Road, Bilgola Beach (Lot 129 in D.P 16902). The work was carried out at the request of Mr Peter Madew, the property owner.

The geotechnical assessment comprised a detailed geological inspection of the property and adjacent areas, along with reference to a cored borehole that was drilled for a previous site investigation. Reference has also been made to the following design drawings:

- Design Drawings 2004-00 to 2004-14 (dated 17 June 2021) by Peter Downes Designs;
- Survey Drawing Ref 2861 DS (Issue A dated 4 March 2021) by Richards & Loftus Surveying Services

Comments relating to geotechnical design issues and constraints are given below and are based on the results of the inspection and the information shown on the above drawings.

2. Previous Assessment

DP has previously undertaken geotechnical investigation on the site for a former owner as presented in our report *Proposed Excavation for Garage and Driveway, 521 Barrenjoey Road, Bilgola (Project 73460.00, dated 16 May 2013)*.

The previous investigation comprised a walkover inspection of the site and surrounding slope by a senior engineering geologist, along with reference to bedrock samples obtained from a cored borehole (Bore 1). The bore had been earlier drilled by others on the existing slope, close to the maximum depth of proposed excavation for the former garage footprint. The approximate location of Bore 1 relative to the current proposed development is indicated on Drawing 1.

The previous investigation revealed that sandstone and siltstone bedrock typically underlies the slope at less than 1 m depth, and that the site exhibited no evidence of previous significant slope instability.

3. Site Description and Geology

Colour photographs (Photos 1 to 6) of the site at the time of the field work are provided on Plate 1, attached to this report.

The site is a near-rectangular shaped residential lot, with major plan dimensions of approximately 45 m by 15 m, located on the high, northern side of Barrenjoey Road. It is bounded by upslope residential lots to the north and by undeveloped bushland blocks to the east and west.

The site typically slopes to the south at about 25° and has a moderate cover of coastal eucalypts, shrubs and grasses (refer to Photos 2, 4 and 5).

Reference to the Sydney 1:100 000 Geological Series sheet indicates that the site is underlain by rocks of the Newport Formation which is the upper unit of the Narrabeen Group of rocks. These rocks are of Triassic age and typically comprise interbedded shale, laminite and lithic to quartz lithic sandstone.

Strata exposed within an excavated batter along the high side of Barrenjoey Road and within two small pits on the site are considered to be consistent with the Newport Formation.

4. Field Work

4.1 Site Inspection

The site was inspected on 11 June 2021 by the same senior engineering geologist who undertook the previous investigation in 2013. The main site observations are:

- strata exposed within the 1.5 m to 2 m high, 40° to 50° sloping excavated batter, located along the high side of Barrenjoey Road, typically comprise weathered sandstone and siltstone bedrock with an overlying colluvial soil profile to around 0.5 m depth (refer to Photo 1),
- there was no evidence of previous significant instability having occurred within the excavated road batter below the site,
- a similar sub-surface profile to that noted with the road batter was observed within the two small, open (partly backfilled?) excavations on the site (refer to Photo 3),
- there was no evidence that previous excavations on the site have led to any significant slope instability,
- there was no evidence of active groundwater seepage from the road batter, the open excavations or from elsewhere on the slope surface across the site.
- a timber log retaining wall at the northern site boundary, supporting an upslope property, appears to be in a satisfactory condition (refer to Photo 6).

In summary, there was little evidence of change on the site since DP's inspection in 2013, apart from the clearing of some weeds and shrubs.

3.2 Results of the Test Bore

The samples of bedrock core recovered from Bore 1 were previously geotechnically logged and photographed by an engineering geologist from DP. The geologist also undertook point load strength testing on representative sections of the core to determine characteristic bedrock strengths.

The full details of the subsurface profile encountered by Bore 1 are provided on the attached Bore Hole Log Sheet along with colour photographs of the core samples. In summary, Bore 1 encountered, very low strength, highly weathered and highly fractured sandstone bedrock below 0.6 m depth, extending to 2 m depth, underlain by medium to high strength, fractured sandstone and siltstone bedrock which extended to the termination of the bore at 8.4 m depth.

For the most part, the defects within the recovered bedrock core comprised bedding partings or joints with clayey or iron-coated surfaces. Their dips typically ranged from sub-horizontal to 50° from horizontal. Some steeper dipping joints were also noted.

5. Proposed Development

It is understood that the proposed development at the site will comprise the construction of a three to four level residence which will step up the slope from a garage located off Barrenjoey Road. Access to the residence will be by way of a horizontal tunnel and vertical shafts/stairs duginto the slope.

DP understands that the tunnel is proposed to be approximately 3 m by 3 m in cross section and will extend from approximately RL41.7 AHD at the rear of the garage for a distance of approximately 25 m to the base of a 23 m high and 2.5 m square, vertical lift-shaft located below the rear (northern end) of the proposed residence. A shorter, combined lift-shaft and stairway will also be located off the tunnel, below the southern end of the residence.

Due to the existing limited access from Barrenjoey Road, it is envisaged that the building works will be staged in the following sequence:

- Traffic control and signage
- Fencing and making initial access to the site off Barrenjoey Road
- Removal of vegetation, clearing, erosion & sediment control
- Excavation for the proposed garage, tunnel, lifts, stairs and house
- Construction of retaining walls, concrete works, foundation and structural works
- Application of cladding and external finishes

- Interior works and fit out
- Final landscaping and fencing.

The footprints of the proposed developments are indicated on Drawing 1.

5. Comments

5.1 Geological Model

Based on observations made on site and on previous experience in the area, the interpreted geological model for the site comprises a moderately sloping site with a possibly 0.5 m to 1 m deep colluvial or residual sandy clay and clayey sand soil profile (containing some ironstone fragments or layers) overlying very low to high strength, highly to moderately weathered interbedded siltstone and sandstone.

The water table possibly underlies the site at between 6 m to 8 m depth, but could rise closer to the surface, particularly following periods of extended wet weather.

An inferred geological cross-section (Cross Section A-A') is shown in Drawing 2, which is attached to this report.

5.2 Recommended Bulk Excavation Protocols

The results of the bore hole and observations of the strata exposed within the road batter suggests that sandstone or siltstone bedrock underlies the footprint of the proposed garage and driveway excavation at a relatively shallow depth. It is therefore considered likely that much of the proposed excavation will be within sandstone/siltstone bedrock, albeit highly fractured to fractured in parts. The colluvial soils and underlying bedrock. can be permanently retained by either the construction of a permanent anchored shotcrete wall or block retaining wall(s) around the perimeter of the proposed excavation.

5.2.1 Temporary Access off Barrenjoey Road

Following clearing of vegetation for the proposed excavation footprint, it is anticipated that initial excavation into the slope from Barrenjoey Road (under traffic control) will be required to provide access for tracked plant so that they can walk further up the slope and to provide a pull-over bay, wide enough for loading out of excavated material. Temporary unsupported batters for these initial excavations should not exceed 1:1 (V:H) within the colluvial soils and 4:1 (V:H) within bedrock. They should be supported by dowels/rock bolts and rockfall mesh (such as Geobrugg Tecco mesh or Maccaferri double twist rockfall mesh). Geo-fabric should be placed behind the rockfall mesh where it is supporting soil.

There will need to be erosion and runoff control measures in place and jersey kerbs and a rock catch fence placed along the edge of Barrenjoey Road.

5.2.2 Bulk Excavations for Garage and Residence

The bulk excavation for the proposed garage (and the residence further up the slope) should then be carried out systematically as a top-down excavation in drops not exceeding 1.5 m to 2.0 m.

For permanent support the overburden soils around the edges of the bulk excavation should be battered back to 1:1 (V:H) in drops not exceeding 1.5 m, each time applying 130 mm thick shotcrete (with SL 82 mesh centrally located), secured with permanent CT rock bolts, inclined 30° below the horizontal and perpendicular to the face. The bolts should be initially secured using a mechanised anchor in a 45 mm diameter hole then be bonded at least 2 m into low to medium strength bedrock with full column grouting.

Excavation should progress systematically, supporting the rock face as it is exposed in maximum 1.5 m to 2.0 m high drops.

The sandstone bedrock may be cut vertically in 1.5 m to 2.0 m drops, and permanently supported with 100 mm thick shotcrete (with SL 82 mesh centrally located) and 3 m long CT bolts. Longer bolts may be required if adversely dipping joints are exposed in the excavated face.

Each excavation drop should be inspected by geotechnical personnel prior to the application of shotcrete, to check for the requirement of additional support.

Drainage behind permanent shotcrete walls should comprise Mebra 100 strip drains, installed at 2 m centres and connected to a toe drain.

Alternatively, the bulk excavation's face could be temporarily supported with rockfall mesh and nails/dowels in the manner described for the temporary access off Barrenjoey Road.

Temporary working platforms required to undertake the excavation may be prepared from track-rolled weathered sandstone with downhill batters no steeper than 1.0:1.5 (V:H) and with fill batter heights not exceeding 2 m. The outer edge of such a platform may need to be secured using dowelled rockfall mesh.

Note that, if the shotcrete walls are designed to be temporary (rather than permanent) structures, the structural engineer will need to take into account the following long-term lateral loading in the design of retaining walls constructed in-front of the shotcrete.

Table 1 - Suggested Retaining Wall Design Parameters

Material	Earth Pressure Coefficient		Bulk Density
	Short term	Long term	
Filling, colluvial soils or highly weathered bedrock	0.3	0.4	20 kN/m^3
Sandstone/siltstone bedrock-low to medium strength, fractured	0.0	0.15	22 kN/m^3

It should be noted that no provision has been made in the above design parameters for:

- water pressures acting on the walls;
- sloping ground above;
- foundation loads from the proposed residence or other structures subsequently constructed upslope.

DP considers that undertaking the proposed bulk excavations in accordance with the recommended protocol will not adversely affect the stability of the site or the adjacent properties.

5.3 Proposed Tunnel and Vertical Shafts

It is recommended that, prior to the commencement of tunnel or shaft construction, a cored borehole be drilled at the location of the northern (upslope) shaft to the full depth of the shaft to check the geological profile.

Based on what can be seen of the geology at present, and on the results of Bore 1, it is anticipated that the tunnel will generally be excavated through medium to high strength, sub-horizontally interbedded sandstone and siltstone as was successfully done for Sydney Water's Whale Beach sewer tunnel, 30 years ago . An arched tunnel roof is considered preferable to a flat roof given the expected nature of the bedding planes within the rock.

It is understood that the tunnel will be approximately 3 m wide by 3 m high and will be free draining towards Barrenjoey Road. It is expected that the proposed tunnel portal will be excavated into the cut rock face at the rear of the bulk excavation for the garage. The portal and tunnel walls may require covering with a sprayed concrete (shotcrete) layer.

Prior to the commencement of tunnelling, the portal face should be marked up with the tunnel outline and a laser set up to keep the tunnel on line. The drilling of a central "burn-cut" hole would provide a free face to break the rock into.

The actual excavation method will depend largely on the contractor appointed for the work. The potential excavation methods range from using jackhammers, hydraulic rock splitters (Dada), a rock hammer mounted on an excavator, a milling head mounted on an excavator, a small roadheader, such as an Alpine R50 or a Mitsui S65. The rock could be removed by a Bobcat front-end loader emptying into skip bins alongside Barrenjoey Road. Water spray to suppress the dust; a conventional excavation and dust control approach used for rock excavation in Sydney, would probably be required within the portal area.

Once underground, excavation could continue, with the milling head, a rock hammer or road header. Productivity is likely to be between 1 and 2 cubic metres of rock excavation per hour with between 1 and 3 m advance per day.

Allowance should be made for 2 m long rockbolts to be installed every 1.5 m advance or as required, to support the tunnel roof. Stainless steel or galvanised; SL41 ARC mesh, or 80 mm of fibrecrete with mesh plates on the rock bolts may also be required for the tunnel roof to prevent fall-out of minor blocks/wedges or weathered bedrock. Note if fibrecrete is used, all seepages will need to be captured using slim-line "Mebra" drains or similar, with the drains discharging to the sidewalls of the tunnel or shaft.

Similar ground support will also be required for the vertical shafts.

The shafts may require head frames within the bulk excavations for the residence to allow setting up of a winder to lower men down to the advancing face and to bring up buckets of spoil. This spoil could be placed on a small conveyor belt to Barrenjoey Road, for removal by truck.

5.4 Groundwater

Some groundwater seepage from along bedding planes and joints will likely be encountered by both the tunnels and the shaft. This seepage will increase after periods of rain and, again, will likely contain soluble iron within the water which will precipitate to form a red-brown iron oxide/hydroxide sludge, once the water comes into contact with the air. Such sludge will require removal periodically to prevent blockage of the drains.

Seepage water can be directed into a spoon drain and run out of the tunnel portals for disposal.

5.5 Structural Foundations

Foundations for the various new structures can be designed as pad footings sized for allowable bearing pressures of:

Table 2 - Suggested Foundation Design Parameters

Material Type	Allowable Bearing Capacity	Allowable Bond Strength
Residual sandy clay soil	150 kPa	NA
Medium to high strength sandstone	3000 kPa	350 kPa compression 250 kPa tension

All excavations for new footings should be inspected by a geotechnical professional prior to the placement of reinforcement and concrete, so as to confirm that intact strata of sufficient bearing capacity and stability has been reached.

5.6 Slope Risk Analysis

There is no evidence of previous landslide or rockfalls on the site. Therefore, following The Australian Geomechanics (2007) Landslide Risk Management Guidelines, it is considered that there is a low risk of landslide damage to property or to people on the site or adjacent sites, provided the proposed works are properly designed and carefully constructed.

Potential geotechnical hazards located above, below and beside the site have been assessed for risk to property and life using the general methodology outlined by the Australian Geomechanics Society (Landslide Risk Management AGS Subcommittee 2007).

For the purposes of this assessment, an acceptable level of geotechnical risk for the property is “Low” while an accepted annual probability of loss of life is 1 x 10^{-6}.

Identified hazards are summarised in Table 3, together with qualitative assessments of likelihood, consequence and slope instability risk to the existing and proposed residential structures after completion of construction which has had appropriate engineering design and construction methodologies.

Table 3 - Property Slope Instability Risk Assessment for Existing and Proposed Developments

Hazard	Likelihood	Consequence	Risk
Failure of the bulk excavations, horizontal tunnel or vertical shafts/stairway destabilising the site	Rare – if the recommendations in this report are followed	Major	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 (death) of an individual)

$P_{(H)}$ is the annual probability of the hazardous event occurring (e.g. failure of the cliff-line)

$P_{(S:H)}$ is the probability of spatial impact by the hazard (e.g. of the failure reaching the residence, taking into account the distance of a given event from the residence)

$P_{(T:S)}$ is the temporal probability (e.g. of the residence being occupied by the individual) at the time of the spatial impact

$V_{(D:T)}$ 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 slope or excavation instability is summarised in Table 4.

Table 4 - Life Risk Assessment for Existing and Proposed Developments

Hazard	$P_{(H)}$	$P_{(S:H)}$	$P_{(T:S)}$	$V_{(D:T)}$	Risk $R_{(LoL)}$
Failure of the bulk excavations, horizontal tunnel or vertical shafts/stairway destabilising the site	10^{-5}	0.5	0.25	0.5	6.25×10^{-7}

When compared to the requirements of the Northern Beaches (Pittwater) Council and the AGS, it is considered that both the existing and proposed development meets 'Acceptable Risk Management' criteria with respect to life under current and foreseeable conditions.

Provided construction is undertaken in accordance with the recommendations contained in this report, construction of the proposed alterations and additions is not expected to affect the overall stability of the site or negatively influence the geotechnical hazards identified in Tables 1 and 2.

5.7 Stormwater Disposal

It is expected that the presence of relatively shallow bedrock will preclude the effective use of stormwater absorption pits or transpiration beds on this site. Therefore, it is recommended that all stormwater generated from the new developments on the site be piped to the edge of Barrenjoey Road via a system of appropriately sized pipes and storage/detention tanks .

6. Conditions Relating to Design and Construction Monitoring

To comply with Council conditions and to enable the completion of Forms 2B and 3, required as part of the construction, building and post-construction certificate requirements of the GRMP, it will be necessary for Douglas Partners Pty Ltd to:

Form 2B

- Review the geotechnical content of all structural drawings.

Form 3

- Inspect all new bulk, tunnel, shaft, and footing excavations for the new works to confirm compliance to design with respect to stability and allowable bearing pressure.

7. Design Life and Requirement for Future Geotechnical Assessments

DP interprets the reference to design life requirements specified within the IGRMP 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 retaining walls on the site.
- Tunnel and shaft support

In order to attain a structure life of 100 years as required by the Council Policy, it will be necessary for the structural engineer to incorporate appropriate construction detailing including the use of double corrosion protected rockbolts 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 3.

Note that the programme given in Table 5 is provisional and should be subject to review and/or deletion at the conclusion of construction.

Table 5 - Recommended Maintenance and Inspection Program

Structure	Maintenance/Inspection Task	Frequency
Stormwater drains, subsoil drains, pipes and pits	Owner to inspect to ensure that the drains, pipes and pits are free of debris and sediment build-up. Clear surface grates of vegetation/litter build-up.	Every year or following each significant rainfall event.
Retaining, tunnel and shaft walls, tunnel roof	Owner to check wall for deviation from as-constructed condition.	Every two to three years

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).

8. References

1. Pittwater Council's Geotechnical Risk Management Policy (2009).
2. Australian Geomechanics Society (AGS), Practice Note Guidelines for Landslide Risk.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 521 Barrenjoey Road, Bilgola Beach in accordance with DP's proposal SYD200695 dated 29 June 2020 and acceptance received from Mr Peter Madew, site owner. The work was carried out under DP's Conditions of Engagement.

This report is provided for the exclusive use of Mr Peter Madew for this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or another 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 and at the time the work was carried out. 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 scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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 a 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 / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

We trust that these comments are sufficient for your present requirements. If further assistance is required, please do not hesitate to contact the undersigned.

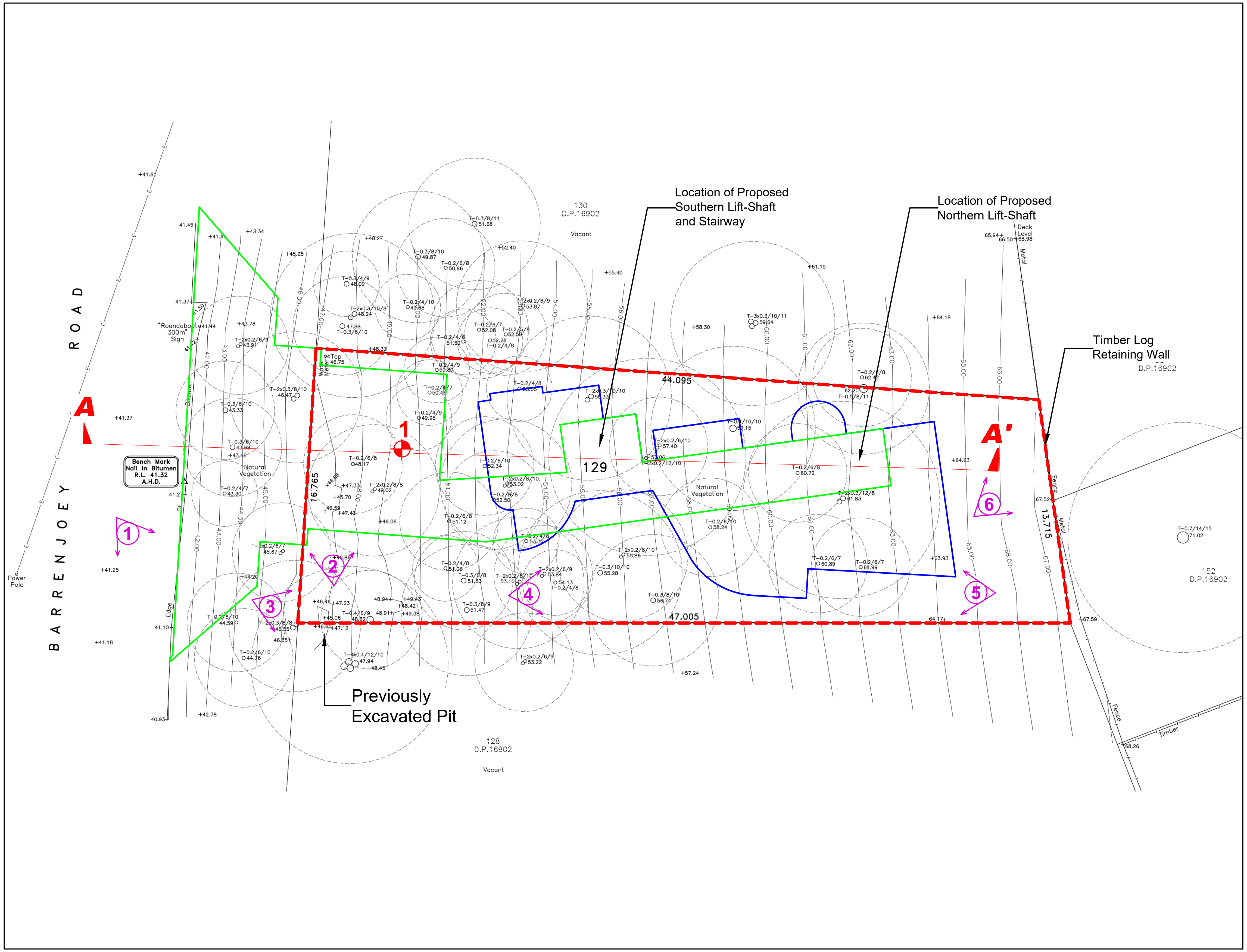
Yours faithfully
Douglas Partners Pty Ltd

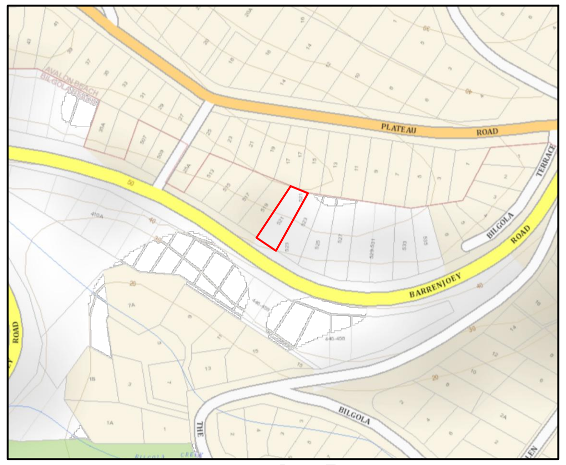
Reviewed by

David Murray
Senior Associate

John Braybrooke
Principal

Attachments: Notes about this report
Drawings 1 and 2
Plate 1 - Site Photographs 1 to 6





Locality Plan

LEGEND

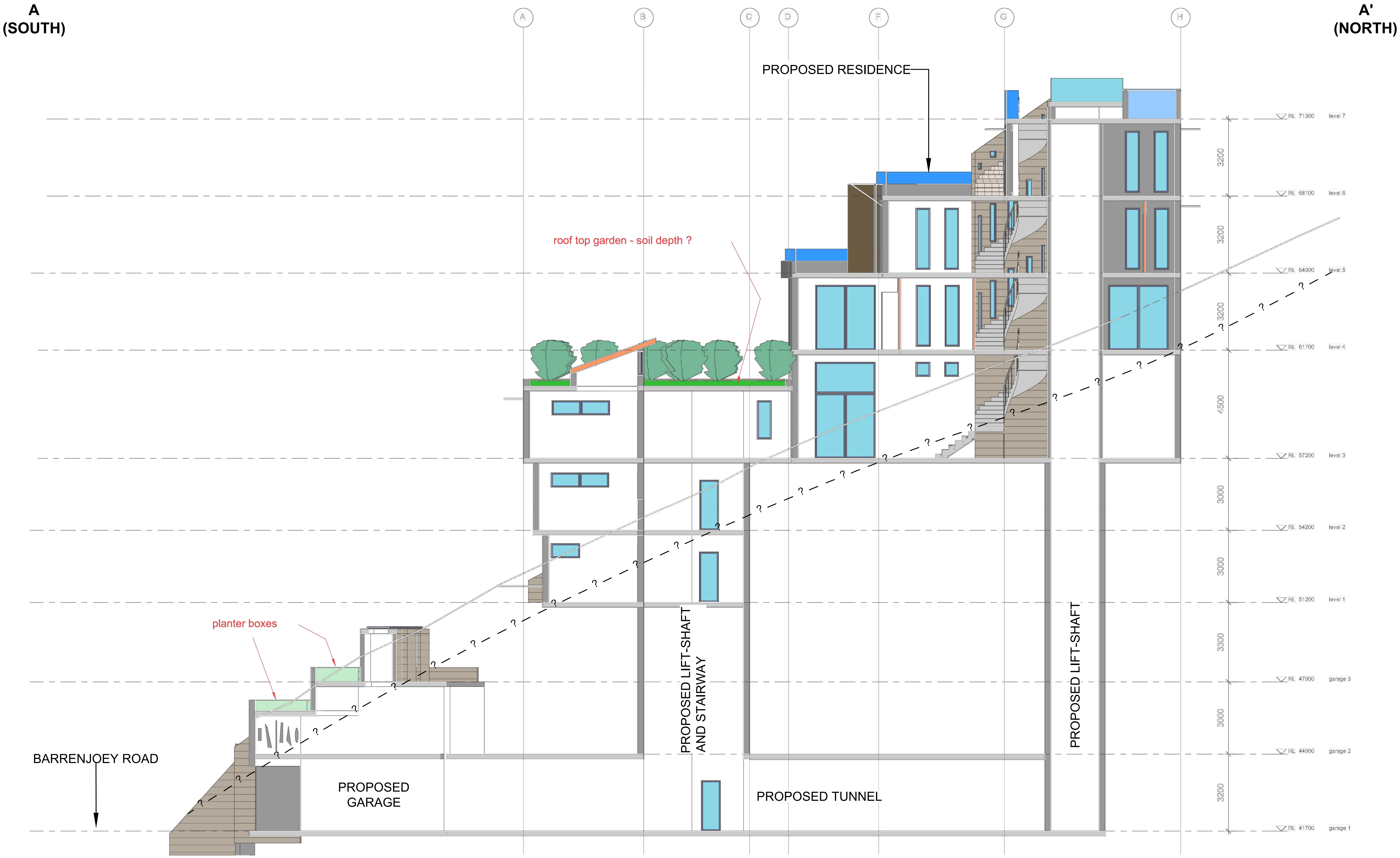
- Site Boundary
- Proposed Level 3 Footprint
- Proposed Footprint of Garage, Tunnel and Lift-Shaft
- Test Bore (Approximate Location)
- Cross Section (Refer to Drawing 2)
- Photo location and direction of view

NOTE:
1: Plan adapted from Drawing Reference 2861 DS, provided by Richards & Loftus, Job No. 2861, Issue A (Dated 04.03.2021)

0 1 2 3 4 5 6 7 8 9 10 15 20 25m
1:250 @ A3

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Mr Peter Madew		TITLE: **Geotechnical Features**	PROJECT No: 203454.00
OFFICE: Sydney	DRAWN BY: MG	**Proposed New Residence**	DRAWING No: 1
SCALE: 1:250 @ A3	DATE: 18.06.2021	**521 Barrenjoey Road, Bilgola Beach**	REVISION: 0



NOTE:
1: Plan adapted from Drawing No. A2 2004 12, provided by Peter Downes Designs (Dated 13.05.2021)
2: Refer to Drawing 1 for location of cross-section

0 1.0 2.0 3.0 4.0 5.0 7.5 10 15m

1:150 @ A3

LEGEND

— ? — — Inferred Level of Weathered Siltstone / Sandstone Bedrock

Douglas Partners *Geotechnics / Environment / Groundwater*	CLIENT: Mr Peter Madew	TITLE: **Cross-Section A-A'**	PROJECT No: 203454.00
	OFFICE: Sydney / DRAWN BY: MG	**Proposed New Residence**	DRAWING No: 2
	SCALE: 1:150 @ A3 / DATE: 18.06.2021	**521 Barrenjoey Road, Bilgola Beach**	REVISION: 0

Photo 1: Bedrock exposed in excavated batter alongside Barrenjoey Road. (Google Streetview March 2020)

Photo 2: Typical slope conditions across lower (southern) end of site

Photo 3: Bedrock exposed within pit located towards lower end of site

Photo 4: Typical conditions looking upslope along eastern site boundary

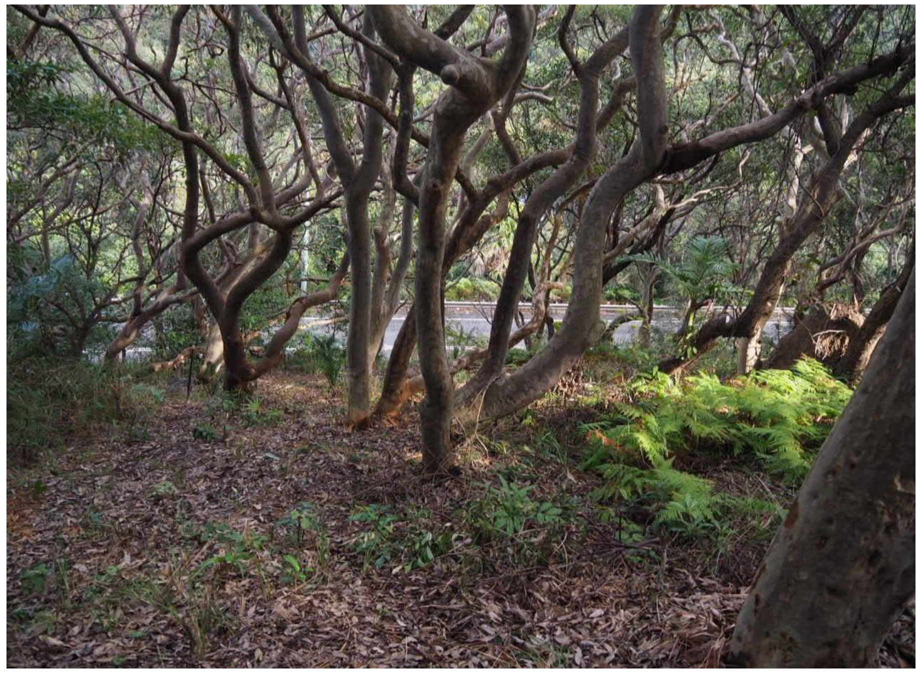


Photo 5: Typical conditions looking upslope along eastern site boundary

Photo 6: Timber log wall at northern boundary supporting upslope property

(Photos 2 to 6 taken on 11 June 2021)

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT:	Mr Peter Madew			TITLE:	**Site Photographs**	PROJECT No:	203454.00
	OFFICE:	Sydney	DRAWN BY:	DEM		**Geotechnical Assessment for Proposed New Residence**	PLATE No:	1
	SCALE:	NA	DATE:	Jun-21		**521 Barrenjoey Road, Bilgola Beach**	REVISION:	A

BOREHOLE LOG

CLIENT: Mr Peter Madew
PROJECT: Proposed New Residence
LOCATION: 521 Barrenjoey Road, Bilgola Beach

SURFACE LEVEL: 49.5 AHD*
EASTING: 344716
NORTHING: 6276124
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 203454.00
DATE: 3/4/2013
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0–0.6	NON CORE -(description not provided)					
0.6	SANDSTONE - very low strength, highly weathered, highly fractured to fractured, brown, fine grained sandstone. Some medium strength bands	0.75m: J50°, cu, ro, fe 1.17m: CORE LOSS: 200mm 1.5m to 1.67m: J85°-90°, ir, ro, cly 1.8m to 1.95m: Ds 150mm	C	93	38	PL(A) = 0.4
1.37						
2.03	SANDSTONE - medium then high strength, highly then highly to moderately weathered, fractured, brown, fine grained sandstone with some medium strength siltstone bands	2.11m: J35°, pl, sm, cly 5mm 2.16m: J25°, cu, ro, fe 2.58m: J30°, pl, sm, cly 2.73m: B5°, cly 2.78m: J70°, cu, ro, cly 3.03m to 3.12m: Ds 90mm				PL(A) = 0.8 PL(A) = 0.4
3.62		3.38m: CORE LOSS: 240mm 4.02m: J85°-90°, pl, ro, fe 4.36m: J50°, pl, ro, cly	C	90	75	PL(A) = 1.3
4.4	SILTSTONE/SANDSTONE - medium strength, moderately weathered, fractured, grey and brown siltstone/sandstone	4.87m: J40°, pl, ro, cln 5.07m: B5°, cly 5mm 5.1m: J85°-90°, un ro, fe				PL(A) = 0.8
5.35	SANDSTONE - medium and medium to high strength, moderately then slightly weathered, fractured grey and brown, fine grained sandstone. Some low and medium strength siltstone bands	5.35m: J45°, pl, sm, cly 5.7m: CORE LOSS: 200mm				PL(A) = 1.2
5.9		6m: J85°, un, ro, fe 6.75m: J20°, he,fe Unless specified defects are typically B0°-10°, fe or cly and J25°-50°, pl-un, ro fe and/or cly	C	93	79	PL(A) = 0.2 PL(A) = 1 PL(A) = 1
8.4	Bore discontinued at 8.4m					

RIG: PRO-LINE **DRILLER:** TIGHTSITE **LOGGED:** SI **CASING:**
TYPE OF BORING: Auger to 0.6m then NMLC coring
WATER OBSERVATIONS: Not recorded
REMARKS: Location co-ordinates in MGA 94 Zone 56; *Approximate level interpreted from Survey Drawing

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)

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

MR PETER MADEW
PROPOSED NEW RESIDENCE – BILGOLA BEACH

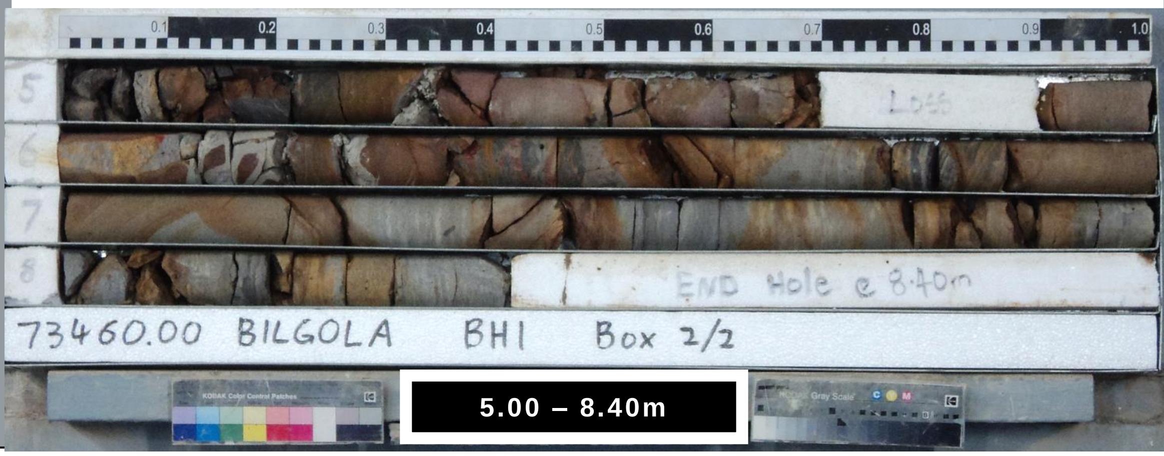
BORE 1 PROJECT 203454.00 MAY 2013

0.60 – 5.00m

DOUGLAS PARTNERS PTY LTD

MR PETER MADEW
PROPOSED NEW RESIDENCE – BILGOLA BEACH

BORE 1 PROJECT 203454.00 MAY 2013



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About this Report

Douglas Partners

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.

Sampling

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

Disturbed samples taken during drilling provide 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 it to obtain 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.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) 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 samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure 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 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. 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	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

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

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

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
Friable	Fr	-

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	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

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;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

  Soil tends to stick together.

  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.

  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it 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 Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

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

* 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
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

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 occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

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 stronger. 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

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

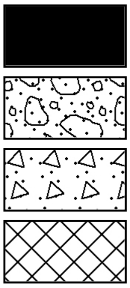
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

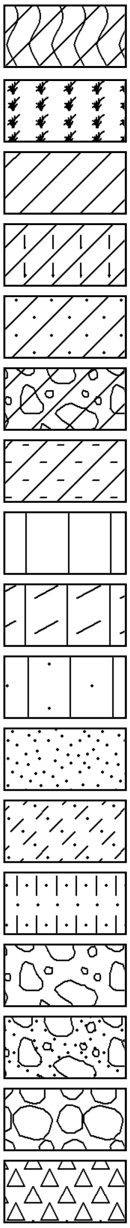
Graphic Symbols for Soil and Rock

General



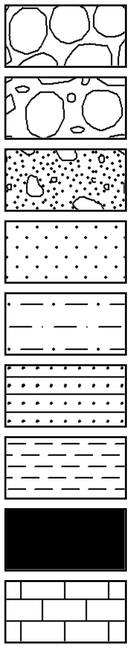
- Asphalt
- Road base
- Concrete
- Filling

Soils



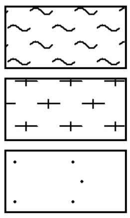
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



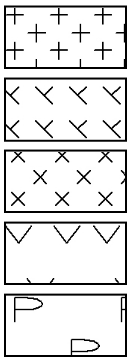
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks



- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

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.

PITTWATER COUNCIL

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for Mr Peter Madew
Name of Applicant
Address of site 521 Barrenjoey Road Bilgola Beach

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

I, ______________________ *on behalf of* ______________________________
(Insert Name) (Trading or Company Name)

on this the ______________________________ 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 $2million.
I have:

Please mark appropriate box

- ☐ 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
- ☑ I 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 am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.
- ☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Assessment - Proposed New Residence
Report Date: 28 June 2021
Author: David Murray
Author's Company/Organisation: Douglas Partners P/L

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

- Survey Dwg Ref 2861 DS (Iss.A - 4/3/21) - Richard + Loftus S.S.
- Architectural 2004-00 to 2004-14 (mostly 17-6-21) by Peter Downs Designs

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 ……[signature]……
Name …… Scott Easton ……
Chartered Professional Status…… CPEng ……
Membership No. …… 1371997 ……
Company…… Douglas Partners ……

PITTWATER COUNCIL

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

Development Application for Mr Peter Madew
Name of Applicant
Address of site 521 Barrenjoey Road, Bilgola Beach

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: Geotechnical Assessment - Proposed New Residence
Report Date: 28 June 2021
Author: David Murray
Author's Company/Organisation: Douglas Partners P/L

Please mark appropriate box

- ☑ Comprehensive site mapping conducted 2013, 11-6-21 (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 2013
- ☑ 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.

Signature ……[signature]……
Name …… Scott Easton ……
Chartered Professional Status …… CPEng ……
Membership No. …… 1371997 ……
Company …… Douglas Partners ……