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Report on
Geotechnical Investigation

Proposed New Residence
9 Ocean Road
Palm Beach

Prepared for
John Bubb & Christina Neumann-Bubb

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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
Author		08 January 2020
Reviewer	for John Braybrooke (JCB)	08 January 2020

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Report on Geotechnical Investigation
Proposed New Residence
9 Ocean Road, Palm Beach

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a new residence at 9 Ocean Road, Palm Beach. The investigation was commissioned in an email dated 10 October 2019 by John Bubb & Christina Neumann-Bubb the property owners and was undertaken in accordance with Douglas Partners' proposal SYD191075 dated 23 October 2019.

The construction of a new three-level residence is proposed, and site investigation was carried out to provide information on subsurface conditions for preliminary design of excavations, excavation support and building foundations.

The investigation comprised a detailed geotechnical investigation and site walk-over mapping of the property and accessible adjacent areas, along with a review of records held by this company for nearby sites, followed by the drilling of five boreholes.

Comments relating to geotechnical design issues and constraints are given below and are based on the results of the investigation.

2. Site Description and Regional Geology

Colour Plates 1 to 6 depicting some of the site features are included in Appendix A.

The site comprises a near rectangular, residential lot, 9 Ocean Road (Lot 22 D.P. 11552), measuring some 63 m along the northern boundary and 48 m along the southern boundary with the site approximately 21 m wide. The site is located on the high, western side of Ocean road, on the eastern foot-slopes of the hill forming the Barrenjoey Peninsula. The site has an existing residential dwelling located close to the boundary with Ocean Road (refer to Photo 1).

The total cross fall from the western (upslope) boundary to the eastern boundary at Ocean Road is in the order of 32 m, giving an average slope angle across the entire lot of approximately 25°. The site is bisected by a prominent 4 m to 6 m high, sub-vertical sandstone cliff line across the upper slope (refer to Photo 2). Typical slope angles above and below the cliff-line are in the order of 15° and 25° respectively.

The site is bounded by residential properties to the north, south and upslope to the west.

Reference to the Sydney 1:100 000 Geological Series Sheet 9130 indicates that the site is underlain by rocks of the Narrabeen Group of rocks, close to the boundary with the overlying Hawkesbury Sandstone. Both formations are of Triassic age, the Narrabeen Group comprising interbedded lithic sandstones, siltstones and shales and the Hawkesbury Sandstone generally comprising medium to coarse grained quartz sandstone.

The investigation confirmed the geological mapping with medium grained sandstone exposed within the cliff-line at the top of the site and as detached boulders on the slope below considered consistent with the Hawkesbury Sandstone. Similarly, fine grained micaceous sandstone and laminite encountered in the test bores on the lower slope is consistent with Narrabeen Group rocks.

3. Field Work Methods

The field work for the investigation included:

- drilling of five boreholes (BH01 to BH05) using a portable Proline geotechnical drilling rig at the locations shown on Drawing 1 in Appendix B (refer to Photo 5);
- boreholes were drilled into medium to high strength bed-rock to total depths ranging from 4.52 m to 11.43 m using diamond core drilling equipment to obtain continuous core samples of colluvial boulders and in-situ bedrock underlying the site;
- geological mapping was carried out by an senior engineering geologist to identify specific geomorphological features and to carry out a slope stability hazard risk assessment.

The boreholes were logged and sampled by an experienced engineering geologist. The rock cores recovered from the boreholes were photographed, followed by Point Load Strength Index (Is_{50}) testing on selected samples.

The ground surface levels and coordinates at the borehole locations were measured using a high precision differential GPS with an accuracy of about 0.1 m and are relative to Australian Height Datum (m AHD). BH5 location has been estimated as GPS equipment was not available due to the late addition of the borehole.

4. Field Work Results

4.1 Geological Mapping

The site was inspected by a senior engineering geologist on the 2 December 2019 and selected site features are noted on Drawings 1 and 2. The principal observations made during the inspections are given below:

- there was no evidence of significant cracking within the walls of the existing residence that could be attributed to previous slope movement;
- the eastern site boundary is located approximately 20m from sand dunes along the western side of Ocean Road;

- the front (eastern) yard of the lot is generally grassed and paved with some shrubs and small trees around the yard perimeter and a low sandstone retaining wall for garden planters. A paved drive way slopes up 1.3 m towards the residence, from Ocean Road;
- a two-story weather-board residence with basement garage and metal roof is located directly west of the eastern boundary (refer to Photo 1). The residence has a garage basement cut into the toe of the existing slope. A cemented sandstone flagging wall has been used to support the cut face. Large colluvial float stones are exposed within the sandstone flagging wall (refer to Photo 6). At the time of the inspection there was no groundwater seepage through the drainage holes in the sandstone flagging wall;
- the lower slope directly above the residence has been largely cleared of vegetation with only weeds and low height shrubs growing on the slope. The slope immediately behind is partially supported by a low sandstone wall. There are some detached sandstone boulders to around 2 m diameter stacked behind the north-west corner of the residence; (Photo 5);
- the sandstone cliff-line located on the upper slope is approximately 3 m to 5 m high and is partially over grown with trees, vines and weeds. The slope then grades gently up to the neighbouring property where a sandstone block wall is located;
- there are large angular sandstone blocks lying at the base of the cliff-line, on the slope below the cliff-line and above the existing residence (refer to Photos 1 and 3). The largest of these boulders is in excess of 7 m diameter. Heavily weathered surfaces of the boulders suggests that they have been in their present positions for many years, probably centuries;
- there is no evidence of any recent collapse from the sandstone cliff line (Photo 2). A open joint within the cliff-line infers that a section of the cliff-line is detached (refer to Photo 4). The block is supported on the colluvium slope below and is tilted back onto the sandstone cliff face but it has not been established whether this section of cliff-line is founded on bedrock or colluvium;
- jointing evident within the cliff line is typically dipping and striking obliquely out of the face to the north-east and north-west. Bedding planes dipping at various orientations at up to 15 degrees below horizontal are also evident. The joint/ bedding spacing is typically in the order of 3 m to 5 m;
- there was no evidence of active undercutting or any erodible layers within the cliff line, on the site, that could lead to large-scale collapse in the foreseeable future;
- the sandstone cliff-line is approximately 3 m to 5 m high and is partially over grown with trees, vines and weeds. The slope then grades gently up to the neighbouring property where a sandstone block wall is located;
- the neighbouring property directly to the north (8 Ocean Road) has been excavated into the colluvial slope. Large sandstone boulders (‘floaters’) are incorporated into the 4 m to 5 m high concrete block and cemented sandstone block retaining walls located behind the residence;
- an eroded sandstone cliff-line is visible behind the residence on the neighbouring property to the south (10 Ocean Road), above a paved outdoor area. It appears that rock bolts have been used to support the joint blocks on the cliff-line;
- a stormwater drain runs beyond the northern boundary down the slope. An overflow outlet is located half way up the lower slope which could potentially discharge uncontrolled flows during large rain fall events.

4.2 Subsurface Investigation

Details of the subsurface conditions encountered are given in the borehole logs in Appendix C, together with notes explaining descriptive terms and classification methods used. The sequence of subsurface materials encountered at the test locations is described below:

Fill	fill was encountered only in BH4 below the garage concrete slab to a depth of 0.7 m. The fill generally comprised pale-yellow-brown and orange sandy gravel with sandstone cobbles;
Colluvium	encountered in BH1 to BH3 and BH5 to depths of between 1.45 m and 6.58 m. The colluvium generally included large sandstone "float-stones" separated by pale grey and orange silty clay layers. The sandstone "float-stones" had bedding angles not consistent with insitu rock (either sub vertical to vertical);
Sandstone/ Laminite Bedrock:	medium to high strength fine to coarse grained sandstone or laminite of the Narrabeen Group Formation was located below the colluvium or, for BH4 below the fill material. The bedrock tended to be slightly fractured to unbroken to the termination depths of these boreholes.

The standing water level was measured in selected bores on 12 December 2019 and indicated ground water levels in BH2 and BH3 of 8.4 m and 6.45 m depth respectively below ground level. BH1 was blocked off at 2.6 m and was dry to that depth.

5. Comments

5.1 Proposed Development

The foot print of the proposed new development on the site is indicated on Drawing 1. It is understood that the proposed development on the site will comprise:

- A three level, brick and weatherboard residence with concrete floors, metal roof and timber decks located on the eastern and northern sides of the residence on the upper two levels. The residence will be excavated into the existing slope and minimal excavation below existing levels is anticipated.

- There is a stepped landscaped area to the rear of the residence with excavations of between 5 m to 6 m required into the existing slope to achieve the required final levels.

Stormwater drainage from the new development will be directed via pipes and appropriately sized detention tanks to the existing Council stormwater system.

5.2 Interpreted Geological Model

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

The development footprint is underlain by a layer of colluvial approximately 1.45 m to 6.58 m deep followed by sandstone and laminite bedrock. The colluvium layer appears to be thickest in the mid-section of the lower slope towards the northern boundary and shallows up across to the southern boundary where bed rock is exposed as weathered outcrop.

A sandstone cliff-line and outcrops are located at the top of the lower slope and appears to be the source of the colluvium which has gradually built up, over an extended period of time, down slope. Sandstone is near ground surface above the cliff-line across to the western neighbour's property.

5.3 Slope Stability Assessment

There is no evidence of recent significant instability on the subject site or on the adjacent residential lots to the north and south. Furthermore, the prevailing slope angle on the colluvium slope above the existing residence of around 30° coincides with the expected long-term angle for stability, provided the slope does not become saturated.

However, uncontrolled excavation into the slope or random disposal of stormwater could undermine the colluvium and bring about slope instability. Furthermore, the colluvium slope could be susceptible to ongoing long-term downhill creep movements irrespective of the degree of development undertaken.

Hence, careful planning of the proposed excavation and control of stormwater run-off from onsite and off-site sources will be required. Slope instability could occur if the planning of the development does not have due regard for the site conditions.

The presence of numerous large boulders on the site and on adjacent areas indicates that there is a possibility that other boulders will become detached from the cliff upslope and may roll or slide down onto the site. Inspection of the cliff face and the existing boulders, however, suggests that the likelihood of a boulder falling onto the site within the design life of the structure is rare, with an estimated annual probability of less than 1 in 10,000

5.4 AGS Slope Stability Risk Assessment

The risk of slope instability from hazards on the site has been assessed in accordance with the methods of the Australian Geomechanics Society (AGS) "Practice Note Guidelines for Landslide Risk Management, 2007". Identified hazards within and above the site are summarised in Tables 1, 2 and 3, together with an assessment of the likelihood of their occurrence after construction, the possible consequences and the calculated risk.

Table 1: Potential Hazard Identification

Hazard	Description	Potential Impact	Strategies to minimise occurrence or impact
1	Soil creep leading to very slow failure of piered foundations or retaining walls	Unlikely to occur as slope shows no signs of current soil creep and the proposed excavations are to be supported by permanent retaining structures	Design and install robust retaining structures as discussed in this report
2	Proposed buildings being hit by rapid detachment of boulders from the sandstone cliff line	Boulders unlikely to roll or slide as far as the proposed new residence due to the tabular shape of most blocks	Ensure regular checks of known outcrops and removal of loose blocks when identified.
3	Rapid gross instability occurring in slope arising from proposed excavation	Unlikely to affect new residence if soil and rock stabilisation as discussed in this report are implemented	Design and install robust retaining structures as discussed in this report

A qualitative assessment of likelihood, consequence and slope instability risk to the existing structures from the identified hazards is summarised in Table 2.

Table 2: Slope Instability Risk Assessment for Risk to Property

Hazard	Likelihood	Consequence	Risk
1 – Soil creep leading to very slow failure of piered foundations or retaining walls	Unlikely – following installation of engineered designed retaining walls and drainage measures	Minor to low	Low
2 – Proposed buildings being hit by rapid detachment of boulders from the sandstone cliff line	Rare - no recent block falls observed, outcrops located away from building	Medium to Major	Low
3 – Rapid gross instability occurring in slope arising from proposed excavation	Rare - if soil and rock stabilisation measures, as discussed in this report, are adopted	Major	Low

The key finding of this assessment is that if stormwater runoff is carefully controlled then the risk of instability of the batter slope and subsequent damage to the building below 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 3.

Table 3: 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 leading to very slow failure of piered foundations or retaining walls	1.0×10^{-4}	Person below retaining walls at time of collapse	0.2	0.1	1.0×10^{-4}	2.0×10^{-10}
2 – Proposed buildings being hit by rapid detachment of boulders from the sandstone cliff line	1.0×10^{-5}	Person downslope in the proposed new residence	0.1	0.3	0.1	3.0×10^{-9}
3 – Rapid gross instability occurring in slope arising from proposed excavation	1.0×10^{-5}	Person downslope in the proposed new residence	0.3	0.3	0.1	9.0×10^{-8}

When compared to the Landslide Risk Management Guidelines of the AGS, it is considered that the current site meets ‘Acceptable Risk Management’ criteria with respect to both property and life for established areas where possible landslide hazards have existed for many years under current and foreseeable conditions.

Provided that the slope is engineered and the development is constructed following recommendations outlined in this report there should be no increase to the geotechnical hazards identified in Tables 2 and 3.

5.5 Excavation Conditions

Reference to the supplied architectural drawings indicates that excavation into the lower slope for the new residence will require a 5 m – 6m high cut into the existing slope. There will also be some additional excavation above the new residence of similar depths to construct the landscaped pavement area.

The investigation has indicated that colluvium prevails over most of the proposed excavation with BH5 indicating some bedrock close to the surface along the southern boundary. Therefore, it is recommended that any existing retaining walls onsite be removed in short sections so that the sub-surface profile can be progressively uncovered, and the appropriate shoring system(s) determined.

Where found to be present, the colluvium will need to be progressively retained (removed or underpinned in the case of large sandstone boulders) as the existing walls are removed and the excavation extended. Some of the boulder-sized sandstone “float stones” near the edge of the proposed excavation footprint are probably bearing on colluvial soil.

Temporary battering of the colluvium along the sides of the excavation, where there is sufficient space available, should have a maximum slope of 1H:1V. Unsupported excavation and temporary battering of the colluvium below the cliff line should not be carried out, except as narrow slots, as this would risk undermining the large detached boulders on the slope above the site. Although not expected to be widespread within the excavation faces, in-situ bedrock of at least low strength is expected to stand unsupported in the short-term but retaining support should be provided for permanent excavations in this material.

Possible methods for supporting excavations in the soil and weathered bedrock include:

- construction of a temporary or permanent reinforced shotcrete wall at no steeper than 0.5:1 (H:V). The shotcrete wall should be anchored into the underlying bedrock.

- construction of permanent retaining walls comprising a vertical, temporary or permanent soldier pile and panel retaining wall, with either anchored or socketed, cantilevered pile.

It is suggested that the design of the retaining system be based upon an average bulk unit weight of 20 kN/m^3 and 22 kN/m^3 for soil and rock respectively. Walls on the sides of the excavation may be designed with a triangular earth pressure distribution using lateral earth pressure coefficients (for horizontal backfill conditions) as detailed in Table 5 and with horizontal pressures acting on the wall calculated using the following formula

Any retaining structures supporting the slope above the excavation, however, should be designed using an earth pressure coefficient (K) of 0.5 in the soils, to allow for the slope above the excavation. Alternatively, the structures may be designed to support an infinite slope at 30 degrees through the soils.

$$\sigma_z = K\, z\, \gamma$$

Where:

σ_z =Horizontal pressure at depth z
K =Earth pressure coefficient
Z =Depth (m)
Γ =Unit weight of soil and rock

Additional pressure should be allowed for where surcharging occurs either from existing or proposed building footings or other loadings, such as trees. All design assumptions for retaining structures should be carefully checked and confirmed by regular geotechnical inspections during the excavation.

It is expected that positive drainage measures will be incorporated in the construction of retaining walls to prevent water pressure build up behind the walls. Drainage is normally provided behind shotcrete walls in the form of strip drains.

An anchored sprayed concrete wall may provide adequate permanent structural support; however, it is recommended that a false wall (single brickwork or block work) also be installed as a facing for aesthetic purposes and to avoid dampness.

With the consent of adjacent property owners and giving due regard given to the location of underground services, soldier pile walls or shotcrete walls may be restrained laterally by a system of 'temporary' anchors and eventually propped by the new residential structure. Such temporary anchors would become redundant once the proposed structure is completed and would not place any restraints on future development or excavation of the adjacent sites. Anchors drilled into low strength bedrock (or better) may be designed using a maximum allowable bond stress of 600 kPa, with a minimum bond length of 3 m.

5.6 Vibration

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. Excavations within soil and low strength rock are not expected to generate excessive vibration. The use of heavy ripping and rock hammers for the large medium to high strength sandstone boulders may generate vibration which should be monitored.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVi for human comfort.

Based on the experience of DP and with reference to AS2670, it is suggested that a maximum PPVi of 8 mm/s (applicable at the foundation level of existing buildings/structures) be employed at this site for both architectural and human comfort considerations, although this vibration limit may need to be reduced if there are sensitive buildings, structures or equipment in the area.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

To minimise the effects of hydraulic rock hammer equipment, the work method should allow for:

- excavation of loose or rippable sandstone blocks by bucket or single tyne attachments prior to commencement of rock hammering. Care should be taken to ensure that existing, loosened blocks do not extend into adjacent sites.
- rock sawing around the perimeter of the excavation.
- progressive breakage from open excavated faces.
- selective breakage along open joints where these are present.
- use of rock hammers in short bursts to prevent generation of resonant frequencies.
- the movement of large blocks away from existing structures prior to breaking up for transport from site

5.7 Dilapidation Surveys

Dilapidation surveys should be carried out on adjacent buildings, pavements and infrastructure that may be affected by the excavation works. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that claims for damage due to construction related activities can be accurately assessed.

5.8 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014). This includes filling and natural materials that may be removed from the site. Accordingly, environmental testing will need to be carried out to classify spoil prior to transport from the site.

5.9 Foundations

Where possible, it is recommended that all new structures be supported on footings founded on in-situ bedrock. A combination of pad footings and bored piers may be adopted. Note bored piers may encounter medium or high strength sandstone boulders within the collouvium.

Foundation strata for the development will range from sandstone or laminite bedrock of generally medium strength exposed in the deepest sections of the basement, to very low strength, highly weathered sandstone or laminite further upslope where the development steps up the hillside.

Maximum allowable bearing pressures are suggested for pads or piers founded into sandstone or laminite bedrock (including contiguous piles or soldier piles) are given in Table 4.

Table 4: Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure	
	End Bearing (kPa)	Shaft Adhesion (Compression)* (kPa)	End Bearing (mPa)	Shaft Adhesion (Compression)* (kPa)
Very low strength sandstone /laminite	700	70	3	100
low strength sandstone /laminite	1,000	100	8	150
Medium strength or stronger sandstone/laminite	3,500	350	20	800

Confirmation of the actual bearing stratum will require geotechnical inspection during the construction of all the footings.

5.10 Ground Slabs

The floors at basement level and the landscaping areas to the rear of the property can be designed as slabs on ground, assuming proper compaction is given to the subgrade on which the slabs are cast. It may be necessary to over-excavate any large sandstone boulders or in-situ bedrock exposed within the floor of the cut and backfill with fine crushed rock to provide a uniform, trafficable and free draining surface. In any case, the subgrade surface should be compacted to at least 98% of the maximum density obtained in the laboratory standard compaction test prior to the casting of the slabs.

Under-floor drainage should be included to reduce the possibility of uplift pressures and the rise of moisture within the below ground sections of the development.

5.11 Stormwater and Groundwater

Groundwater seepage into the excavations will occur particularly following extended periods of wet weather. This seepage should be controlled by perimeter drains leading to a sump or sumps from which clean water could be pumped into the stormwater system. To this end, some treatment of groundwater or stormwater collected on site may be required to remove suspended solids and soluble iron prior to disposal off site.

Drainage measures will also be required immediately upslope of the proposed residence. These could comprise a concrete lined dish drain above the crest discharging to the sides of the excavation or through pipes down the face. Drainage should also be provided behind and through contiguous pile walls and any shotcrete applied to the excavation face, and along the base of the excavation face. All drainage from the excavation face and from above the crest should be connected to the site's stormwater disposal system, subject to appropriate treatment (if required).

Note that iron oxides will probably precipitate from the groundwater as a brown gelatinous sludge and allowances in the design of permanent drainage systems should be made for removal of this sludge, and for cleaning or flushing of pipework.

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 footing excavations for the new works to confirm compliance with the design with respect to allowable bearing pressure and stability.

7. Design Life and Requirement for Future Geotechnical Assessments

Douglas Partners Pty Ltd (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;
- existing and proposed retaining walls on the site.

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 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 5.

Note that the program given in Table 5 is provisional and is subject to review 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.
Existing or proposed retaining walls	Owner to check wall for deviation from as-constructed condition.	Every two to three 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).

8. References

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

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 9 Ocean Road, Palm Beach, in accordance with DP's proposal dated 23 October 2019. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of John Bubb & Christina Neumann-Bubb 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 on 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 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 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

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.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

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.

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

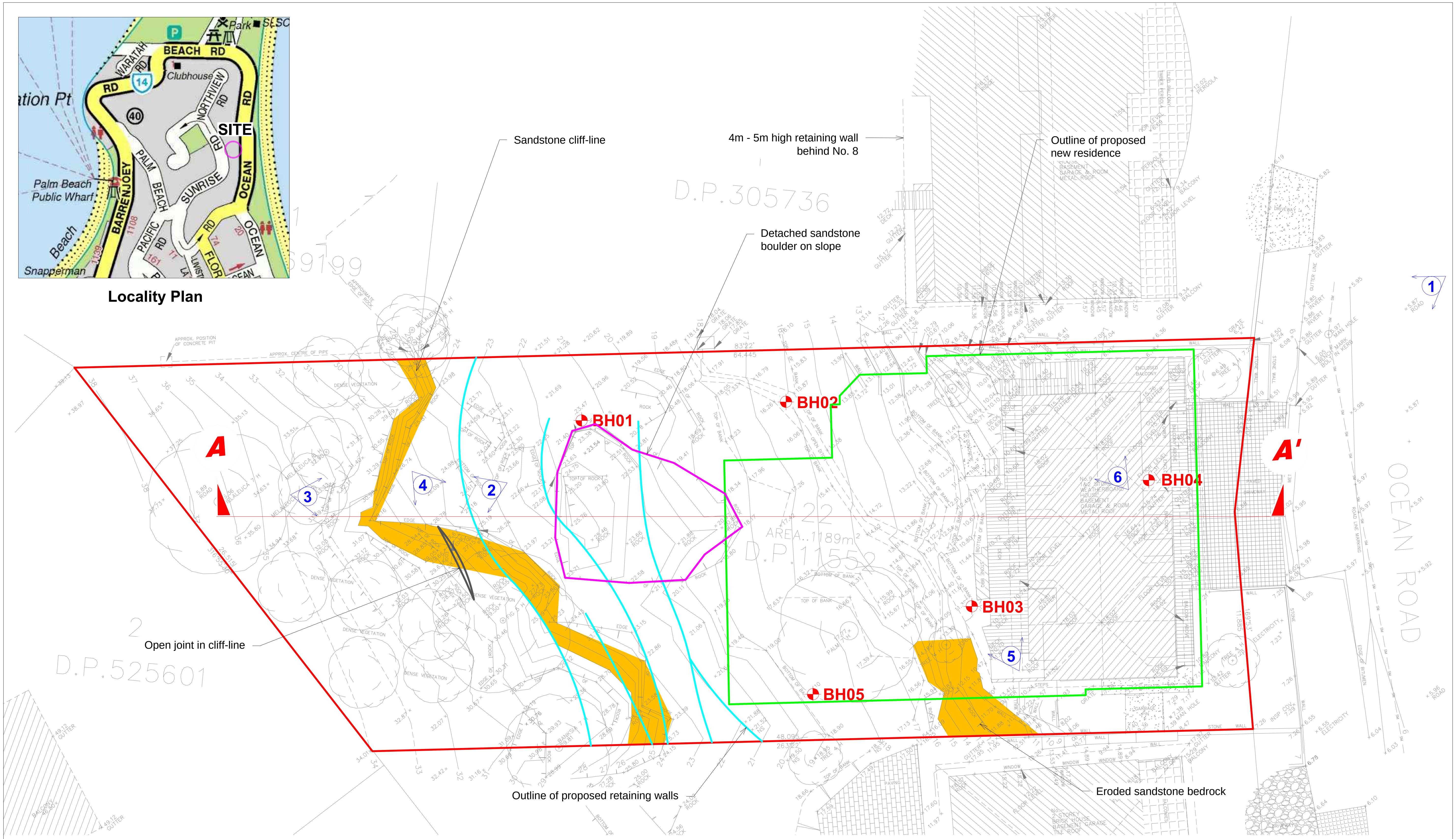
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

Appendix B

Site Photos
Drawings



0 1 2 3 4 6 8 10 15 20m

1:200 @ A3

LEGEND

● Borehole Locations

A A' Geotechnical Cross-section A-A'

1 Photo number with direction of view

NOTE:

1: Base image from Drawing No. 819B (dated 21.8.2019) provided by Clerke Surveyors Pty Ltd

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: John Bubb & Christina Neumann-Bubb

OFFICE: Sydney

DRAWN BY: IT

SCALE: 1:200 @ A3

DATE: 19.12.2019

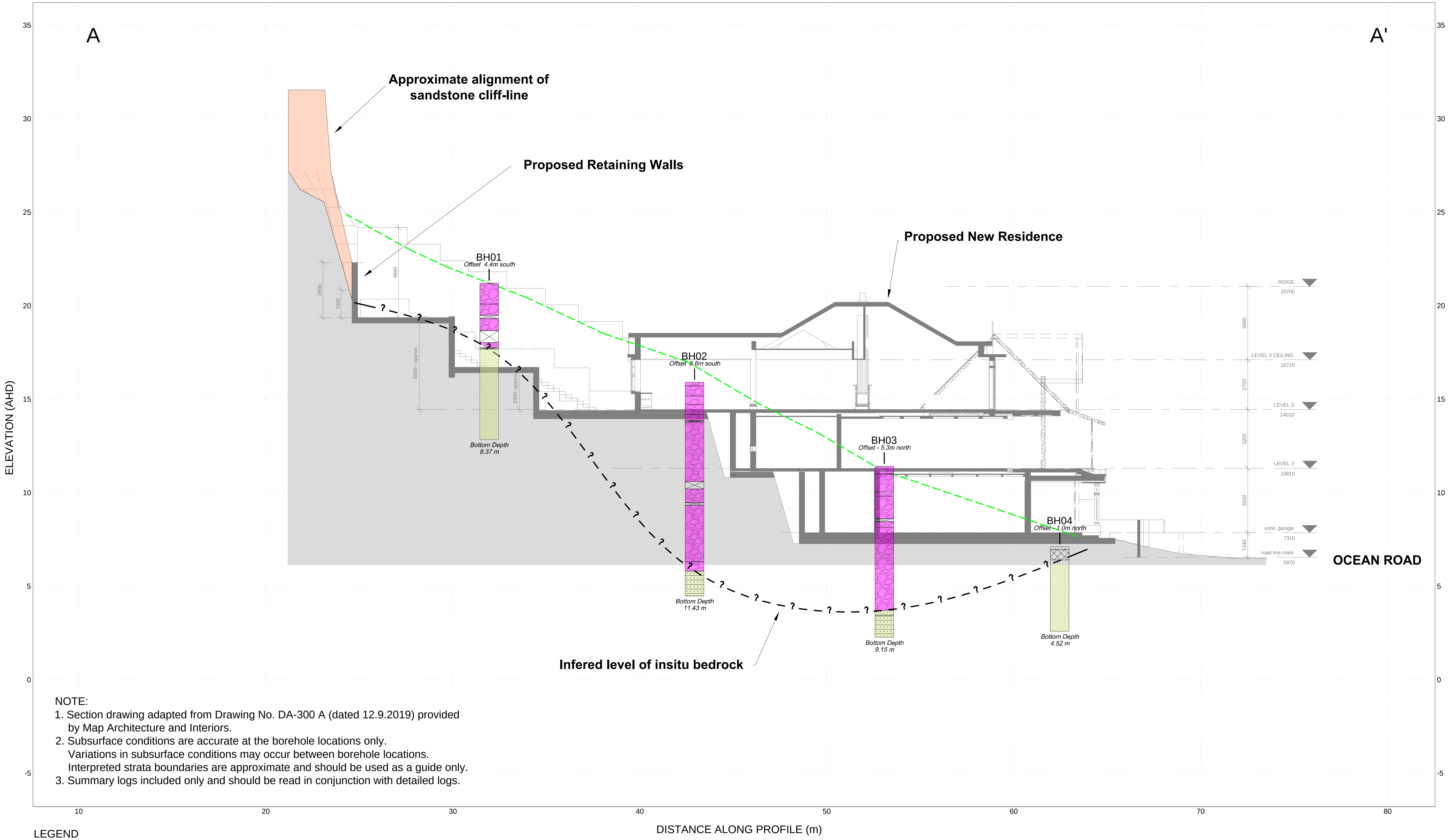
TITLE: **Geological Features and Location of Tests**
Proposed New Residence
9 Ocean Road, Palm Beach

N

PROJECT No: 86970.00

DRAWING No: 1

REVISION: 0



NOTE:
1. Section drawing adapted from Drawing No. DA-300 A (dated 12.9.2019) provided by Map Architecture and Interiors.
2. Subsurface conditions are accurate at the borehole locations only. Variations in subsurface conditions may occur between borehole locations. Interpreted strata boundaries are approximate and should be used as a guide only.
3. Summary logs included only and should be read in conjunction with detailed logs.

LEGEND

☒ Core Loss	▨ Laminite
▨ Colluvium	▨ Sandstone
▨ Concrete	
▨ Filling	

TESTS / OTHER

- - - - - Approximate Existing Ground Level

0 4

Horizontal Scale (metres)

Natural Scale

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: John Bubb & Christina Neumann-Bubb		TITLE: **Interpreted Geotechnical Cross-section A-A'** **Proposed New Residence** **9 Ocean Road, Palm Beach**	PROJECT No: 86970.00
OFFICE: Sydney	DRAWN BY: IT		DRAWING No: 2
SCALE: 1:200 (H) 1:200 (V) @ A3	DATE: 16.12.2019		REVISION: 0

Detached sandstone boulder on slope behind residence

Photo 1: View of existing residence (centre) from Ocean Road

Photo 2: Sandstone cliff-line located upslope of residence

Detached sandstone boulder on slope

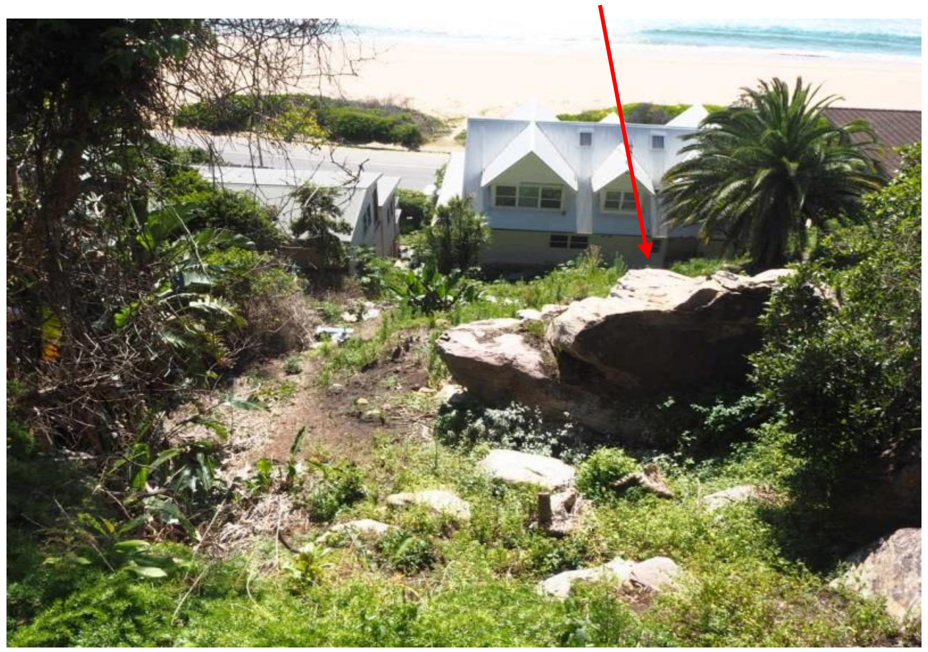


Photo 3: View from crest of clii-line looking east towards Ocean Road

Photo 4: Open joint in sandstone cliff-line

Photo 5: Drilling Bore 03 with portable equipment

Photo 6: Detached sandstone boulders ('floaters') visible in rear garage wall

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: John Bubb & Christina Neumann Bubb		TITLE:	**Geotechnical Assessment**	PROJECT No:	86970
OFFICE: Sydney	DRAWN BY: DEM		**Proposed New Residence**	PLATE No:	1
SCALE: NA	DATE: 3 Dec 2019		**9 Ocean Road, Palm Beach**	REVISION:	A

Appendix C

Field Works Results

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.

Soil Descriptions

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:

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

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

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
$\underline{\triangledown}$	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

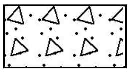
Symbols & Abbreviations

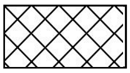
Graphic Symbols for Soil and Rock

General

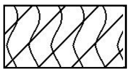

Asphalt

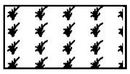

Road base

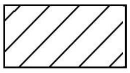

Concrete

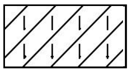

Filling

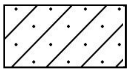
Soils

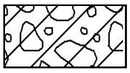

Topsoil

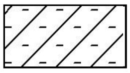

Peat


Clay

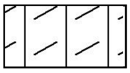

Silty clay

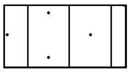

Sandy clay

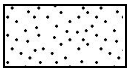

Gravelly clay

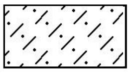

Shaly clay

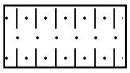

Silt


Clayey silt


Sandy silt

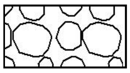

Sand

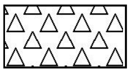

Clayey sand


Silty sand

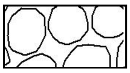

Gravel

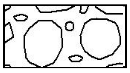

Sandy gravel


Cobbles, boulders

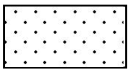

Talus

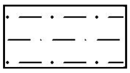
Sedimentary Rocks

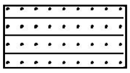

Boulder conglomerate

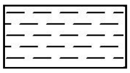

Conglomerate


Conglomeratic sandstone

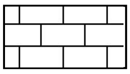

Sandstone


Siltstone

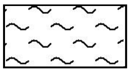

Laminite

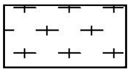

Mudstone, claystone, shale

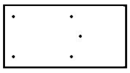

Coal


Limestone

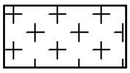
Metamorphic Rocks

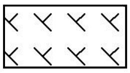

Slate, phyllite, schist

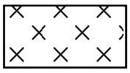

Gneiss

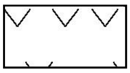

Quartzite

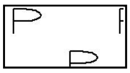
Igneous Rocks


Granite


Dolerite, basalt, andesite


Dacite, epidote


Tuff, breccia

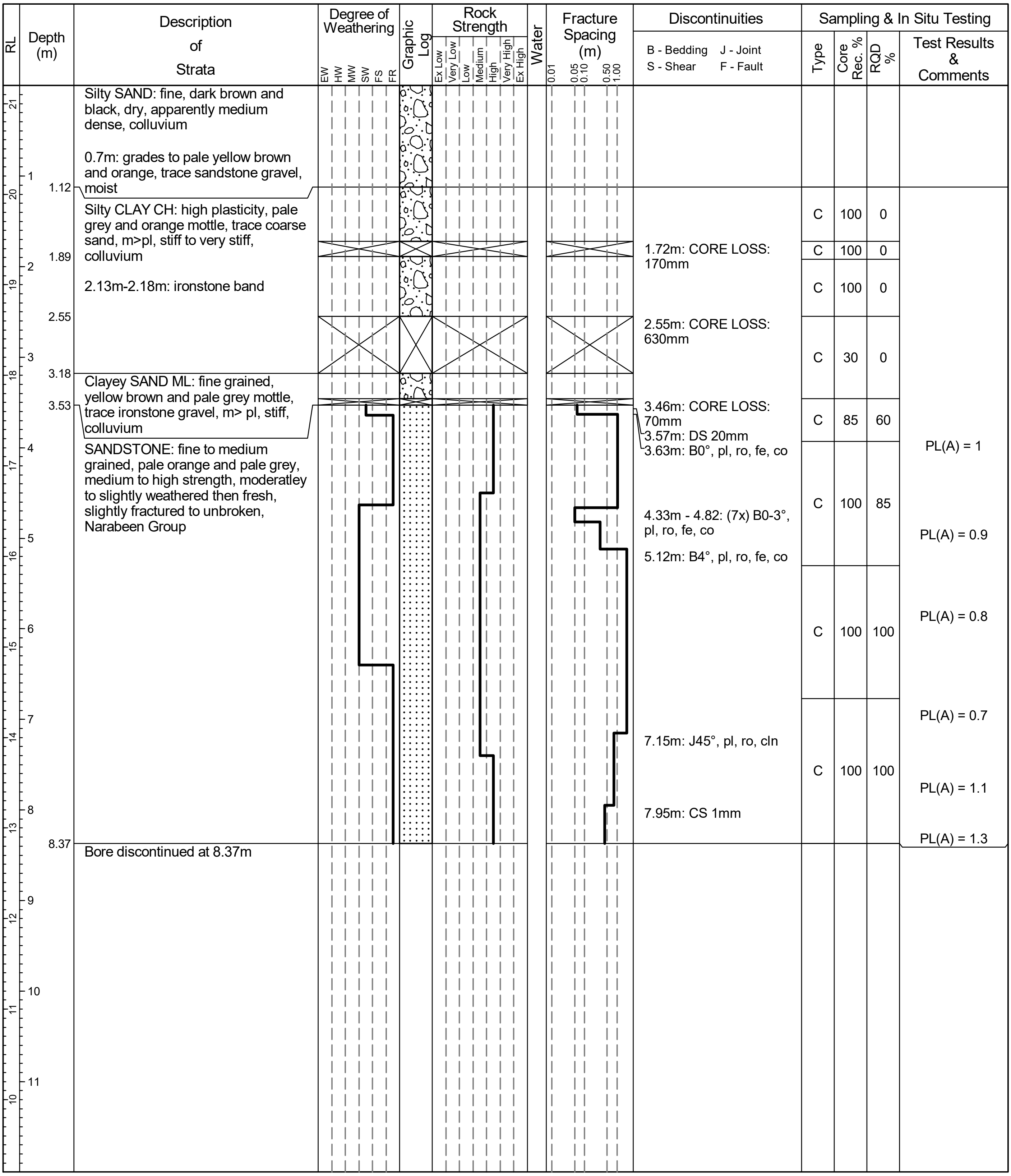

Porphyry

BOREHOLE LOG

CLIENT: John Bubb & Christina Neumann-Bubb
PROJECT: Proposed New Residence
LOCATION: 9 Ocean Road, Palm Beach

SURFACE LEVEL: 21.2 AHD
EASTING: 344461
NORTHING: 6281563
DIP/AZIMUTH: 90°/--

BORE No: BH01
PROJECT No: 86970.00
DATE: 2-12-2019
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
	Silty SAND: fine, dark brown and black, dry, apparently medium dense, colluvium 0.7m: grades to pale yellow brown and orange, trace sandstone gravel, moist					
1.12	Silty CLAY CH: high plasticity, pale grey and orange mottle, trace coarse sand, m>pl, stiff to very stiff, colluvium 2.13m-2.18m: ironstone band		C	100	0	
		1.72m: CORE LOSS: 170mm	C	100	0	
1.89			C	100	0	
2.55		2.55m: CORE LOSS: 630mm	C	30	0	
3.18	Clayey SAND ML: fine grained, yellow brown and pale grey mottle, trace ironstone gravel, m> pl, stiff, colluvium					
3.53	SANDSTONE: fine to medium grained, pale orange and pale grey, medium to high strength, moderatley to slightly weathered then fresh, slightly fractured to unbroken, Narabeen Group	3.46m: CORE LOSS: 70mm 3.57m: DS 20mm 3.63m: B0°, pl, ro, fe, co	C	85	60	PL(A) = 1
		4.33m - 4.82: (7x) B0-3°, pl, ro, fe, co 5.12m: B4°, pl, ro, fe, co	C	100	85	PL(A) = 0.9
			C	100	100	PL(A) = 0.8
		7.15m: J45°, pl, ro, cln 7.95m: CS 1mm	C	100	100	PL(A) = 0.7 PL(A) = 1.1 PL(A) = 1.3
8.37	Bore discontinued at 8.37m					

RIG: Proline **DRILLER:** Tightsite **LOGGED:** NB **CASING:** HQ to 1.0m

TYPE OF BORING: Hand auger to 1.0m, NMLC coring to 8.37

WATER OBSERVATIONS: No free groundwater observed whilst hand augering

REMARKS: 20% water loss from 6.5m onwards

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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BORE:BH01 PROJECT:86970.00 DECEMBER 2019

1.2-6.0m

BORE:BH01 PROJECT:86970.00 DECEMBER 2019

6.0-8.37m

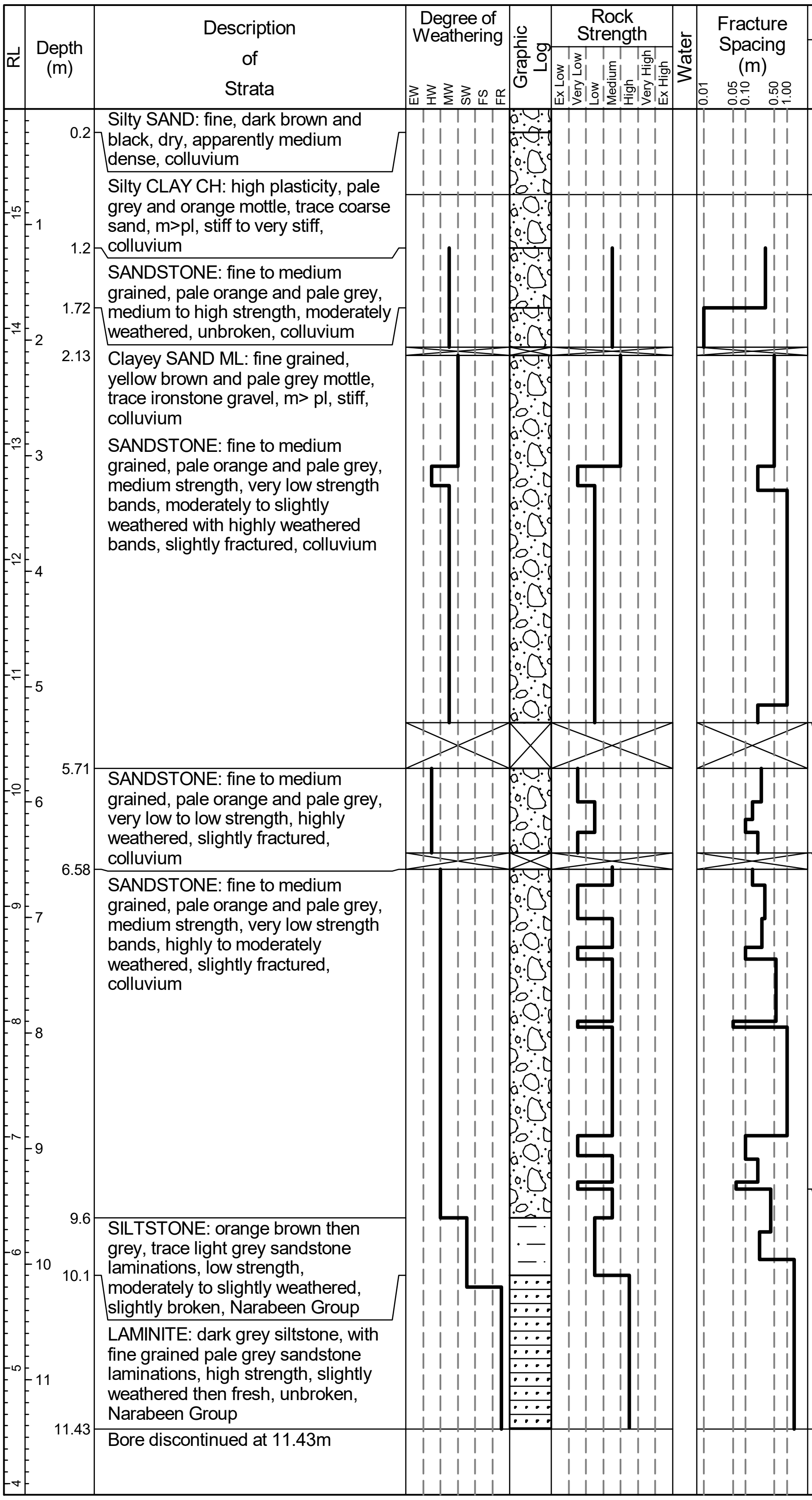
BOREHOLE LOG

CLIENT: John Bubb & Christina Neumann-Bubb
PROJECT: Proposed New Residence
LOCATION: 9 Ocean Road, Palm Beach

SURFACE LEVEL: 15.9 AHD
EASTING: 344472
NORTHING: 6281564
DIP/AZIMUTH: 90°/--

BORE No: BH02
PROJECT No: 86970.00
DATE: 3-12-2019
SHEET 1 OF 1

Depth (m)	Description of Strata
	Silty SAND: fine, dark brown and black, dry, apparently medium dense, colluvium
0.2	Silty CLAY CH: high plasticity, pale grey and orange mottle, trace coarse sand, m>pl, stiff to very stiff, colluvium
1.2	SANDSTONE: fine to medium grained, pale orange and pale grey, medium to high strength, moderately weathered, unbroken, colluvium
1.72	Clayey SAND ML: fine grained, yellow brown and pale grey mottle, trace ironstone gravel, m> pl, stiff, colluvium
2.13	SANDSTONE: fine to medium grained, pale orange and pale grey, medium strength, very low strength bands, moderately to slightly weathered with highly weathered bands, slightly fractured, colluvium
5.71	SANDSTONE: fine to medium grained, pale orange and pale grey, very low to low strength, highly weathered, slightly fractured, colluvium
6.58	SANDSTONE: fine to medium grained, pale orange and pale grey, medium strength, very low strength bands, highly to moderately weathered, slightly fractured, colluvium
9.6	SILTSTONE: orange brown then grey, trace light grey sandstone laminations, low strength, moderately to slightly weathered, slightly broken, Narabeen Group
10.1	LAMINITE: dark grey siltstone, with fine grained pale grey sandstone laminations, high strength, slightly weathered then fresh, unbroken, Narabeen Group
11.43	Bore discontinued at 11.43m



Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)

2.06m: CORE LOSS: 70mm
3.09m: DS 170mm
3.3m: J60°, pl, cly 10mm
5.16m: J45°, pl, cly 2mm
5.31m: CORE LOSS: 400mm
6.15m: J60°, pl, cly 2mm
6.44m: CORE LOSS: 140mm
8.0m - 8.6m: J80-90°, un, cly 10mm
8.98m: DS 80mm
9.29m: DS 60mm
9.35m: J45°, pl, ro, cln
9.72m: B2°, pl, ro, fe, co
9.96m: J40°, un, ro, fe, co

Sampling & In Situ Testing

Type	Core Rec. %	RQD %
C	95	95
C	100	90
C	100	100
C	65	23
C	91	91
C	100	60
C	100	100

Test Results & Comments:
PL(A) = 1.2
PL(A) = 0.4
PL(A) = 0.9
PL(A) = 0.4
PL(A) = 0.2
PL(A) = 0.4
PL(A) = 0.2
PL(A) = 2.9
PL(A) = 2

RIG: Proline **DRILLER:** Tightsite **LOGGED:** NB **CASING:** HQ to 1m
TYPE OF BORING: Hand auger to 1.0m,, NMLC coring to 11.43m
WATER OBSERVATIONS: No free groundwater observed whilst hand augering
REMARKS: 40% water loss from 6m onwards

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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BORE:BH02 PROJECT:86970.00 DECEMBER 2019

0.74-5.0m

BORE:BH02 PROJECT:86970.00 DECEMBER 2019

5.0-10.0m

BORE:BH02 PROJECT:86970.00 DECEMBER 2019

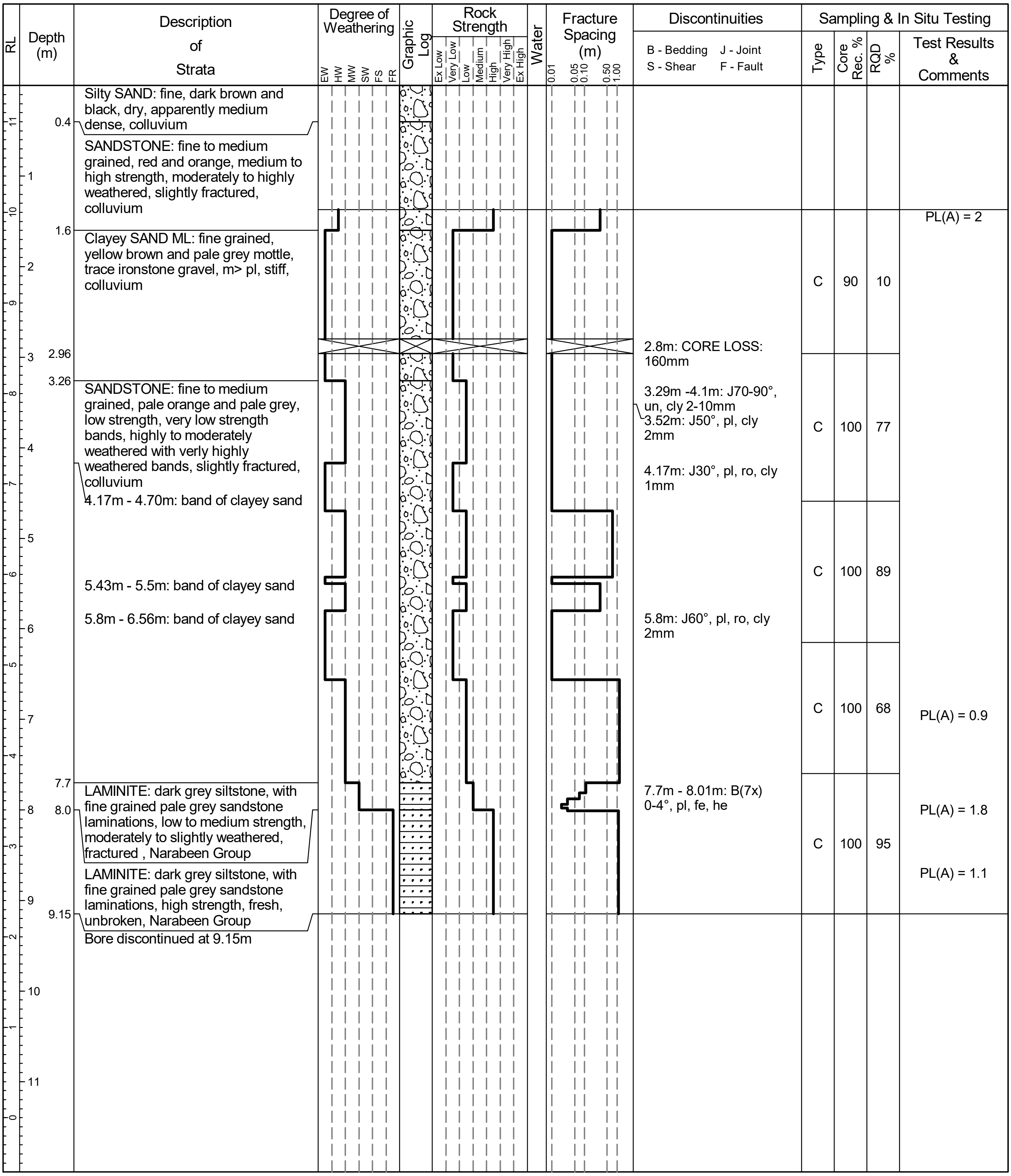
10-11.43m

BOREHOLE LOG

CLIENT: John Bubb & Christina Neumann-Bubb
PROJECT: Proposed New Residence
LOCATION: 9 Ocean Road, Palm Beach

SURFACE LEVEL: 11.4 AHD
EASTING: 344482
NORTHING: 6281553
DIP/AZIMUTH: 90°/--

BORE No: BH03
PROJECT No: 86970.00
DATE: 4-12-2019
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
0.4	Silty SAND: fine, dark brown and black, dry, apparently medium dense, colluvium					
1.6	SANDSTONE: fine to medium grained, red and orange, medium to high strength, moderately to highly weathered, slightly fractured, colluvium					PL(A) = 2
2.96	Clayey SAND ML: fine grained, yellow brown and pale grey mottle, trace ironstone gravel, m> pl, stiff, colluvium		C	90	10	
3.26		2.8m: CORE LOSS: 160mm				
	SANDSTONE: fine to medium grained, pale orange and pale grey, low strength, very low strength bands, highly to moderately weathered with verly highly weathered bands, slightly fractured, colluvium 4.17m - 4.70m: band of clayey sand	3.29m -4.1m: J70-90°, un, cly 2-10mm 3.52m: J50°, pl, cly 2mm 4.17m: J30°, pl, ro, cly 1mm	C	100	77	
	5.43m - 5.5m: band of clayey sand 5.8m - 6.56m: band of clayey sand	5.8m: J60°, pl, ro, cly 2mm	C	100	89	
7.7			C	100	68	PL(A) = 0.9
8.0	LAMINITE: dark grey siltstone, with fine grained pale grey sandstone laminations, low to medium strength, moderately to slightly weathered, fractured , Narabeen Group	7.7m - 8.01m: B(7x) 0-4°, pl, fe, he				PL(A) = 1.8
9.15	LAMINITE: dark grey siltstone, with fine grained pale grey sandstone laminations, high strength, fresh, unbroken, Narabeen Group		C	100	95	PL(A) = 1.1
	Bore discontinued at 9.15m					

RIG: Proline **DRILLER:** Tightsite **LOGGED:** NB **CASING:** HQ to 1.3m
TYPE OF BORING: Hand auger to 1.3m, NMLC coring to 9.15m
WATER OBSERVATIONS: No free groundwater observed whilst hand augering
REMARKS:

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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BORE:BH03 PROJECT:86970.00 DECEMBER 2019

1.37-6.0m

BORE:BH03 PROJECT:86970.00 DECEMBER 2019

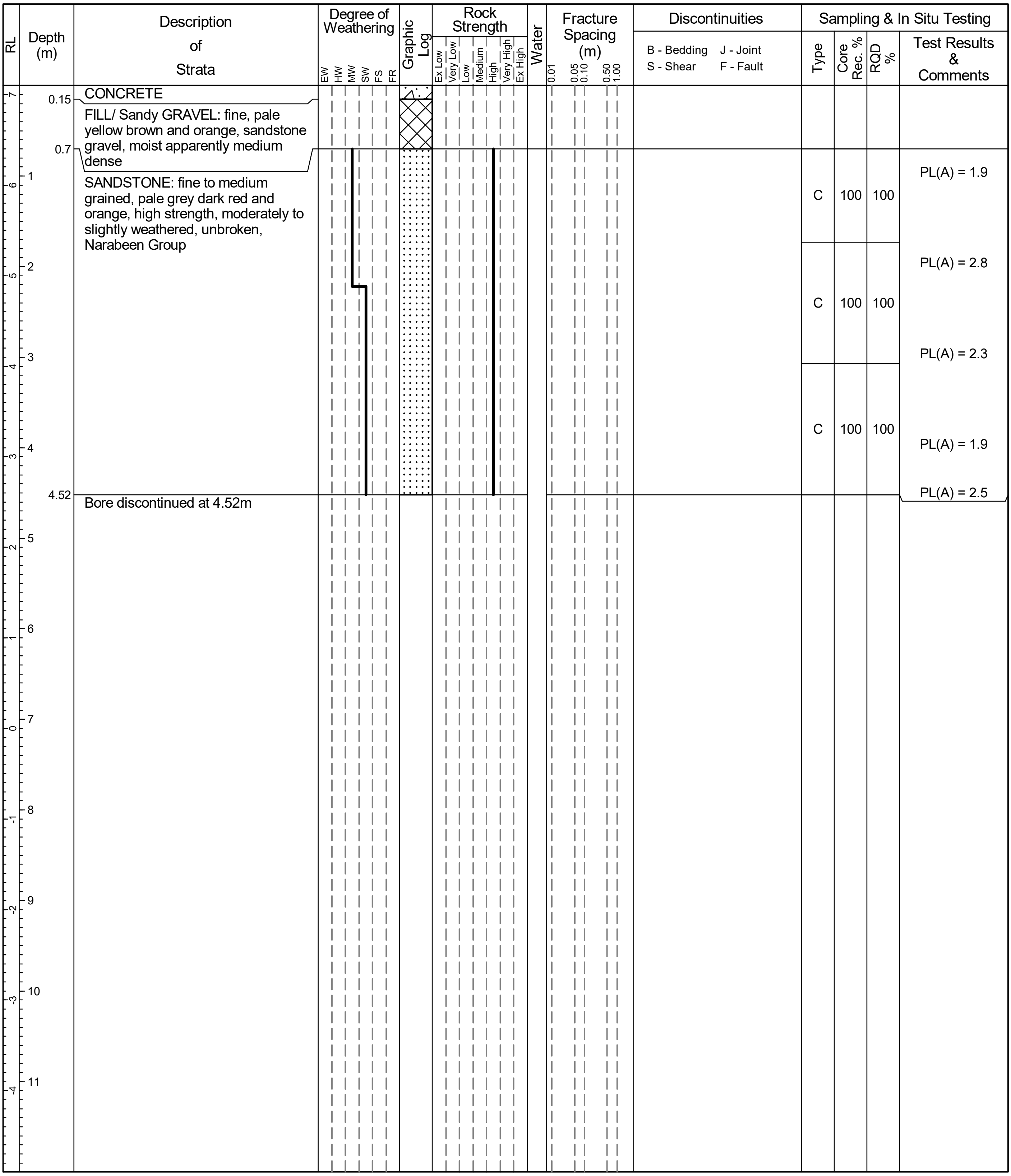
6.0-9.15m

BOREHOLE LOG

CLIENT: John Bubb & Christina Neumann-Bubb
PROJECT: Proposed New Residence
LOCATION: 9 Ocean Road, Palm Beach

SURFACE LEVEL: 7.1 AHD
EASTING: 344495
NORTHING: 6281558
DIP/AZIMUTH: 90°/--

BORE No: BH04
PROJECT No: 86970.00
DATE: 6-12-2019
SHEET 1 OF 1



Depth (m)	Description of Strata	Type	Core Rec. %	RQD %	Test Results & Comments
0.15	CONCRETE				
0.7	FILL/ Sandy GRAVEL: fine, pale yellow brown and orange, sandstone gravel, moist apparently medium dense				
	SANDSTONE: fine to medium grained, pale grey dark red and orange, high strength, moderately to slightly weathered, unbroken, Narabeen Group	C	100	100	PL(A) = 1.9
		C	100	100	PL(A) = 2.8 PL(A) = 2.3
4.52		C	100	100	PL(A) = 1.9 PL(A) = 2.5
	Bore discontinued at 4.52m				

RIG: Proline **DRILLER:** Tightsite **LOGGED:** NB **CASING:** HQ to 0.7m
TYPE OF BORING: Diatube to 0.15m, Hand auger to 0.7m, NMLC coring to 4.52m
WATER OBSERVATIONS: No free groundwater observed whilst hand augering
REMARKS: 100% water loss from 0.9m onwards

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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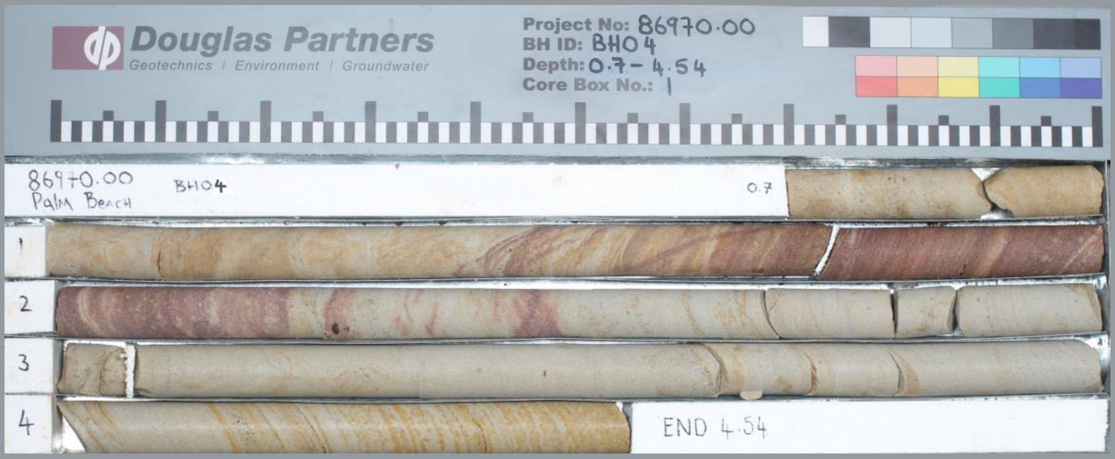
BORE:BH04 PROJECT:86970.00 DECEMBER 2019



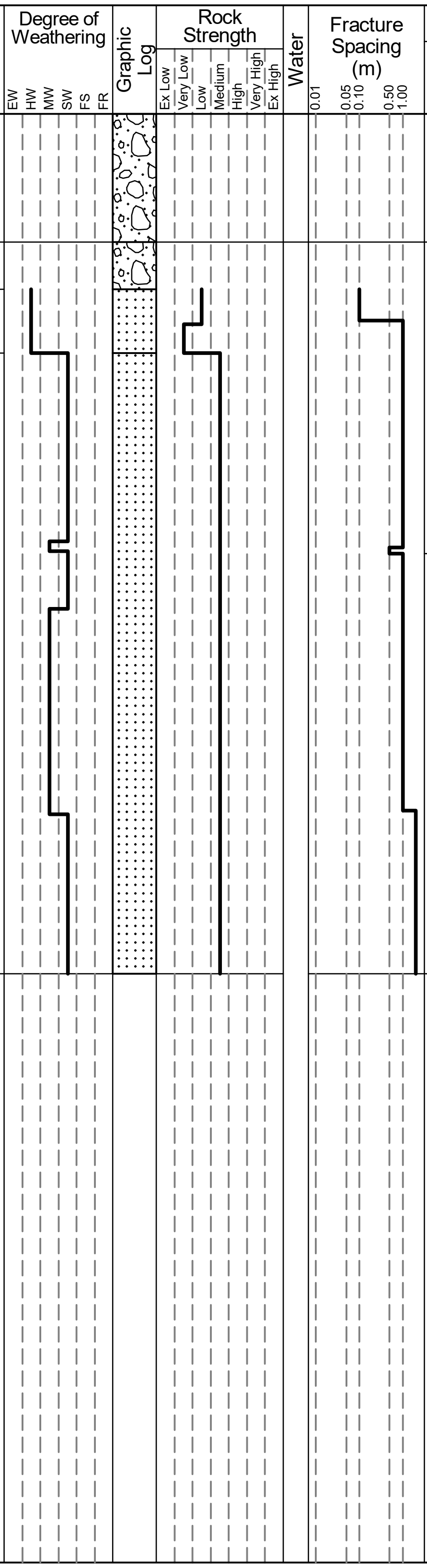
0.7-4.54m

BOREHOLE LOG

CLIENT: John Bubb & Christina Neumann-Bubb
PROJECT: Proposed New Residence
LOCATION: 9 Ocean Road, Palm Beach

SURFACE LEVEL: 17.5 AHD
EASTING: 344475
NORTHING: 6281551
DIP/AZIMUTH: 90°/--

BORE No: BH05
PROJECT No: 86970.00
DATE: 5-12-2019
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
	Silty SAND: fine, dark drown and black, dry, apparently medium dense, colluvium					
1.06	Silty CLAY CH: high plasticity, pale grey and orange mottle, trace coarse sand, m>pl, stiff to very stiff, colluvium					
1.45	SANDSTONE: fine to medium grained, pale orange and pale grey, very low to low strength, highly weathered, fractured, Narabeen Group	1.45m - 1.71m: B(x7), 0°, un, fe, co	C	100	71	
1.98	SANDSTONE: fine to medium grained, pale orange and pale grey and red, medium to high strength, moderately to slightly weathered, slightly fractured, Narabeen Group	3.59m: B0°, pl, ro, fe, st 3.64m: DS 10mm	C	100	90	
			C	100	100	PL(A) = 0.7
		5.77m: B0°, pl, ro, fe, co	C	100	100	PL(A) = 0.95 PL(A) = 0.9
7.12	Bore discontinued at 7.12m					

RIG: Proline **DRILLER:** Tightsite **LOGGED:** NB **CASING:** HQ to 1.0m
TYPE OF BORING: Hand auger to 1.0m, , NMLC coring to 7.12m
WATER OBSERVATIONS: No free groundwater observed whilst hand augering
REMARKS: RL heights estimated from Google Earth

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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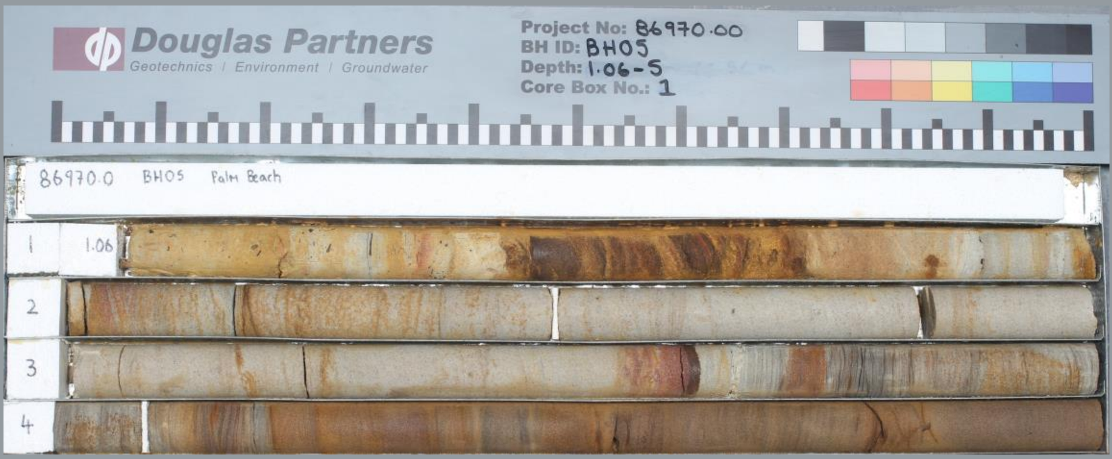
BORE:BH05 PROJECT:86970.00 DECEMBER 2019



1.06-5.0m

BORE:BH05 PROJECT:86970.00 DECEMBER 2019

5.0-7.12m

Appendix D

AGS Guidelines

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

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.

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.

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

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.

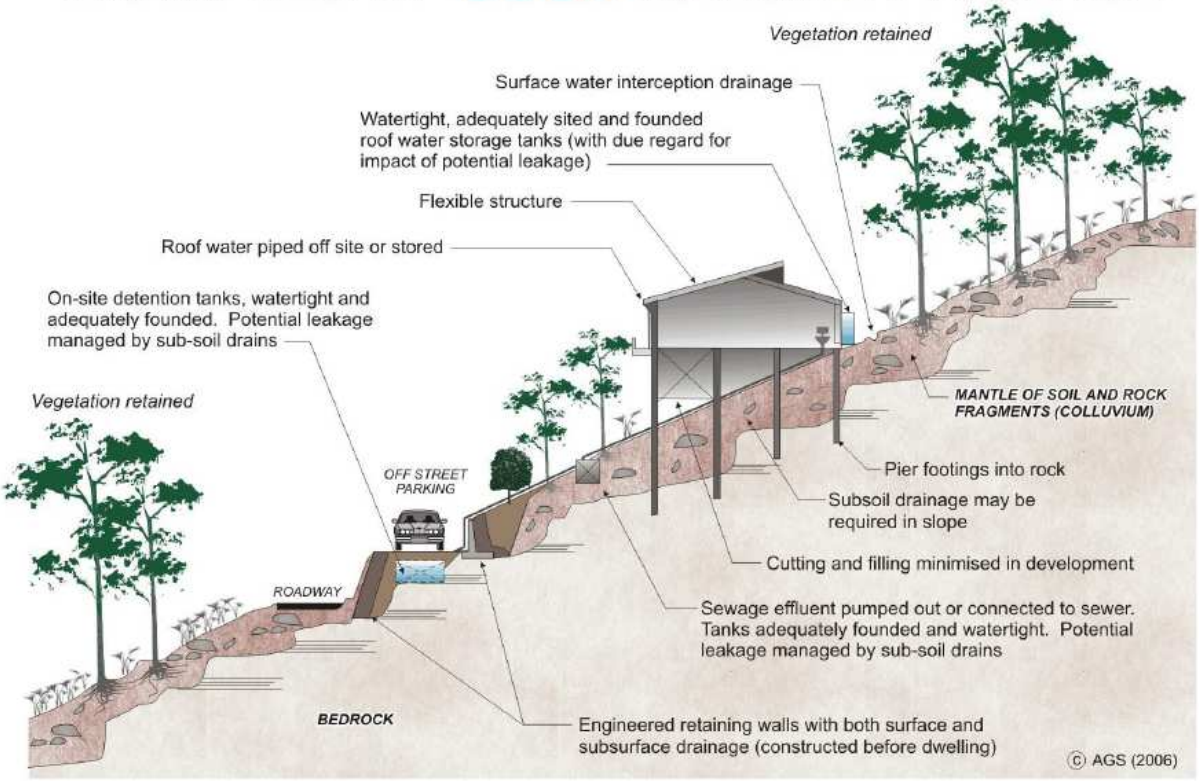
(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional 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

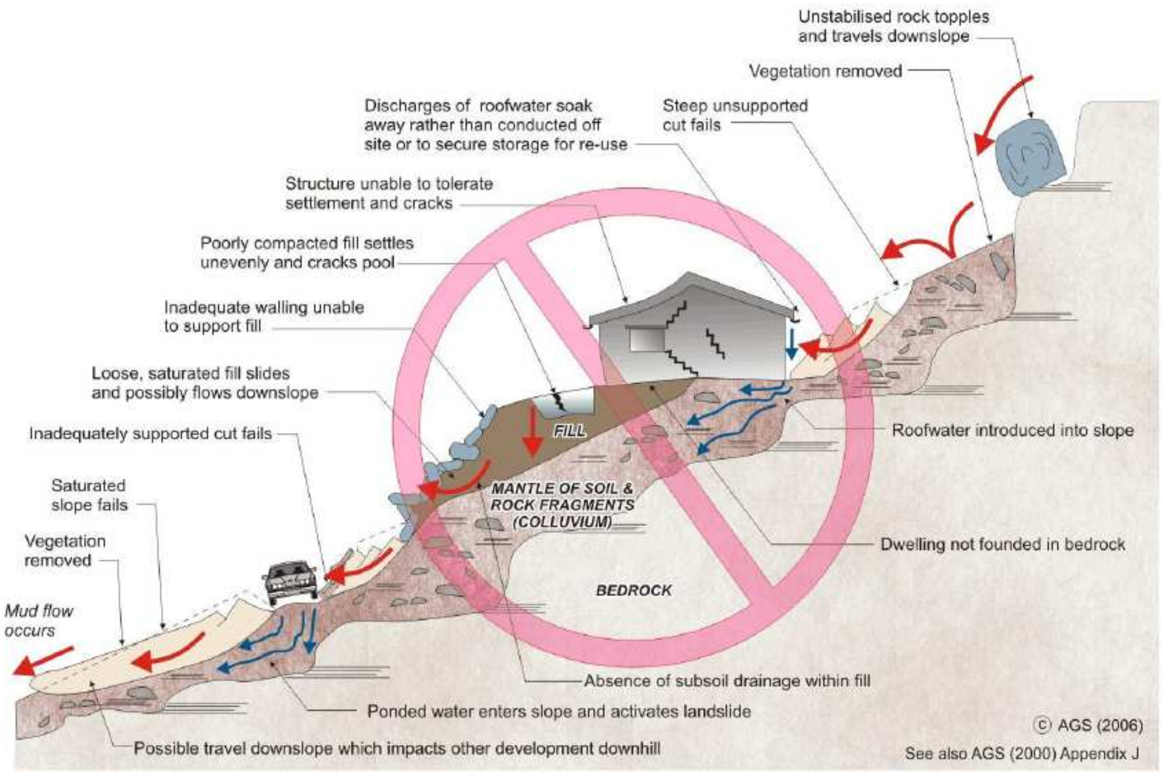
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE



GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for John Bubb + Christina Neumann - Bubb
Name of Applicant

Address of site 9 Ocean Road Palm Beach

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 8 January 2020 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:

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: Proposed New Residence
Report Date: 8.1.20
:
Author: Nick Burrows
Author's Company/Organisation: Douglas Partners Pty LtD.

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

DA 001-004 / 101-104 / 201-202 / 301-303 / 401 / 501
Rev D 2020-01-06 (MAP) architecture + Interiors
Adam Clerke Survey Plan Job 819B 21.8.19

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
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 John Bubb + Christina Neumann-Bubb
Name of Applicant

Address of site 9 Ocean Road, Palm 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: Proposed New Residence
Report Date: 8.1.20
Author: Nick Burrows
Author's Company/Organisation: Douglas Partners Pty Ltd

Please mark appropriate box

☑ Comprehensive site mapping conducted 2.12.19
(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 2.12.19

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