Douglas Partners
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Report on Geotechnical Investigation

Proposed Mixed Use Development
231 Whale Beach Road, Whale Beach

Prepared for
Leslie Cassar

Project 45636.01
September 2019

Douglas Partners
Geotechnics / Environment / Groundwater

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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	PP. [signature] FOR GEOFF YOUNG	27 September 2019
Reviewer	[signature]	27 September 2019

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

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Report on Geotechnical Investigation
Proposed Mixed Use Development
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1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed mixed use development at 231 Whale Beach Road, Whale Beach. The assessment was commissioned in an email dated 22 August 2019 by Karla Wilford from Richard Cole Architects (RCA) on behalf of the owner, Leslie Cassar, and was undertaken in accordance with Douglas Partners' proposal SYD180588 dated 22/08/2018 .

It is understood that the proposed development will involve construction of a new five-storey building including a single level for basement carparking. The basement level will require excavation into the slope to depths of approximately 14 m on the western end of the site with the depth of excavation reducing to approximately 3 m on the eastern end of the site. The assessment is required for planning purposes and for submission with a development application to Northern Beaches Council.

Douglas Partners Pty Ltd (DP) has previously carried out a geotechnical investigation for Leslie Cassar on the site in September 2008 (DP Report No.45636.00, dated 4 September 2008) for a previously proposed development, which did not proceed. Since that time, the proposed development has been revised. This report is using the field work results from the September 2008 to address the geotechnical issues for the currently proposed development.

The previous investigation included a site inspection by a senior geotechnical engineer, drilling of three rock cored boreholes and installation of two groundwater monitoring wells for measurement of groundwater levels. Laboratory testing of selected rock core samples was undertaken, followed by engineering analysis and reporting. Details of the field work are summarised in this report, together with comments on design and construction issues.

Information provided for use in this report include architectural drawings by RCA (DA00 to DA56, dated July 2019) and a site survey plan (three sheets) by Rygate and Company Pty Limited (Drawing No. 78055, dated 21.4.17).

In accordance with current Northern Beaches (Pittwater) Council's Geotechnical Risk Management Policy the site lies within Hazard Zone 1. This assessment has been prepared in accordance with the Geotechnical Risk Management Policy and includes the following:

- determination of a geotechnical model of the inferred subsurface profile;
- identification, description and reporting of geotechnical hazards; and
- risk assessment for property and life.

2. Site Description

A site locality plan (Drawing 1) is included in Appendix C and shows the site and surrounding area. The site is located toward the base of an east-facing hill which falls toward Whale Beach which is located about 50 m to 100 m to the east of the site. The site is an irregular-shaped lot covering an area of 844 m^2 with a 30 m long western frontage to Whale Beach Road and a 30 m long eastern frontage to Surf Road.

Within the site, ground surface levels fall to the east from approximately RL 21 m to RL 9 m, relative to Australian Height Datum (AHD), with an average slope of approximately 15 degrees. The ground slope reduces to approximately 5 degrees to the east of the site, between Surf Road and Whale Beach.

At the time of the previous investigation the site was occupied by a one to two-storey brick building on the western end of the site and a three to four-storey brick building on the eastern part of the site. A brick paved footpath approximately 4 m wide was located between Whale Beach Road and the western boundary. The eastern end of the site comprised a series of terraced areas formed by brick and concrete retaining walls approximately 1 m to 2 m high. The retaining walls on the eastern part of the site were generally in poor condition with cracking and obvious signs of rotation and movement observed.

On the adjacent properties to the north and south of the site are one to two-storey brick houses set back approximately 4 m from the common boundaries. A concrete block wall extended along the northern boundary. A sandstone block and concrete block retaining wall approximately 0.5 m to 1.5 m high extended along the southern boundary (retaining the soil to the north).

3. Geology

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is located near the intersection of Hawkesbury Sandstone which typically comprises medium to coarse grained quartz sandstone with some shale bands or lenses and the Newport Formation which typically comprises interbedded shale, laminite and sandstone. The previous field work confirmed the mapping and indicated the site is underlain by Hawkesbury Sandstone on the western part of the site and possibly Newport Formation (interbedded sandstone and laminate) on the lower, eastern end of the site.

4. Field Work Methods

The field work for the previous 2008 investigation included a site inspection by a senior geotechnical engineer, three boreholes (BH 1 to BH 3 inclusive), four dynamic cone penetrometer tests (DCP 4 to DCP 7 inclusive) and installation of two groundwater monitoring wells.

The boreholes were drilled to depths of 7.5 m to 14.0 m using a bobcat-mounted drilling rig. The boreholes were initially drilled using spiral augers and rotary wash boring within the soil and extremely weathered rock to depths of 4.0 m to 4.7 m. They were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the rock. Standard Penetration Tests (SPT's) were carried out at 1.5 m depth intervals to sample the soil and assess the in-situ strength of the materials.

Soil samples and rock cores were returned to the DP office where they were logged by a geologist, the cores photographed and Point Load Strength Index (Is50) tests carried out on selected samples of the rock core.

The DCPs were taken to refusal at depth of 1.3 m to 2.3 m. The DCP test involves driving a 16 mm diameter steel rod with a 20 mm diameter cone tip into the ground using a 9 kg hammer falling freely over a height of 510 mm. The number of blows required to penetrate each successive 150 mm interval is measured and used to assess the soil strength.

Groundwater observations were made during auger drilling of the boreholes and groundwater monitoring wells (50 mm diameter slotted PVC) were installed in BH 2 and BH 3 to allow for future measurement of groundwater levels. The groundwater levels within the monitoring wells were measured on 3/7/08 and again on 1/9/08. No long term monitoring of groundwater levels was carried out.

The ground surface levels at the test locations were interpolated from spot levels shown on the survey plan by Rygate & Company Pty Ltd (Ref. 72649, dated 15/5/07).

5. Field Work Results

Details of the conditions encountered in the bores are given in the borehole logs in Appendix B, together with colour photographs of the rock core samples, DCP results and notes defining classification methods and descriptive terms. The test locations are shown on Drawing 2 in Appendix C.

The boreholes penetrated a subsurface profile comprising fill to depths of up to 3 m over clayey sand then sandstone bedrock at depths of between 2.0 m to 4.7 m. The various strata are summarised below.

Pavements 100 mm and 130 mm thick concrete in BH 1 and BH 3 respectively and 50 mm thick brick pavers over concrete 90 mm thick in BH 2.

Fill to depths of 2.3 m and 3.0 m in BH 1 and BH 2 respectively. The fill generally comprised sand with inclusions of gravel, clay and organic material. The SPT results within the fill correspond with loose sandy soils.

Clayey Sand comprising medium dense clayey sand in BH 1 to a depth of 4.7 m (RL 16.5 m) and loose clayey sand to a depth of 2.2 m (RL 6.3 m) in BH 3.

Sandstone encountered in BH 1, BH 2 and BH 3 at depths of 4.7 m (RL 16.5 m), 3.0 m (RL 17.8 m) and 2.2 m (RL 6.3 m) respectively. The rock generally comprised extremely low to very low strength sandstone approximately 1.5 m to 2.0 m thick (BH 2 and BH 3 only) over medium and high strength, slightly fractured and unbroken sandstone. The sandstone in BH 3 included thick bands of medium strength laminite (interbedded fine grained sandstone and shale). The rock cores included some joints with dips ranging from 45 to 85 degrees below the horizontal plane.

Seepage was observed during auger drilling at a depth of 2.5 m in BH 2 and 2.0 m in BH 3. No seepage was observed during auger drilling in BH 1. Groundwater levels within the standpipes were measured at depths 3.0 m in BH 2 and 1.2 m in BH 3 on 1/9/08. The observed groundwater is most likely running along the top of the clayey sand and rock surface.

6. Point Load Strength Index Testing

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index ($I_{s(50)}$) values. The results of the testing are shown on the borehole logs at the corresponding depth.

The I_{s50} values for the rock have been used to estimate the unconfined compressive strength (UCS) based on a UCS:$I_{s(50)}$ ratio of 1:20. The $I_{s(50)}$ values for the rock cores ranged from approximately 0.6 MPa to 2.0 MPa, corresponding to a medium to high strength classification (estimated UCS ranging from 12 MPa to 40 MPa).

7. Geotechnical Model

Two geological cross sections (Sections A-A' and B-B') showing the interpreted subsurface profile between test locations are shown on Drawings 3 and 4 in Appendix C. The orientations of the cross-sections are shown on Drawing 2. The sections show interpreted geotechnical divisions of underlying soil and rock together with the extent of the proposed new building and excavation.

The interpreted geological model for the site comprises:

- loose sandy fill to depths of approximately 2.0 m to 3.0 m on the western end of the site and probably to a shallower depth at other locations on the site;
- loose to medium dense clayey sand to depths of approximately 2 m to 5 m overlying bedrock;
- a bedrock profile comprising Hawkesbury Sandstone overlying Newport Formation below approximately RL 5 m. The Hawkesbury Sandstone may be encountered to a depth of approximately 15 m on the western end of the site. The rock generally comprises extremely low to very low strength rock about 1.5 m to 2.0 m thick over medium and high strength rock, however, in some cases the weaker rock is not present.

As indicated on Drawings 3 and 4, it is anticipated that the sandstone surface will step down the slope in a series of benches separated by near-vertical cliff faces typically 2 m to 3 m high and running parallel to the contours of the slope (crossing the site from north to south). The cliff faces are the result of previous (ancient) separation and downslope movement of blocks formed by the prominent north-south and east-west striking joints.

Groundwater is expected to flow along the top of the natural clayey sand and rock surface. It is anticipated that groundwater flows may also occur within fractured zones and joints within the rock, as evident from iron-stained joints in the rock cores. Groundwater seepage flows are likely to increase following periods of extended wet weather.

8. Comments

8.1 Proposed Development

Based on architectural drawings by Richard Cole Architects Pty Ltd (DA00 to DA13 inclusive, dated July 2019) it is understood that the proposed development will involve demolition of the existing buildings and construction of a new five-storey building including a single level of basement carparking. The basement level (RL 6.00 m) will require excavation into the slope to depths of approximately 14 m at the western end of the site with the depth of excavation reducing to approximately 3 m at the eastern end of the site. The excavation will be set back approximately 3.5 m to 4 m from the western boundary, 1.5 to 3 m from the southern boundary and zero to 3 m from the northern boundary. Although the existing retaining walls to the east of the site (in poor condition) fall outside the site boundary it is understood these walls will be demolished and replaced with new retaining structures.

Australian Geoguides for Slope Management and Maintenance (Australian Geomechanics Society (AGS) Landslide Risk Assessment March 2007) provides various guidelines for hillside construction. Geoguide LR8 from AGS (March 2007) is included in Appendix D and provides examples of good and poor hillside construction practice.

8.2 Risk Assessment

Northern Beaches (Pittwater) Council's Geotechnical Risk Management Policy (GRMP - 2009) indicates that the site lies within Hazard Zone 1 which is defined as an area where the likelihood of instability is assessed to be possible to almost certain. The site is located toward the base of an east-facing hill with an average slope of approximately 15 degrees. The geotechnical inspection indicated that there was no evidence of current, significant overall slope instability on the site or adjoining sites. The retaining walls on the eastern end of the site were in poor condition with rotation and cracking of the walls observed, however, it is anticipated that this distress is most likely the result of inadequate design and possibly poor drainage of the ground behind the walls. These walls will be demolished and replaced with properly designed retaining structures as part of the new development.

The site has been assessed in accordance with the methods of AGS (March 2007) and the Pittwater Council GRMP. Identified hazards on the site and adjacent properties are summarised in Table 1, together with qualitative assessment of likelihood, consequence and risk after construction.

Table 1 - Property and Life Risk Assessment for Proposed Development

Hazard	Likelihood	Consequence	Risk
Erosion scour of soil and filling profile	Possible to Unlikely	Property – Minor	Very Low
		Life - Insignificant	1×10^{-6}
Potential failure of new retaining structures	Rare, provided adequately designed and constructed	Property – Major	Low
		Life - Major	1×10^{-6}
Slide or fall of joint blocks or wedges of rock within the proposed excavation	Unlikely, provided regular geotechnical inspection is carried out and stabilisation provided, where required	Property – Minor	Very Low
		Life - Medium	1×10^{-8}

Note *Likelihood assumes work will be carried out in accordance with the recommendations provided in this report.

When compared to the requirements of the GRMP, it is considered that the site is suitable for the proposed development and that the development proposal can achieve the Acceptable Risk Management criteria for both property and life for current or reasonably anticipated site conditions. Further geotechnical investigation, inspection and supervision as described in the following sections will be required to maintain risks within acceptable levels.

8.3 Earthworks

8.3.1 Excavation Conditions

The investigation indicates that the proposed excavation will be through sandy soils, extremely low to very low strength rock then medium and high strength, slightly fractured to unbroken sandstone.

Excavation of fill, residual soils and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of any medium to high strength bands and/or ironstone bands within the weathered rock sequence. The medium and high strength, slightly fractured and unbroken rock may be effectively unrippable and will probably require large hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material. It is suggested that rock saws or rotary milling heads attached to the excavator should be employed along or close to site boundaries and adjacent structures to reduce vibrations and minimise over-break and fracturing of the sandstone.

The excavation rate that can be achieved within the medium and high strength rock varies considerably and is dependent upon the degree of jointing in the rock, the rock strength, the type of machinery being used and the skill of the operator. Some of these factors vary between individual contractors and it is therefore recommended that bulk excavation tenderers be required to make their own assessment of the equipment required to carry out the work.

8.3.2 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with current NSW EPA Guidelines (2014). Under these guidelines, a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes fill and natural materials that will be removed from site. Accordingly, environmental testing will need to be carried out to classify spoil. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site.

8.4 Groundwater Seepage

Groundwater seepage should be expected along the top of the clayey sand and rock surface and through joints and fractures in the rock, particularly following periods of extended wet weather. This seepage should be readily controlled by perimeter drains used to direct seepage around the excavations to the stormwater drainage system.

8.5 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings and pavements before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to vibrations or construction related activities can be accurately assessed. As a minimum, this requirement should include the buildings to the north and south of the site (No. 229 and No. 233 Whale Beach Road) and probably the footpath and Whale Beach Road pavement adjacent to the western boundary.

8.6 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits.

As a guide, Australian Standards AS 2187 (Explosives Code) recommends a maximum vector sum peak particle velocity (VSPPV) of 10 mm/sec for houses and low rise residential buildings. Neighbours may, however, find vibration levels above about 3 mm/s as being strongly perceptible to disturbing. Based on the experience of DP with rock excavations in Sydney it is suggested that a maximum peak particle velocity in any component direction (PPVi) of 8 mm/sec be employed at this site to reduce the risk of structural damage to surrounding buildings. This should be reduced to 3 mm/sec for any buildings founded on loose sandy soils. This vibration limit is applicable at the foundation level of existing buildings and may need to be modified following review of building condition surveys, vibration trials and/or proposed excavation plant.

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. The initial stages of the excavation, during the vibration trial, should be undertaken in the centre of the site to minimise the risk of damage to surrounding structures.

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;
- use of rock sawing or milling heads around the perimeter of the excavation;
- selective breakage along open joints where these are present;
- use of rock hammers in short bursts to prevent generation of resonant frequencies; and
- the movement of large blocks away from existing structures prior to breaking up for transport from site.

8.7 Excavation Support

8.7.1 General

Given the proximity of the excavation to the boundaries, it will be necessary to provide shoring support for the soils and extremely low to low strength rock. It may be possible to have unsupported vertical excavations within sandstone of medium strength or stronger.

Prior to detailed design and planning it is suggested that additional rock cored boreholes should be drilled along the northern and southern boundaries. This information is required to better assess the depth and nature of the soil and weathered rock profile to be retained and also to identify the depth to rock that is suitable for unsupported vertical excavation.

Excavations in sandstone of medium or greater strength will generally be self-supporting (subject to joint orientation) and may be cut vertically. All vertical rock faces must be progressively inspected by a geotechnical engineer at 1.5 m depth intervals to check for adversely inclined joints and detached blocks and to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcreting of fractured or highly weathered zones or rock bolting where adverse joints form potentially unstable wedges of rock.

8.7.2 Retaining Walls/Shoring

Vertical excavations within the soils and extremely low to very low strength rock will require both temporary and permanent lateral support during and after excavation. The houses on the properties to the north and south of the site are set back at least 6 m from the proposed excavation, however, there are sandstone block and concrete block walls and retaining walls closer to the boundaries that must also be considered.

A bored soldier pile wall with shotcrete infill panels may be suitable in some areas with shallow soils, however, it will be necessary to limit the pile spacing and panel heights to reduce wall movements and potential collapse of the sandy soils between piles. Where the sandy profile is deeper it is suggested that a contiguous pile wall should be used, particularily where the excavation is located closest to adjacent structures and walls. Uncased bore piles may be used, however, an allowance for the use of temporary liners to prevent collapse of the sandy soils must be made. Alternatively, Continuous Flight Auger (CFA) grout or concrete injected piles could be used to avoid problems associated with collapsing sandy soils. At this stage, where soldier piles are considered, it is suggested that shoring piles should be spaced at no greater than 1.5 m centres with shotcrete panels constructed in 1.5 m depth intervals within sandy soils increasing to at least 2 m depth intervals within extremely low to low strength rock. Additional boreholes and tests pits should be carried out along the site boundaries to assess the soil profile and allow refinement of the shoring design and sequencing.

Preferably, shoring piles should be founded on rock below the base of the bulk excavation level in order to provide lateral restraint at the base of the excavation and avoid the risk of adversely inclined joints or wedges in the rock undermining the base of the piles. On the western end of the site, where considerable rock excavation is expected, it may be possible to terminate the shoring piles within medium strength or stronger rock above the bulk excavation level. It will be important for a geotechnical engineer to assess the stability of the rock directly beneath each pile and identify whether any stabilisation is required. The toe of the piles above bulk excavation should be restrained with rock bolts or anchors.

Suitably sized drilling rigs fitted with rock augers will be required to penetrate medium and high strength rock and productivity may be low within high strength rock.

8.7.3 Design

The design of the shoring will depend somewhat upon whether it is cantilevered or restrained by multiple rows of temporary rock anchors. It is anticipated that at least one or two rows of rock anchors will be required to provide lateral restraint to shoring piles above the top of medium strength or stronger rock (i.e. self-supporting rock).

It is suggested that design of cantilevered shoring systems (or shoring with a single row of anchors) be based on a triangular earth pressure distribution based on earth pressure coefficients provided in Table 2. Active earth pressures (Ka) may be used where some wall movement is acceptable, and at rest earth pressures (Ko) should be used where wall movement is to be minimised.

Table 2 – Recommended Earth Pressure Coefficients and Bulk Unit Weights

Material	Earth Pressure Coefficient		Bulk Unit Weight (kN/m3)
	Active (Ka)	At Rest (Ko)	
Fill and Clayey Sand	0.4	0.6	20
Extremely Low to Low Strength Rock	0.3	0.45	21
Medium Strength of Stronger Rock	0*	0*	22

Note * Provided that no adverse jointing is present in the rock (to be confirmed by progressive inspection by a geotechnical engineer)

Where more than one row of temporary anchors is used it is suggested that design of shoring is based on a trapezoidal earth pressure distribution. Where there are no movement sensitive structures in close proximity to the excavation the maximum pressure (kPa) could be calculated using 6H (H equals the depth to the top of medium strength or stronger rock). Where the wall movement is to be minimised the maximum pressure could be calculated using 8H. The pressure distribution should increase from zero at the surface to the maximum value at a depth of 0.2 H and then decrease from the maximum at a depth of 0.8H back to zero at the base of the excavation.

All surcharge loads should be allowed for in the shoring design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Passive resistance for piles founded below the base of the excavation may be based on an allowable passive restraint equal to 300 kPa in extremely low to very low strength rock and 1500 kPa in medium strength or stronger rock. Passive resistance should be assumed to start at least 0.5 m below bulk excavation level.

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to the stormwater drainage system.

8.8 Rock Anchors

The design of temporary rock anchors for the support of excavations and shoring systems may be based on an allowable bond stress of 70 kPa in extremely low to very low strength rock and 500 kPa in medium strength or stronger rock. The anchors should be bonded behind a line drawn up at 45° from the base of the shoring, and "lift-off" tests should be carried out to confirm the anchor capacities. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities. It should be noted that permission will be required from adjacent property owners prior to installing bolts/anchors below their land.

It is anticipated that the building will restrain the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would generally need careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

8.9 Excavation Induced Ground Movements

For deep rock excavations, as proposed on the western end of the site, there is a possibility that there will be some horizontal movement due to stress relief effects. Release of these stresses due to the excavation may cause horizontal movements along the rock bedding surfaces and partings. Generally, it is not practicable to provide restraint for the relatively high in-situ horizontal stresses. Based on experience with monitoring of deep rock excavations, lateral stress relief movements on the adjacent ground surface in the order of 1 mm to 2 mm per metre depth of rock excavation could be expected. Empirical data suggest that most of the movement occurs during or shortly after the bulk excavation phase.

8.10 Foundations

Following bulk excavation it is anticipated that medium and high strength sandstone will be exposed over most of the carpark footprint, however, the rock may be up to 5 m deep at the eastern end.

All structural loads should be uniformly supported on the underlying rock (preferably medium strength or greater rock) for which pad footings should generally be appropriate. Piles will be required to reach rock on the eastern end of the site. Uncased bored piles may be suitable, however, the use of temporary liners to prevent collapse of the sandy soils will be required. Alternatively, CFA grout or concrete injected piles could be used to avoid problems associated with collapsing sandy soils.

Footings founded on sandstone of medium strength or stronger may be designed for a maximum allowable bearing pressure of 3500 kPa. This bearing pressure should be reduced to 1500 kPa for footings which are founded close to the edge of vertical excavations in medium strength or stronger rock. Pile sockets, where required, may be designed using an allowable shaft adhesion of 70 kPa in extremely low to very low strength rock and 500 kPa in medium strength or greater rock.

It is likely that the base of the carpark will expose medium and high strength rock for which an increased bearing pressure of up to 6000 kPa may be achievable, however, more detailed inspection and investigation comprising spoon testing will be required if this higher pressure is to be adopted. Spoon testing involves drilling a 50 mm diameter hole below the base of the footing, to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak/clay bands. If weak seams are detected then footings may need to be taken deeper to reach suitable foundation material.

All footings must be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. This is also required for subsequent completion of the Northern Beaches (Pittwater) Council GRMP Form 3 (Final Geotechnical Certificate – Post Construction Geotechnical Certificate).

8.11 Stormwater Control

The existing site stormwater disposal system appears to be by collection from the roof and subsequent down-slope dispersal into the main stormwater drainage system along Surf Road.

The drainage measures for the new development should include appropriately sized, grated surface drains and pits to collect all surface and roof stormwater. The stormwater, together with water collected by subsoil drains behind retaining structures, should be directed down-slope in a controlled manner and it is recommended that it be disposed by direct discharge into the existing stormwater system.

It is assumed that the sewer is and will be connected to the main sewer via a pump to street system.

9. Design Life

DP interprets the reference to design life requirements specified in the GRMP to refer to structural elements designed to retain the site and maintain the risk of instability within acceptable limits.

Specific structures that may affect the maintenance of site stability are considered to include retaining structures, stormwater and subsoil drainage systems. These features should be designed and maintained for the design life of the proposed structures, which in DPs experience, is normally taken to be in the order of 60 years. In order to attain a life of 100 years as required by the GRMP, it will be necessary for the structural engineer to incorporate appropriate design and structural inspection considerations and for the property owner to adopt and implement a maintenance and inspection program, details of which are included in Section 10.4.

10. Construction and Maintenance Requirements

10.1 General

It is considered that the site is suitable for the proposed development and that the development can be carried out within the “Acceptable Risk Management” criteria as defined by the GRMP, subject to the conditions detailed in the following sections and the assumption that the conditions on the subject and adjacent sites do not change in a manner that would adversely affect the proposed development.

10.2 Construction Certificate Requirements

There will be a requirement for DP to examine all structural drawings prepared for the project to verify that the recommendations given in this report have been adopted or taken into account by the structural engineer to enable completion of GRMP Forms 2a and 2b for Construction Consent.

All engineering support structures should have their design life nominated by the structural engineer together with an inspection/maintenance program required to attain the notional design life.

10.3 Construction Inspection Requirements

Inspection of excavations, retaining walls and footings, by a geotechnical consultant, will be required during construction to enable completion of a GRMP Form 3.

Geotechnical inspections should include:

- drilling of shoring piles to confirm the correct depth and foundation strata is achieved;
- all vertical rock excavations at 1.5 m depth intervals to check for adversely inclined joints and to advise on stabilisation requirements;
- all pad footings or piles to check that bedrock of sufficient bearing capacity and stability has been achieved; and
- all subsurface drainage measures and drainage behind retaining walls exceeding 1 m height.

10.4 Maintenance and On-going Inspection Requirements

To attain a life of 100 years, it will be necessary to adopt and implement a detailed inspection regime as outlined in Table 3. It will also be necessary to ensure that subsequent owners and occupants of the property are aware of the ongoing nature and frequency of the inspections, and maintenance requirements.

Table 3 – Recommended Maintenance and Inspection Program

Structure	Maintenance/Inspection Task	Frequency
Drainage lines	Inspect to ensure line is flowing and not blocked.	Every 5 years or following each significant rainfall event.
Drainage pits	Inspect to ensure that pits are free of debris and sediment build-up. Clear surface grates of vegetation/litter build-up.	During normal grounds maintenance, following each significant rainfall event or every 5 years.

Retaining walls	Inspect walls for the presence of cracking or rotation from vertical, or as-constructed condition	Every 5 years or following each significant rainfall event.
General slopes	Inspect slopes and batters for indications of movement which may comprise tension cracks, backscarps of freshly exposed soil.	Every 5 years or following each significant rainfall event.

If the maintenance inspections reveal noticeable changes, prompt reference should be made to an appropriate professional (e.g. structural or geotechnical engineer).

11. Limitations

Douglas Partners (DP) has prepared this report for this project at 231 Whale Beach Road, Whale Beach in accordance with DP's proposal dated 22 August 2018 and acceptance received from Leslie Cassar dated 22 August 2019. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Leslie Cassar for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during a previous 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. The advice may also be limited by budget constraints imposed by others or by site accessibility.

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

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

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires 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.

Copyright

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.

About this Report

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

Information for Contractual Purposes
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

Appendix B

Previous Field Work Results and Site Photographs

Douglas Partners
Geotechnics • Environment • Groundwater

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and Classification Methods

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

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

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

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

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

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

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

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

Sampling

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

Disturbed samples taken during drilling 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 with 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.

Details of the type and method of sampling are given in the report.

Drilling Methods.

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

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

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of 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 sampling.

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

Continuous Spiral Flight Augers — the hole is advanced using 90—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 in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

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

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

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

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test 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 short of full penetration, 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 tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: —

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

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

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

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

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

$$q_c = (12 \text{ to } 18)\, c_u$$

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

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

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. 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 relatively similar tests are used.

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

Laboratory Testing

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

Bore Logs

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

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

Ground Water

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

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

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

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

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

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

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

Site Inspection

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

DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

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

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

SEDIMENTARY ROCK TYPES

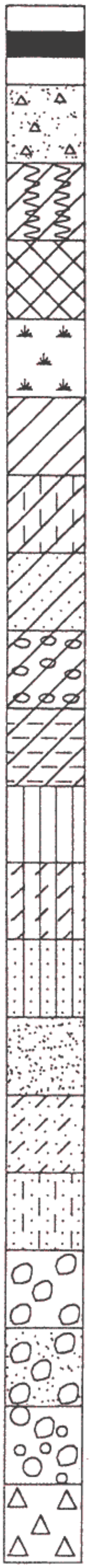
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.

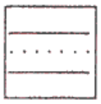
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
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

SEAMS



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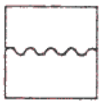
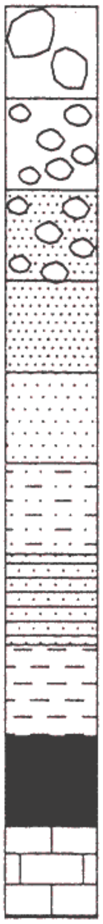


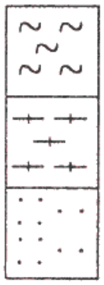
SEAM <10mm

SEDIMENTARY ROCK



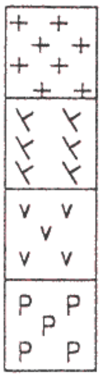
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK



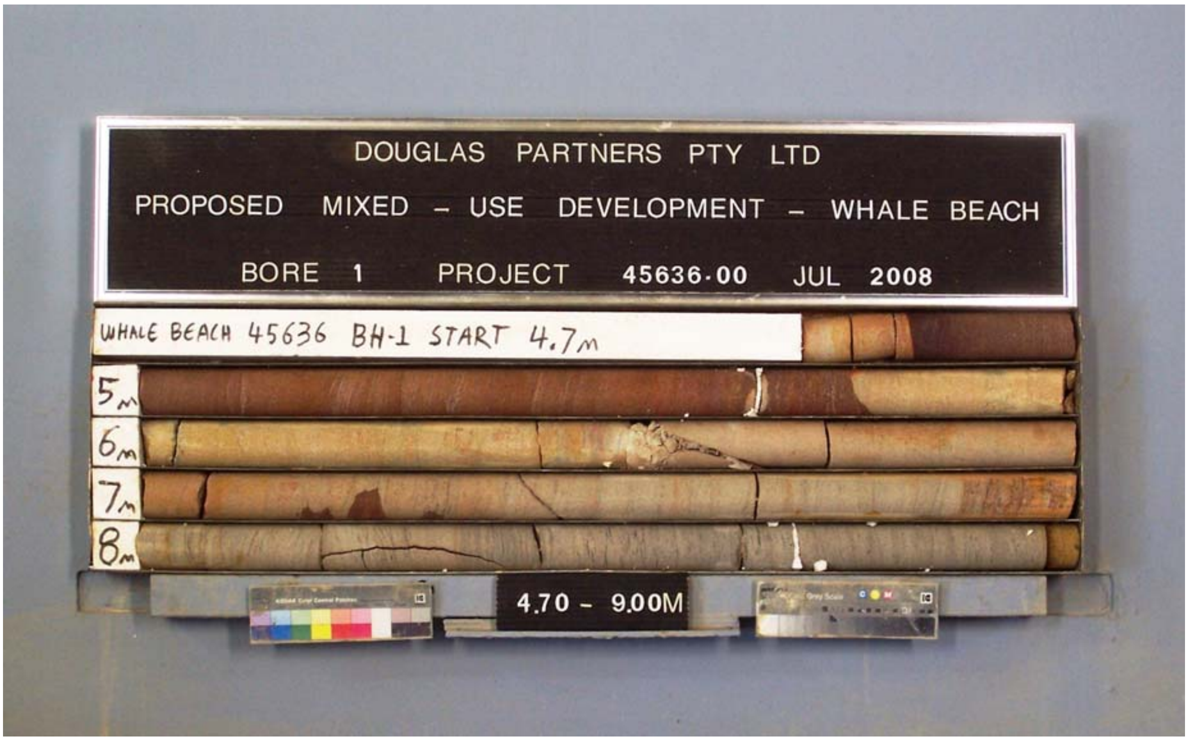
SLATE, PHYLLITE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

Douglas Partners
Geotechnics, Environment, Groundwater


DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 1 PROJECT 45636·00 JUL 2008
WHALE BEACH 45636 BH-1 START 4.7m
5m
6m
7m
8m
4.70 - 9.00M


DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 1 PROJECT 45636·00 JUL 2008
9
10
11
12
13
9.00 -14.00M

BOREHOLE LOG

CLIENT: Leslie Cassar
PROJECT: Proposed Mixed-Use Development
LOCATION: 231 Whale Beach Road, Whale Beach

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

BORE No: 1
PROJECT No: 45636
DATE: 02 Jul 08
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	CONCRETE					
	FILLING - grey brown gravelly sand filling		A			
			A			
1.2	FILLING - light brown, crushed sandstone filling with some clayey sand		S			1,2,3 N = 5
2.3	CLAYEY SAND - medium dense, mottled orange brown, fine grained clayey sand, moist		S			4,6,9 N = 15
3.8	CLAYEY SAND - medium dense, light grey, fine grained clayey sand (extremely weathered sandstone)	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°	S			6,10,11 N = 21
4.7	SANDSTONE - high strength, highly weathered and fresh stained, slightly fractured, light grey and brown, medium to coarse grained sandstone		C	100	100	PL(A) = 1.3MPa
		6.04m: B0°, clayey				PL(A) = 1.2MPa
6.52	SANDSTONE - medium to high then medium strength, fresh stained and fresh, slightly fractured, light grey medium grained sandstone	6.52m: J70°, 10mm clay infill	C	100	99	PL(A) = 1MPa
	- medium strength from 7.4m	7.4m: J45°				PL(A) = 0.6MPa
		8.07m: B5°, clay veneer 8.2m: J75°- 90°, curved				
		8.63m: B0°, clayey				PL(A) = 0.8MPa
	9.35-9.50m: low to medium strength band	9.1m: J85°	C	100	97	PL(A) = 0.9MPa
9.58	SANDSTONE - description next page					PL(A) = 0.3MPa

RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 2.5m

TYPE OF BORING: Diatube to 0.10m; Solid flight auger to 2.5m; Rotary to 4.7m; NMLC-Coring to 14.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ≝ Water level

CHECKED
Initials: STE
Date: 2/9/08

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BOREHOLE LOG

CLIENT: Leslie Cassar
PROJECT: Proposed Mixed-Use Development
LOCATION: 231 Whale Beach Road, Whale Beach

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

BORE No: 1
PROJECT No: 45636
DATE: 02 Jul 08
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01 0.05 0.10 0.50 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
11–7	10.0–14.0	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained sandstone with some carbonaceous laminations *(continued)*						10.45m: B0°, clay veneer	C	100	97	PL(A) = 1.6MPa PL(A) = 1.5MPa
									C	100	100	PL(A) = 1.8MPa PL(A) = 1.7MPa
	14.0	Bore discontinued at 14.0m										

RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Diatube to 0.10m; Solid flight auger to 2.5m; Rotary to 4.7m; NMLC-Coring to 14.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 2/9/08

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DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 2 PROJECT 45636.00 JUL 2008
WHALE BEACH 45636 BH2 START 4.3m
5m
6m
7m
8m
4.30 - 9.00M


DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 2 PROJECT 45636.00 JUL 2008
9
10
11
12
13
9.00 - 13.60M

BOREHOLE LOG

CLIENT: Leslie Cassar		**SURFACE LEVEL:** 20.8 AHD	**BORE No:** 2
PROJECT: Proposed Mixed-Use Development		**EASTING:**	**PROJECT No:** 45636
LOCATION: 231 Whale Beach Road, Whale Beach		**NORTHING:**	**DATE:** 2-3/7/08
		DIP/AZIMUTH: 90°/--	**SHEET** 1 **OF** 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)
0.06	BRICK PAVERS - 50mm thick over sand bedding	
0.15	CONCRETE - 90mm thick	
	FILLING - grey brown, fine to medium grained sand filling with some clay and gravel, moist	
0.9	FILLING - loose, dark grey, fine to medium grained sand filling with some organic debris, moist	
3.0	SANDSTONE - extremely to very low strength, light grey, fine to medium grained sandstone	Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°
4.3	SANDSTONE - high strength, moderately weathered, slightly fractured and unbroken, brown medium to coarse grained sandstone	5.03m: B5°, clayey 5.05m: B5°, 15mm clay 5.66m: B10°, clay veneer
8.55	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained sandstone with some carbonaceous laminations	8.53m: J50°, ironstained 8.66m: B0°, 15mm sand infill 8.76m: B5°, clay smear

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			3,4,3 N = 7
S			1,2,3 N = 5
S			14,25/40mm refusal
C	100	97	PL(A) = 1.1MPa PL(A) = 1.2MPa
C	100	100	PL(A) = 1.7MPa PL(A) = 2MPa PL(A) = 1.3MPa PL(A) = 1.4MPa
C	100	100	PL(A) = 1.5MPa

RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 4.3m

TYPE OF BORING: Diatube to 0.21m; Solid flight auger to 4.3m; NMLC-Coring to 13.6m

WATER OBSERVATIONS: Free groundwater observed at 2.5m whilst augering

REMARKS: Standpipe installed

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 2/9/08

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BOREHOLE LOG

CLIENT: Leslie Cassar
PROJECT: Proposed Mixed-Use Development
LOCATION: 231 Whale Beach Road, Whale Beach

SURFACE LEVEL: 20.8 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 45636
DATE: 2-3/7/08
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
		SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained sandstone with some carbonaceous laminations *(continued)*						10.36m: J10°, carbonaceous band				PL(A) = 1.2MPa
10	11	11.0-11.35m: medium strength, fine grained sandstone							C	100	100	PL(A) = 0.8MPa
9	12											PL(A) = 1.5MPa
8	13								C	100	100	PL(A) = 1.5MPa
7	13.6	Bore discontinued at 13.6m										
6	14											
5	15											
4	16											
3	17											
2	18											
1	19											

RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 4.3m
TYPE OF BORING: Diatube to 0.21m; Solid flight auger to 4.3m; NMLC-Coring to 13.6m
WATER OBSERVATIONS: Free groundwater observed at 2.5m whilst augering
REMARKS: Standpipe installed

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 2/9/08

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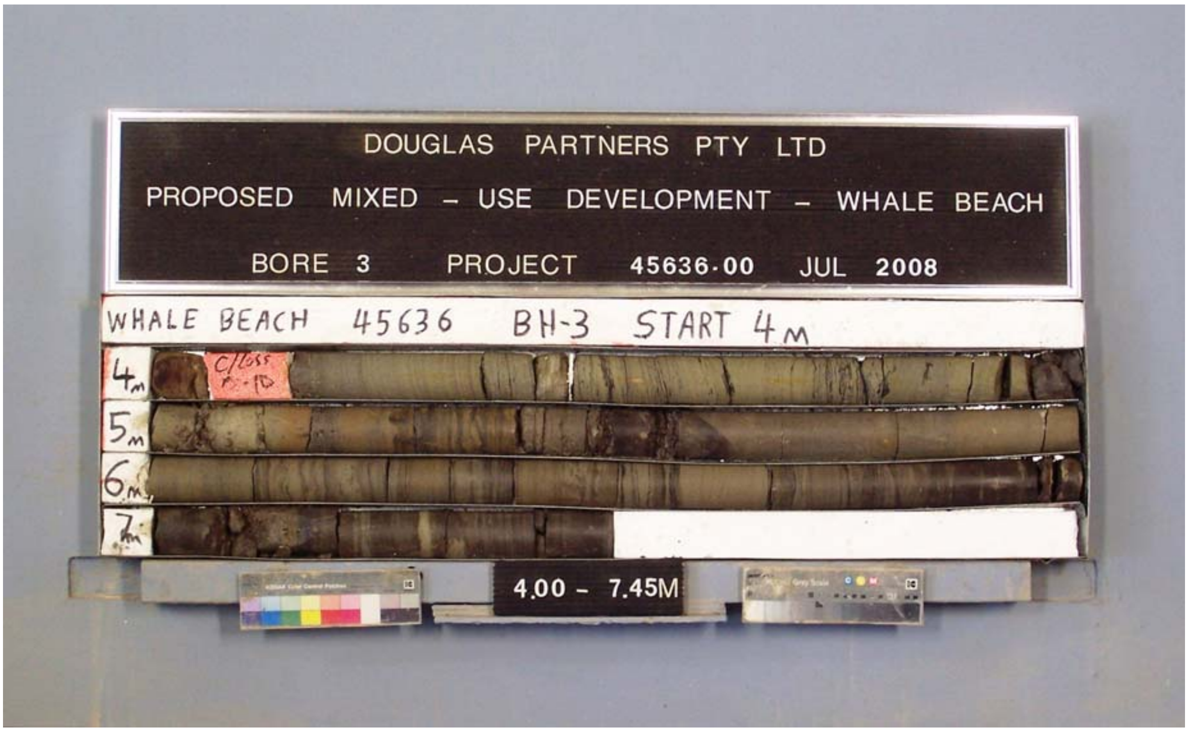
DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 3 PROJECT 45636-00 JUL 2008
WHALE BEACH 45636 BH-3 START 4M
4m
5m
6m
7m
4.00 - 7.45M

BOREHOLE LOG

CLIENT: Leslie Cassar
PROJECT: Proposed Mixed-Use Development
LOCATION: 231 Whale Beach Road, Whale Beach

SURFACE LEVEL: 8.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 45636
DATE: 03 Jul 08
SHEET 1 OF 1

Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
0.13	CONCRETE			A			
	CLAYEY SAND - loose, light grey brown, fine to medium grained clayey sand, moist	▼ (2.0m)		A			
				A			
				S			3,3,5 N = 8
2.2	SANDSTONE - very low strength, light grey brown, fine to medium grained sandstone with ironstone bands		Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°	S			14,30,25 N = 55
4.0 / 4.06	SANDSTONE - medium to high strength, slightly weathered, slightly fractured, light grey, fine to medium grained sandstone		4m: J70, ironstained 4.06m: CORE LOSS: 100mm 4.16m: B0°, clayey	C	78	60	PL(A) = 1MPa
5.05	LAMINITE - medium strength, slightly weathered, fractured, grey fine grained sandstone/siltstone			C	100	91	PL(A) = 0.9MPa
5.7	SANDSTONE - high strength, fresh, unbroken, grey fine to medium grained sandstone with some siltstone laminations and bands		5.54m: J45°, healed				PL(A) = 0.7MPa PL(A) = 1.6MPa
6.8	LAMINITE - medium strength, fresh, slightly fractured, grey laminite		7.1m: J85°				PL(A) = 0.8MPa
7.45	Bore discontinued at 7.45m						

RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 4.0m
TYPE OF BORING: Diatube to 0.13m; Solid flight auger to 4.0m; NMLC-Coring to 7.45m
WATER OBSERVATIONS: Free groundwater observed at 2.0m whilst augering
REMARKS: Standpipe installed

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: STE
Date: 2/9/08

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RESULTS OF DYNAMIC PENETROMETER TESTS

CLIENT LESLIE CASSAR
PROJECT PROPOSED MIXED-USE DEVELOPMENT
LOCATION 231 WHALE BEACH ROAD, WHALE BEACH

DATE 2/7/08
PROJECT NO 45636
PAGE NO 1of 1

TEST LOCATIONS	4	5	6	7						
RL OF TEST	18.6	15.5	17.5	14.5						
DEPTH m	PENETRATION RESISTANCE BLOWS/150mm									
0.00 - 0.15	3	1	1	1						
0.15 - 0.30	1	3	2	3						
0.30 - 0.45	2	2	1	3						
0.45 - 0.60	1	2	1	3						
0.60 - 0.75	1	2	2	5						
0.75 - 0.90	2	7	2	3						
0.90 - 1.05	1	14	1	4						
1.05 - 1.20	2	24	2	4						
1.20 - 1.35	5	29	4	4						
1.35 - 1.50	5	R	10	5						
1.50 - 1.65	8		10	4						
1.65 - 1.80	10		R	15						
1.80 - 1.95	16			30						
1.95 - 2.10	20			R						
2.10 - 2.25	35									
2.25 - 2.40	R									
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										

TEST METHOD AS 1289.6.3.2, CONE PENETROMETER

R = Refusal

TESTED BY: FV
CHECKED BY: STE

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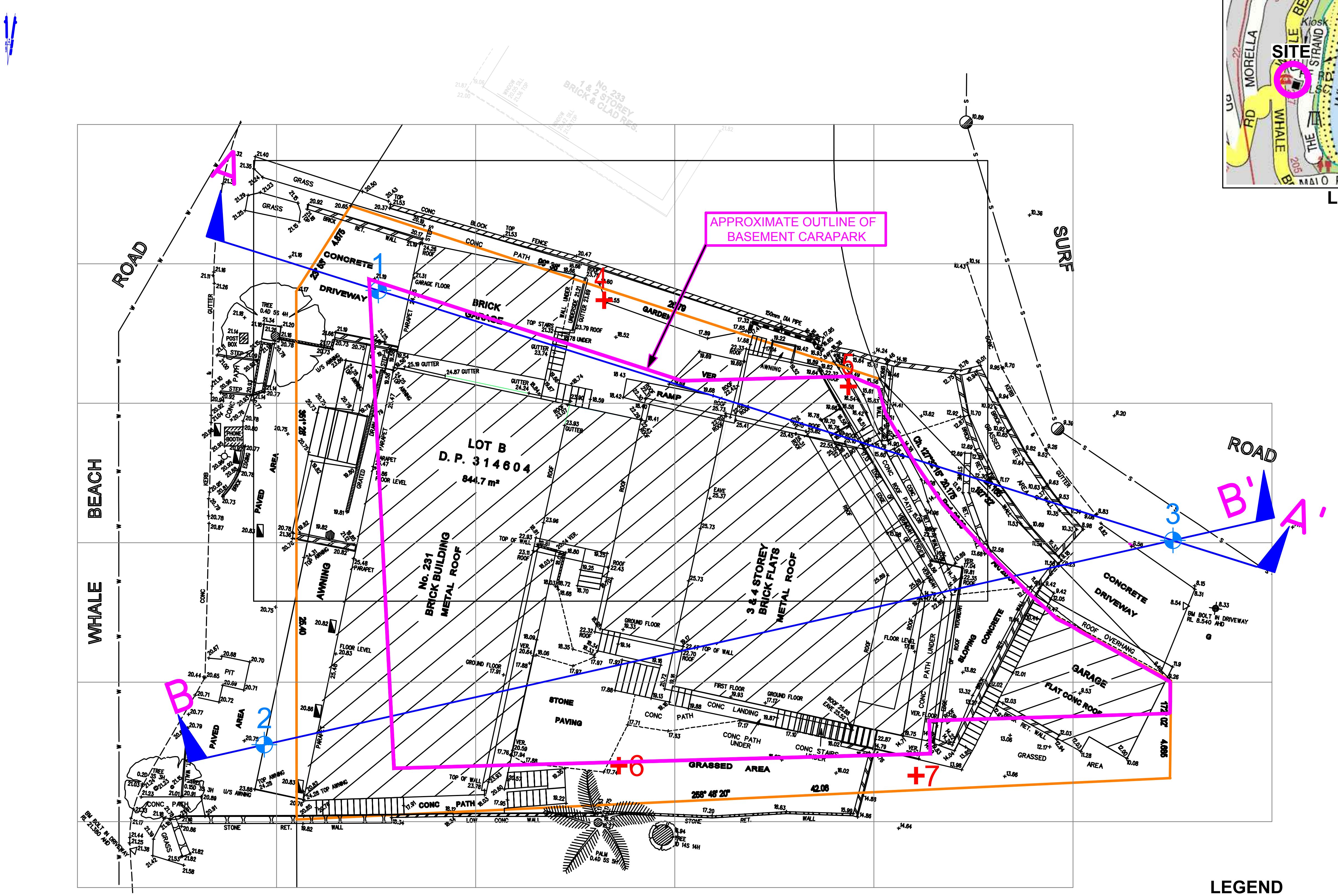
Sydney, Newcastle, Brisbane, Wollongong, Campbelltown

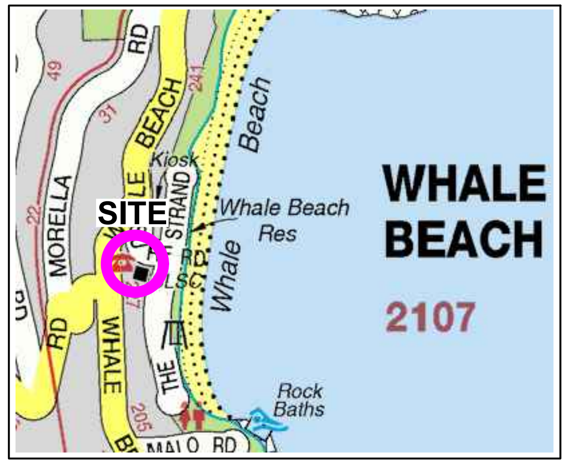
Melbourne, Perth, Wyong, Townsville, Cairns, Darwin

Title	**Locality Map** **Proposed Mix-Use Development** **231 Whale Beach Road, Whale Beach**		
Client: Leslie Cassar			**Office: Sydney**
Drawn by: STE	**Scale: NTS**	**Project Number: 45636**	**Drawing No. 1**
Approved by:		**Date: 1 September 2008**	

Appendix C

Drawings





Locality Plan

NOTE:
1: Base drawing from Rygate and Company Pty Ltd
(Dwg 72649, dated 15.5.2007)
2: Test locations are approximate only and are
shown with reference to existing features.

LEGEND

- Borehole location (2008)
- Dynamic cone penetrometer test location (2008)

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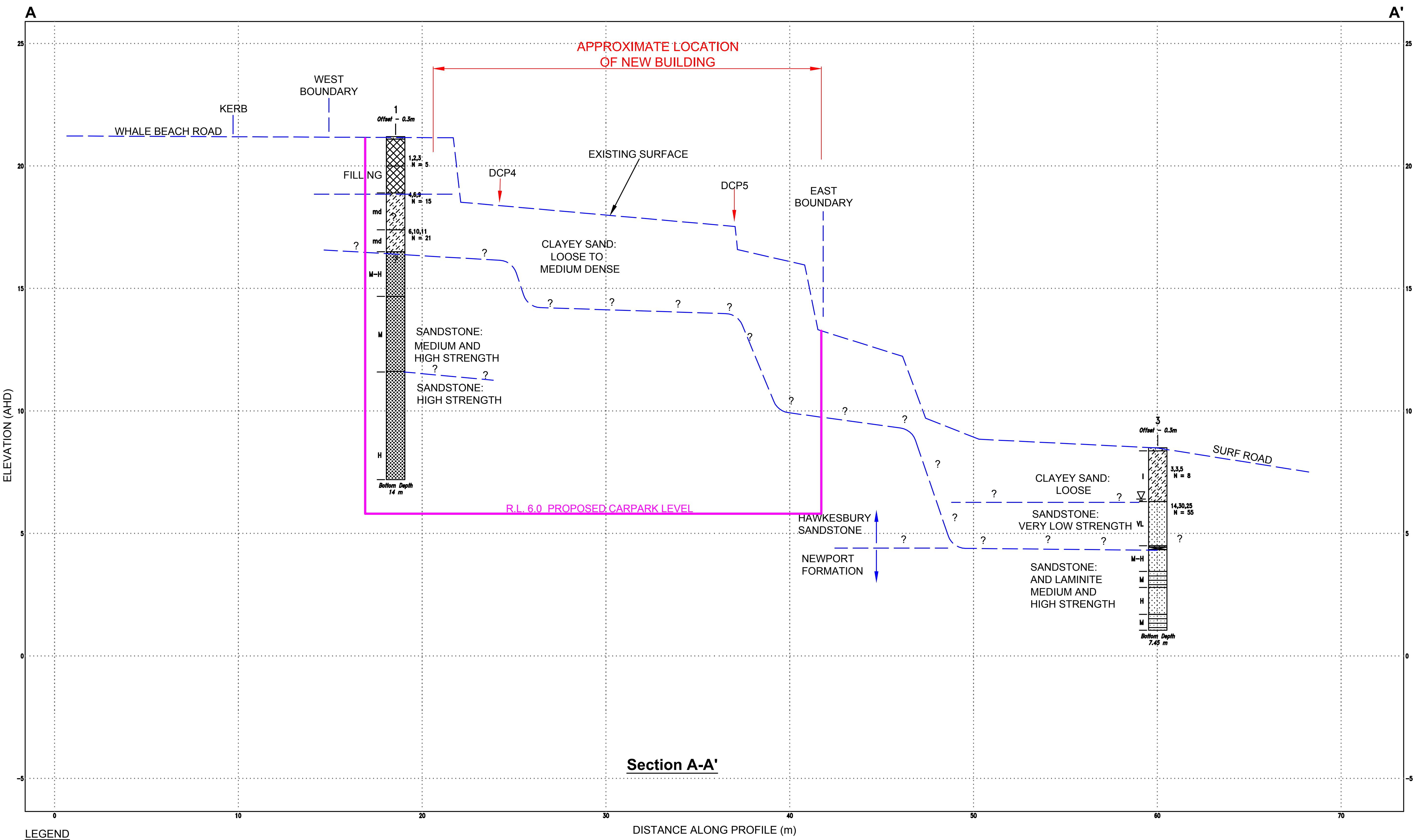
CLIENT: Leslie Cassar		TITLE: **Location of Tests**	PROJECT No: 45636.01
OFFICE: Sydney	DRAWN BY: PSCH	**Proposed Mixed-Use Deveopment**	DRAWING No: 2
SCALE: 1:200 @ A3	DATE: 12.9.2019	**231 Whale Beach Road, WHALE BEACH**	REVISION: 0



Section A-A'

LEGEND

- Concrete
- Fill
- Clayey Sand
- Sandstone coarse grained
- Sandstone fine grained
- Core Loss
- Laminite

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
- Water level

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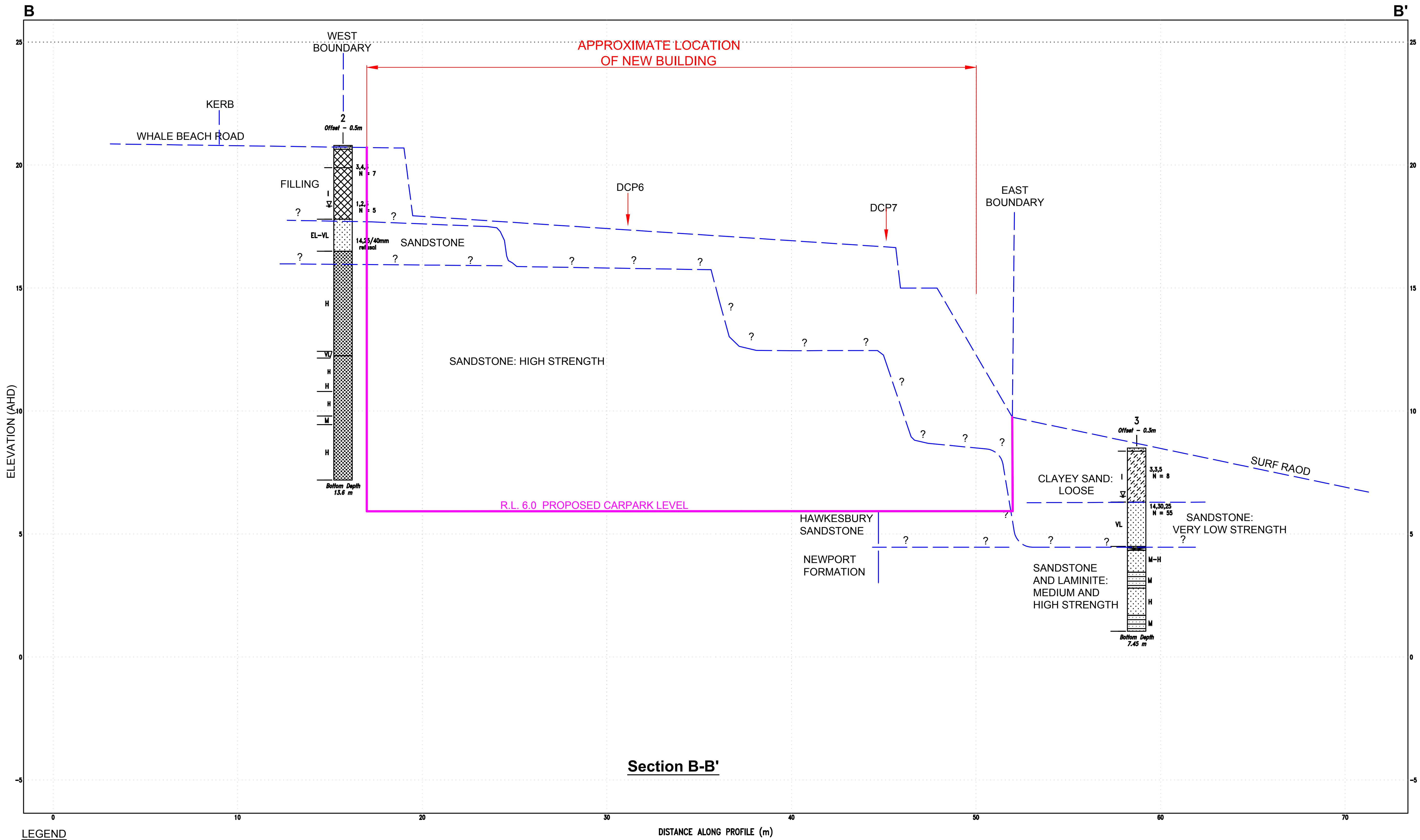
CLIENT: Leslie Cassar		TITLE: **Cross Section A - A'**	PROJECT No: 45636.01
OFFICE: Sydney	DRAWN BY: PSCH	**Proposed Mixed-Use Deveopment**	DRAWING No: 3
SCALE: As shown	DATE: 12.9.2019	**231 Whale Beach Road, WHALE BEACH**	REVISION: 0



Section B-B'

LEGEND

- Concrete
- Fill
- Clayey Sand
- Sandstone coarse grained
- Sandstone fine grained
- Core Loss
- Laminite

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High

SOIL CONSISTENCY
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
Water level

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CLIENT: Leslie Cassar		
OFFICE: Sydney	DRAWN BY: PSCH	
SCALE: As shown	DATE: 12.9.2019	

TITLE: **Cross Section B - B'**
Proposed Mixed-Use Deveopment
231 Whale Beach Road, WHALE BEACH

PROJECT No:	45636.01
DRAWING No:	4
REVISION:	0

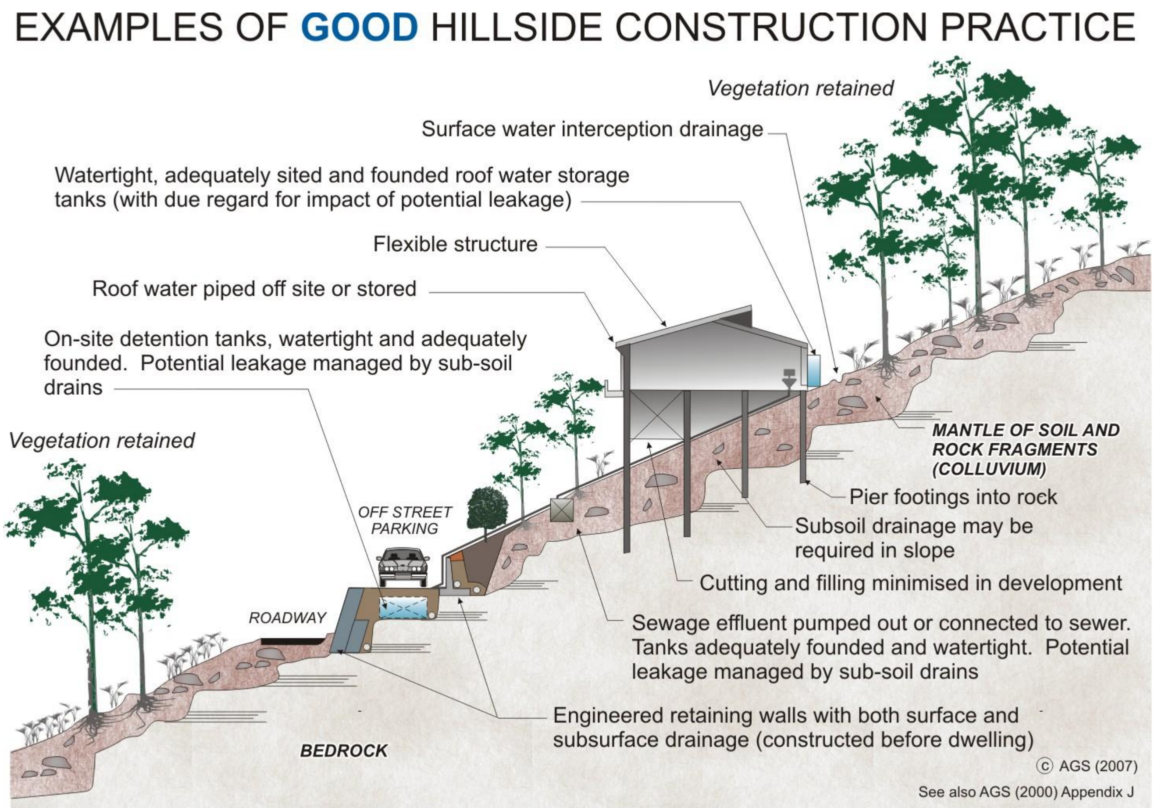
Appendix D

AGS Landslide Risk Assessment Geoguide LR8

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

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



WHY ARE THESE PRACTICES GOOD?

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

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

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

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

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

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

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

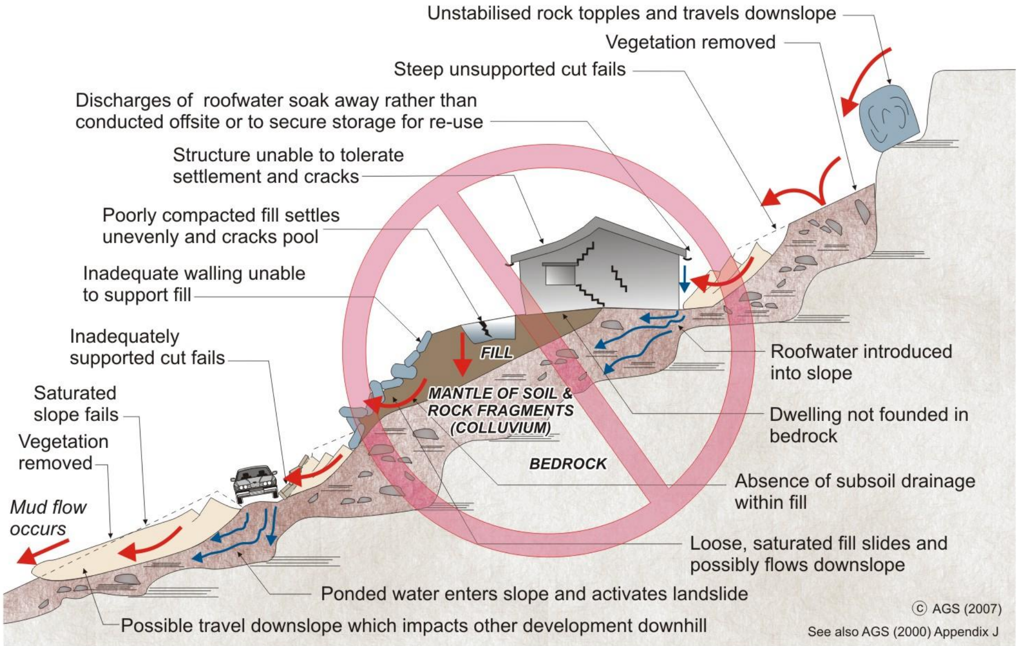
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing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

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

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

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

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

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

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

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

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

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

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

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

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