Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Approved Mixed-Use Development
231 Whale Road, Whale Beach

Prepared for
Leslie Cassar

Project 45636.02
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Integrated Practical Solutions

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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.

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Douglas Partners acknowledges Australia's First Peoples as the Traditional Owners of the Land and Sea on which we operate. We pay our respects to Elders past and present and to all Aboriginal and Torres Strait Islander peoples across the many communities in which we live, visit and work. We recognise and respect their ongoing cultural and spiritual connection to Country.

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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

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1. Introduction

This report presents the results of an additional geotechnical investigation undertaken for an approved mixed-use development at 231 Whale Road, Whale Beach. The investigation was commissioned in an email dated 21 July 2023 by Richard Cole Architecture on behalf of Leslie Cassar, and was undertaken in accordance with Douglas Partners' proposal 45636.02.P.002 dated 26 June 2023. The additional investigation comprised the drilling of two (2) deep boreholes located approximately midway along the northern and southern boundaries to provide supplementary geotechnical information to our previous geotechnical investigation.

Douglas Partners Pty Ltd (DP) has previously carried out a geotechnical investigation for Leslie Cassar in September 2008 (DP Report No. 45636.00, dated 4 September 2008) for a previously proposed development, which did not proceed. Then in 2019, DP provided an updated report (DP Report No. 45636.01, dated 27 September 2019) using the same field work results gathered in 2008 to address some geotechnical issues for the proposed development. Since 2019, the proposed development has been revised and now comprises a deeper basement than originally proposed. The original proposed development comprised a single level basement to RL 10.3 m AHD (Australian Height Datum) and the revised development comprises a two-level basement which will be constructed at about RL 6.5 m AHD, which is about 4 m lower than originally proposed. It is understood that the development application (DA) to the Northern Beaches Council (NBC) has been approved (DA No: DA2020/0442).

The current investigation comprised the drilling of two deep boreholes, laboratory testing, groundwater level measurement and permeability testing. This report includes the field work results from the September 2008 geotechnical investigation and the recent additional borehole and groundwater data. Details of the field work and comments relevant to design and construction are given in this report. A slope risk assessment conducted in 2019 and again in 2023 was undertaken with reference to the Australian Geomechanics Society (AGS) Landslide Taskforce “Practice Note Guidelines for Landslide Risk Management”, 2007 (Ref. 1) is also provided.

Information provided for use in this investigation included an architectural drawing package prepared by Richard Cole Architecture (RCA) for Project No. 1609 (Drawings CC01 to CC28, CC30 to CC36, CC40 to CC48, CC50 to CC CC57, CC60 to CC66, CC68 to CC74, CC 79 to CC84, CC90 to CC94, CC100 to CC102, all dated July 2023).

This report supersedes the previous geotechnical reports for the proposed development.

2. Approved Development

Based on the drawings provided, it is understood that the DA approved development will comprise the demolition of existing structures and construction of a mixed-use development (retail and residential) comprising six levels, with the lower basement level constructed at about RL 6.53 m. The excavation will be between about 3 m and 12 m deep (refer Figure 1) and within about 3 m to 4 m of the western (Whale Beach Road) boundary, about 2 m of the southern boundary and about 0 m to 9 m of the northern boundary. Figure 2 shows the basement level footprint.

Figure 1: Cross section view looking south (taken from RCA drawing No. CC16, dated July 2023)

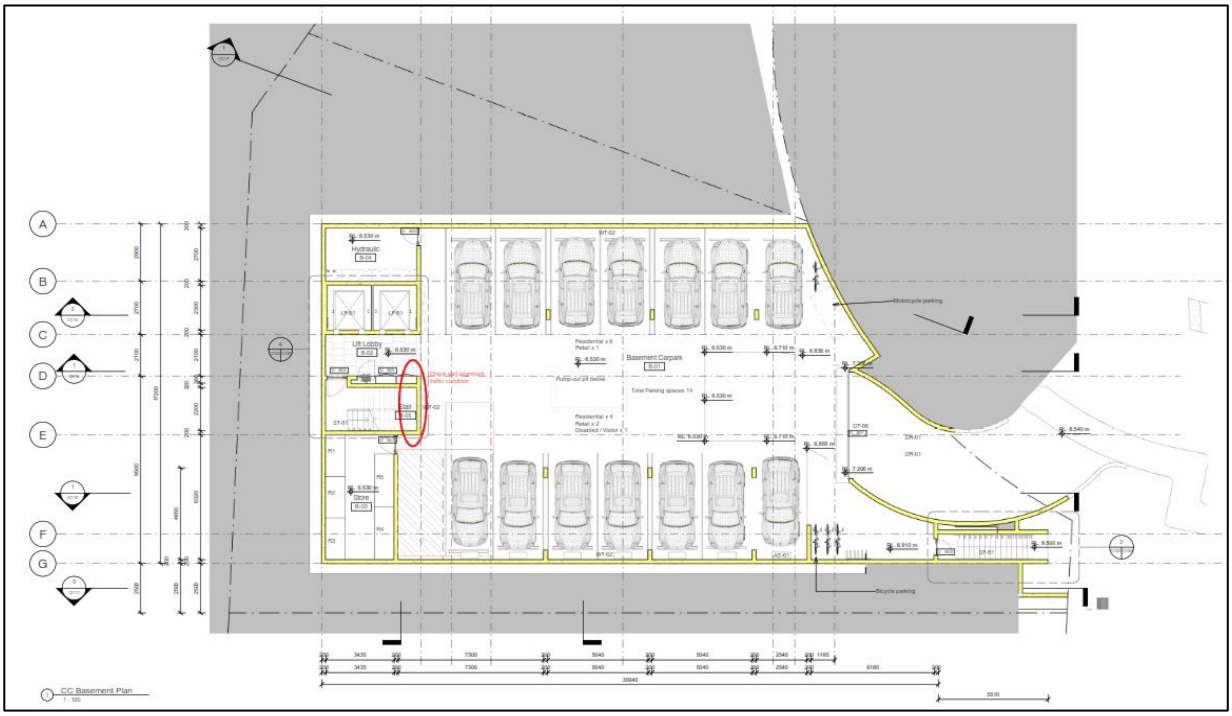


Figure 2: Plan view of the basement footprint (taken from RCA drawing No. CC05, dated July 2023)

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 E.

3. Site Description

A site locality plan (Drawing 1) is included in Appendix B. The site is located toward the base of an east-facing hill which falls toward Whale Beach which is located 50 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 AHD, with an average slope of approximately 15°. The ground slope reduces to approximately 5° to the east of the site, between Surf Road and Whale Beach.

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 existing slope below the building on the eastern side of the site (i.e., above Surf Road) is about 7 m high and at the time of the investigation had been temporarily supported with two rows of one-tonne ballast bags.

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).

4. Published Data

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.

5. Previous DP Investigation

The results from the 2008 investigation are summarised below with the borehole logs and core photographs given within Appendix C. The previous borehole locations (BH1, BH2 and BH3) are shown on Drawing 1 in Appendix B and they generally encountered a subsurface profile comprising fill to depths of up to 3 m overlying clayey sand (colluvium) underlain by sandstone bedrock at depths of between about 2.0 m to 4.7 m depth. 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 of 3.0 m in BH 2 and 1.2 m in BH 3 on 1/9/2008 and measured again during the current investigation (refer Section 6.3).

6. Field Work

6.1 Field Work Methods

The field work for current investigation was supervised by a geotechnical engineer between 14 and 18 September 2023 and comprised:

- Walkover inspection of the site by an experienced geotechnical engineer;
- Review of Before You Dig Australia (BYDA) drawings and a Sydney Water sewer plan followed by scanning for buried services at the proposed borehole locations;
- Drilling of three (3) boreholes (BH101 to BH103). BH101 was drilled with a hand-auger to 1.7 m depth where it was aborted due to refusal on concrete. BH102 was drilled through the surface concrete pavement with a diatube coring barrel. BH102 and BH103 were then drilled through the soil profile with a hand-auger to depths of 1.8 m and 3.3 m and extended into rock using (proline) NMLC-sized diamond core drilling equipment to depths of 16.00 m and 15.96 m, respectively, to obtain 50 mm diameter continuous samples of the rock for identification and strength testing purposes;
- Installation of groundwater monitoring wells into BH102 and BH103; and
- Measurement of groundwater levels in all groundwater monitoring wells (BH2, BH3, BH102 and BH103); and
- Permeability testing in the four wells.

The locations of the boreholes are shown in Drawing 1 in Appendix B. The borehole location coordinates and elevation were recorded with a high-precision GPS relative to GDA2020 MGA56.

6.2 Field Work Results

The subsurface conditions encountered in boreholes BH101, BH102 and BH103 are presented in the borehole logs in Appendix D, together with notes defining descriptive terms and classification methods.

BH101 encountered a concrete obstruction at 1.7 m depth and was subsequently aborted and redrilled nearby as BH103.

The subsurface conditions encountered in BH102 and BH103 can be summarised as:

- FILL – encountered in BH102 to 0.4 m depth and BH103 to 2.5 m depth. The fill was generally sandy with some clay and gravel inclusions; overlying
- COLLUVIAL and RESIDUAL SOILS – encountered within BH102 and BH103. The colluvial soils comprised fine to medium grained sand about 0.5 m thick over residual soil comprising apparently stiff, medium plasticity clay to a depth of 3.6 m in BH102 and 3.3 m in BH103; overlying
- BEDROCK – comprising sandstone overlying laminite.

  BH102 initially encountered highly to moderately weathered, fractured and very low to medium strength sandstone that transitioned to fresh, medium to high strength sandstone below 6.3 m depth. High strength, fresh laminite was encountered at 14.6 m depth (about RL3.9 m) and extended to the base of BH102 at 16.0 m depth.

  BH103 encountered moderately weathered and fresh, medium strength sandstone that transitioned to high strength, fresh sandstone below 11.5 m depth. High strength, fresh laminite was encountered at 13.9 m depth (about RL 4.0 m) and extended to the base of BH102 at 16.0 m depth.

6.3 Groundwater Measurements

Groundwater seepage was observed at 0.8 m depth in BH102 during auger drilling. It is anticipated that the observed groundwater seepage is associated with perched groundwater along the sand and clay interface. Groundwater was not observed during auger drilling of BH103 and the essential use of water as a drilling fluid, during the coring of the boreholes, precluded any further groundwater observations.

The groundwater monitoring wells installed within BH102 and BH103 were purged immediately after installation, and the groundwater level measured at completion of the investigation. A summary of the measured groundwater levels from this investigation and the previous investigation are provided in Table 1.

Table 1: Summary of Groundwater Measurements

Borehole	Screen Depth (m)	Surface RL (m AHD)	Date	Measured Groundwater Depth (m)	Approximate Groundwater RL (m AHD)
BH2	Unknown	20.7	1 September 2008	3.0	17.3
			18 August 2023	3.0	17.3
			14 September 2023	3.0	17.3
			18 September 2023	3.0	17.3
			17 October 2023	3.9	18.2
BH3	Unknown	8.7	1 September 2008	1.2	7.5
			18 August 2023	1.2	7.5
			14 September 2023	1.4	7.3
			18 September 2023	1.4	7.3
			17 October 2023	1.4	7.3
BH102	5.0-16.0	18.5	18 September 2023	5.4	13.1
			17 October 2023	4.9	12.6
BH103	6.0-16.0	17.9	18 September 2023	5.1	12.8
			17 October 2023	4.1	11.8

6.4 Field Permeability Testing

Rising head permeability tests were undertaken in all monitoring wells during this investigation to evaluate the rock mass hydraulic conductivity (or permeability). The test involves removing water from the well and measuring the rise in water level within the well at regular time intervals. The results of the permeability tests are summarised in Table 2. The detailed results of the permeability tests are included in Appendix D. Well installation details for BH102 and BH103 are included on the borehole logs in Appendix D.

Table 2: Rising Head Test Results

Borehole	Screened Material	Screen Depth (m)	Hydraulic Conductivity, k (m/s)
BH2	Sandstone	Unknown	3.4×10^{-7}
BH3	Sandstone	Unknown	2.7×10^{-7}
BH102	Sandstone	5.0 - 16.0	1.1×10^{-7}
BH103	Sandstone	6.0 - 16.0	8.7×10^{-7}

The effective screen lengths used to calculate the hydraulic conductivity in BH2 and BH3 has been assumed to be the height of the water column from the bottom of the hole.

6.5 Site Observations

The site was inspected by a geotechnical engineer from DP on 14 September 2023. The main site observations are:

- The western portion of the existing building is mostly in a good to fair condition. Towards the eastern side of the building, significant cracking was observed in the external balcony and also along the external concrete footpath and stairs which is situated close to the crest of a steep 7-9 m high batter which appears to have been temporarily supported with gravel bags. The slope on the eastern side of the site (above Surf Road) and small retaining wall at its base indicate significant slope instability (refer Photos 1 to 4 in Appendix E).
- Cracking in the concrete footpath possibly caused by temporary propping of the balcony above, or slope instability, or both (Refer Photo 3 in Appendix E); and
- Medium to high strength sandstone outcrop observed to the north of the subject site, along the boundary between Surf Road and 233 Whale Beach Road.

7. Laboratory Testing

Selected samples of the rock core were tested in our DP laboratory for axial point load strength index ($I_{s(50)}$) values and the results of the testing are shown on the borehole logs at the corresponding depth.

$I_{s(50)}$ values for the rock cores ranged from 0.09 MPa to 2.7 MPa, corresponding to a very low to high strength classification. These $I_{s(50)}$ results suggest an unconfined compressive strength (UCS) in excess of 40 MPa for the high strength rock encountered during the investigation.

8. Geotechnical Model

Three geological cross sections comprising Section A-A', Section B-B' and Section C-C', showing the interpreted subsurface profile between the borehole locations are shown on Drawing 2, Drawing 3, and Drawing 4, respectively, in Appendix B. The orientations of the cross-sections are shown on Drawing 1. The sections show interpreted geotechnical divisions of underlying soil and rock together with the extent of the approved 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, sand (likely colluvium) and stiff clay (likely residual) to depths of approximately 2 m to 5 m overlying bedrock; and

- A bedrock profile comprising Hawkesbury Sandstone overlying possible 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. A high strength laminite unit exists below about RL4.0 m.

As indicated on Drawings 2 and 3, it is anticipated that the sandstone bedrock surface will step down the slope in a series of benches separated by sub-vertical cliff faces typically 2 m to 3 m high and orientated 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.

9. 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°. The geotechnical inspection indicated strong evidence of current and significant slope instability on the eastern side of the site and advice regarding this hazard has been provided to RCA. The building has significant structural cracks on its balconies and the slope appears to have been temporarily supported with timber props and ballast bags.

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 3, together with qualitative assessment of likelihood, consequence, and risk after construction.

Table 3 - Property and Life Risk Assessment for the Approved Development

Hazard	Likelihood	Consequence	Risk
Slope failure above Surf Road and below the building	Likely (slope failure is ongoing and it will likely fail further following a heavy rainfall event)	Property – Catastrophic	Very High
		Life - Catastrophic	2.2×10^{-2} (not tolerable)
Erosion scour of soil and fill 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.

Excluding the existing slope failure above Surf Road, 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 when compared to the requirements of the GRMP.

It is understood that the residents will soon vacate the building. Once the building has been vacated, and with ongoing survey monitoring and geotechnical inspection, the slope failure hazard above Surf Road will reduce in 'risk' from 2.2×10^{-2} (not tolerable) to 6.6×10^{-5} (tolerable).

Further geotechnical monitoring, inspection and supervision as described in the following sections will be required to maintain risks within acceptable levels.

10. Comments

10.1 Excavation

The plans indicate that up to about 14 m of excavation may be required for the basement floor levels but reducing to about 5 m depth towards Surf Road. It is expected that the excavation will encounter sandy fill, sandy and clayey soils underlain by extremely low to very low strength rock (weathered rock) then medium and high strength sandstone.

The fill and soil should be readily removed using conventional hydraulically operated earthmoving equipment with bucket attachments. Sandstone bedrock excavation will require rock saws, rotary mill heads or hydraulic rock breaking equipment. Rock saws should be used in medium strength or stronger rock along site boundaries close to adjacent structures.

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.

10.2 Excavation Support

10.2.1 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 provided there are no adverse joints/defects in the rock.

10.2.2 Batter Slopes and Vertical Rock Excavations

Batter slopes could be adopted for excavations up to a maximum height of 3 m for soils and extremely low to low strength rock where they are sufficiently distant from site boundaries, existing structures and in-ground services. Recommended temporary and permanent batter slopes are given in Table 4.

Table 4: Recommended Batter Slopes for Exposed Material

Exposed Material	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
Soil (Fill and Clay)	1.5 : 1	2.5 : 1**
Extremely low to low strength rock	0.75 : 1*	1 : 1*

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

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

If surcharge loads are applied near the crest of the slope, then further specific geotechnical review and probably flatter batters or stabilisation using rock bolts or soil nails may be required.

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/anchors where adverse joints form potentially unstable wedges of rock. Some allowance for stabilisation works should be made. Staged rock excavation should be considered along the northern boundary close to adjacent structures.

10.2.3 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. Shoring support will therefore be required from the ground surface down to at least the top of medium strength rock. The houses on the lots 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 clayey soils, however, it will be necessary to limit the pile spacing and panel heights to reduce wall movements and potential collapse of any sandy soils between piles. Where the sandy profile is deeper, such as BH103 where sandy fill and sand extended to a depth of 3 m, it is suggested that a contiguous pile wall be used, particularly where the excavation is located closest to adjacent structures and walls. Uncased bore piles could be considered, 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.

Test pits or additional investigation with boreholes could be used to delineate the extent of deeper sandy soils on the site to refine the shoring design and pile spacing.

Preferably, shoring piles should be founded on rock below the base of the bulk excavation level 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 deep rock excavation is required, 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.

10.2.4 Earth Pressure Design

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of medium strength rock. Table 5 outlines material and strength parameters that may be used for the preliminary design of excavation support structures.

Table 5 – Material and Strength Parameters for Excavation Support Structures

Material	Bulk Density (kN/m^3)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	Ultimate Passive Earth Pressure (kPa)
Fill	18	0.4	0.6	-
Soil (residual and colluvial)	20	0.3	0.45	-
Weathered Rock	22	0.2[1]	0.3[1]	750[2]
Medium Strength Rock	23	0[1]	0[1]	3,000[2]

Notes: [1] Unless unfavourably jointed
[2] Only below bulk/detailed excavation level and where jointing is favourable

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 should be assumed to start at least 0.5 m below bulk excavation level and a reduction factor must be applied to the ultimate values given in Table 5.

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.

10.3 Ground Anchors

Where necessary, the use of declined tie-back (ground) anchors is suggested for the lateral restraint of the perimeter piled walls. Such ground anchors should be declined below the horizontal to allow anchorage into the stronger bedrock materials at depth. The design of temporary ground anchors for the support of piled wall systems may be carried out using the allowable average bond stresses at the grout-rock interface given in Table 6.

Table 6: Recommended Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Very Low and Low Strength Sandstone	100	200
Medium and High Strength Sandstone	500	1000

The parameters given in Table 6 assume that the drilled holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring or top of free standing medium strength rock, and "lift-off" tests should be carried out to confirm the anchor capacities. Trial anchors should be used to confirm bond stress values. It is suggested that ground anchors should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load.

In normal circumstances 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 would require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

It will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services, pipes and subsurface structures during anchor installation.

10.4 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with current NSW Environment Protection Authority (EPA) regulations. Under the NSW EPA Waste Classification Guidelines (2014) a waste / fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from this site. Accordingly, environmental testing will need to be carried out to classify spoil prior to disposal. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site.

It should be noted that some receiving sites, such as those operated by Councils or other bodies might have their own special environmental criteria to be met before admitting any materials. The scope of this investigation did not include sampling and testing for Waste Classification or Contamination Assessment purposes.

10.5 Excavation Vibration

The proposed excavations will include removal of medium and high strength sandstone. Excavation of this material will be undertaken using equipment which is likely to generate vibrations which could potentially disturb the neighbours or damage the nearby structures.

During excavation it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of structure (eg. reinforced concrete or brick structures etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the structure and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s vector sum peak particle velocity (VSPPV). 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 VSPPV for human comfort.

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

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation under the guidance of an experienced vibration specialist. The trial may indicate that smaller hammers or different types of excavation equipment should be used for excavation purposes.

To reduce the effects of vibration from hydraulic rock hammers, the work method should allow for:

- Rock sawing around the perimeter of the excavations;
- Use of rock hammers in short bursts to prevent generation of resonant frequencies; and
- Changing equipment or size of hammers if the vibration trial indicates that the vibrations are potentially damaging or disturbing.

DP suggests that permanent vibration monitors be set up on site to monitor all the vibrations during excavation and to allow for a change of excavation techniques to be undertaken, if required.

10.6 Excavation Induced Rock Movement (Stress Relief)

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.

10.7 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 to document any existing defects so that claims for damage due to construction related activities can be accurately assessed.

10.8 Foundations

The proposed bulk excavation works are expected to expose medium and high strength sandstone over most of the footprint, however very low to low strength rock about 1-2 m thick maybe present on the eastern side in some areas (see Drawing 3 for example).

Design of footings subject to axial compression loading may be based on the parameters provided in Table 7. For bored piles, if required, shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the shaft adhesion values for compression in Table 7.

Table 7: Design Parameters for Footings and Bored (or CFA) Piles

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure		Elastic Modulus (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression)[3] (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression)[1] (kPa)	
Very Low to Low strength rock	1,500	100	3,000	150	150
Medium to High Strength Sandstone	3,500	350	20,000	700	1,200

Notes: 1 Shaft adhesion applies to pile foundations for which the socket sidewalls are adequately cleaned and roughened to "R2" standard (or better) as defined in Pells et. al. (1998)

For traditional 'working-stress' design methods, foundations proportioned based on the allowable bearing pressures in Table 7 would be expected to experience total settlements of less than 1% of the footing width under the applied Working load, with differential settlements between adjacent columns expected to be less than half of this value.

For limit state design methods, selection of the geotechnical strength reduction factor (ϕ_g) in accordance with Australian piling code (AS 2159, 2009) is based on a series of individual risk ratings (IRR), which are weighted on numerous factors and lead to an average risk rating (ARR). Therefore, it is recommended that an appropriate geotechnical strength reduction factor be calculated by the pile designer. Footing settlements may be calculated for assessment of the serviceability limiting state using the elastic modulus values given in Table 7.

All footings should 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 (formerly Pittwater) Council GRMP Form 3 (Final Geotechnical Certificate – Post Construction Geotechnical Certificate). Spoon testing will be required in at least 50% of footing excavations for pad footings that are designed for an allowable end bearing pressure greater than 3,500 kPa. Allowable bearing pressures of 6000 kPa may be possible for pad footings in medium to high strength sandstone however this will be subject to additional proof coring and spoon testing.

Footings that are within the zone of influence of adjacent excavations or pits may need to be designed for a reduced bearing pressure. The zone of influence can be considered as an influence line rising at 45 degrees from the base of the adjacent excavation or pit. Specific geotechnical advice should be sought in relation to the design of such footings.

10.9 Seismic Design

In accordance with AS1170–2007 “Structural Design Actions, Part 4: Earthquake Actions in Australia”, a hazard factor (Z) of 0.08 and a site subsoil Class C_e (shallow soil site) is considered to be appropriate for the site.

10.10 Hydrogeological Conditions and Site Drainage

It is anticipated that during and following periods of wet weather there will be ongoing subsurface seepage both from along the top of rock and also from along any jointing, bedding planes or other structures in the rock which are intersected by the excavation. Surface run-off should be controlled by perimeter drains to direct seepage around the excavations and building structures to the stormwater system.

Appropriate allowance of subsoil drainage should be incorporated into the design and construction to reduce the adverse effects of moisture and to ensure the amenity of all below-ground areas and excavation. From a geotechnical point of view, seepage inflow is expected to be minor and therefore a drained basement is technically feasible (this is subject to approval by Council and relevant authorities). Further analysis can be carried out to predict inflow rates to inform drainage design, if required. The actual inflow will only be known at the time of bulk excavation when inflows can be observed while also making some allowance for increases due to rainfall and other factors.

The design and construction of drainage measures should also allow for future inspection, maintenance and cleaning of drainage lines, particularly of red-brown iron hydroxide sludge.

Due to the sloping topography of the area and relatively shallow depth to bedrock, it is expected that the permanent, regional groundwater table will be below the proposed building basement and that the proposed residential development of the site will have no significant influence on the existing surface and groundwater flow system, both on the site and in the surrounding area.

11. 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 12.4.

12. Construction and Maintenance Requirements

12.1 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.

12.2 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.

12.3 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.

12.4 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 8. 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 8 – 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).

13. Limitations

Douglas Partners (DP) has prepared this report for this project at 231 Whale Road, Whale Beach in accordance with DP's proposal dated 26 June 2023 and acceptance received from Richard Cole Architecture dated 21 July 2023. 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 be 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 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. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

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.

Douglas Partners Pty Ltd

Appendix A

About This Report

Introduction

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

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

Copyright

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

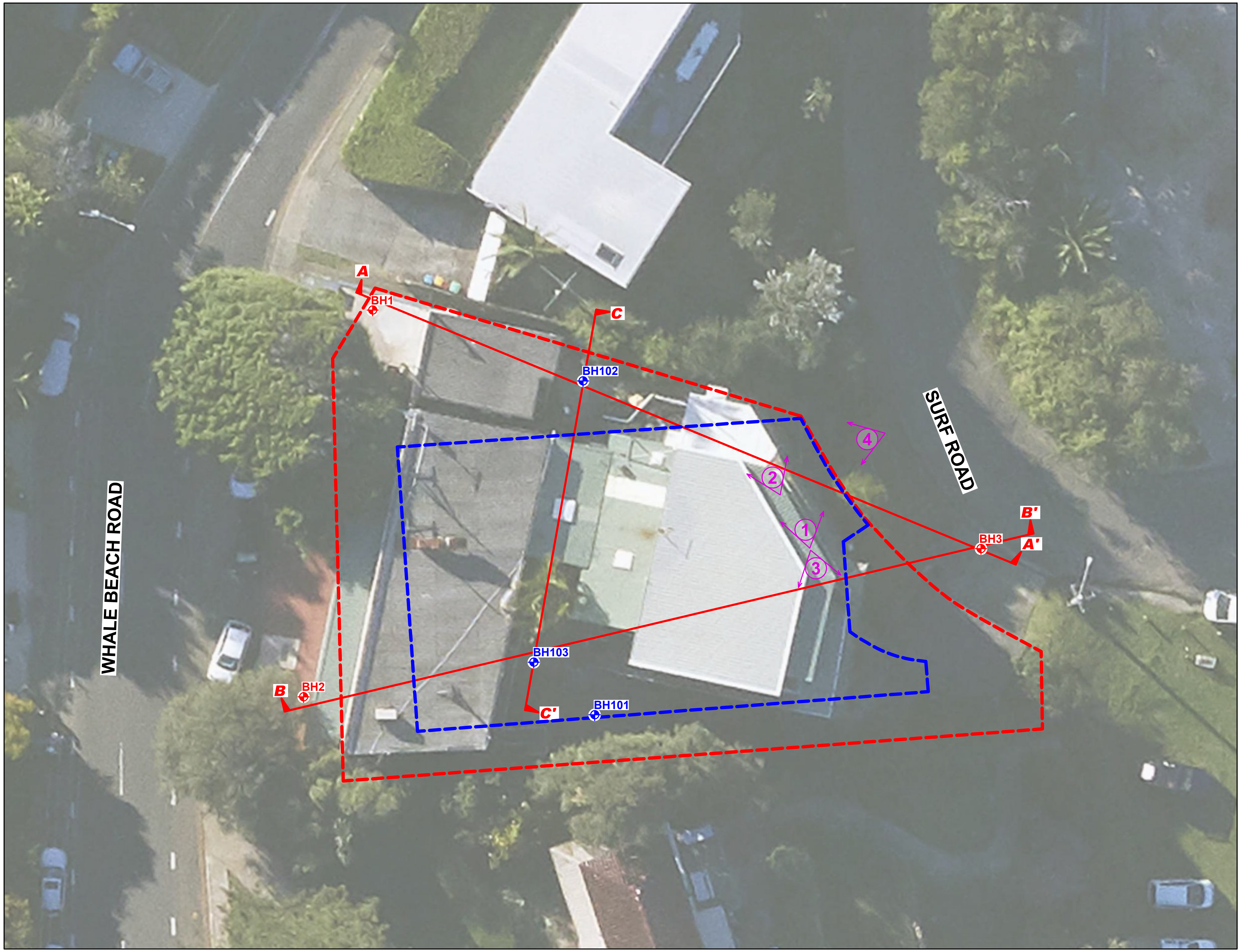
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

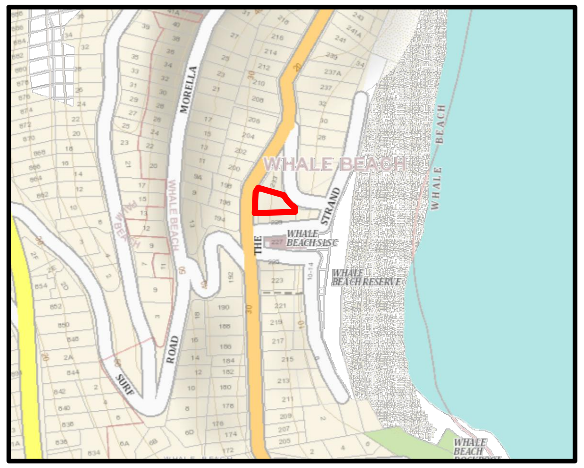
Site Inspection

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

Drawings 1 - 4





Locality Plan

LEGEND

- Approximate Site Boundary
- Approximate Basement Outline
- ⊕ Borehole Location (2008)
- ⊕ Borehole Location (2023)
- ① Photo number with direction of view
- A A' Interpreted Geotechnical Cross Section

NOTE:
1: Base image from MetroMap (Dated 06.05.2023)

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

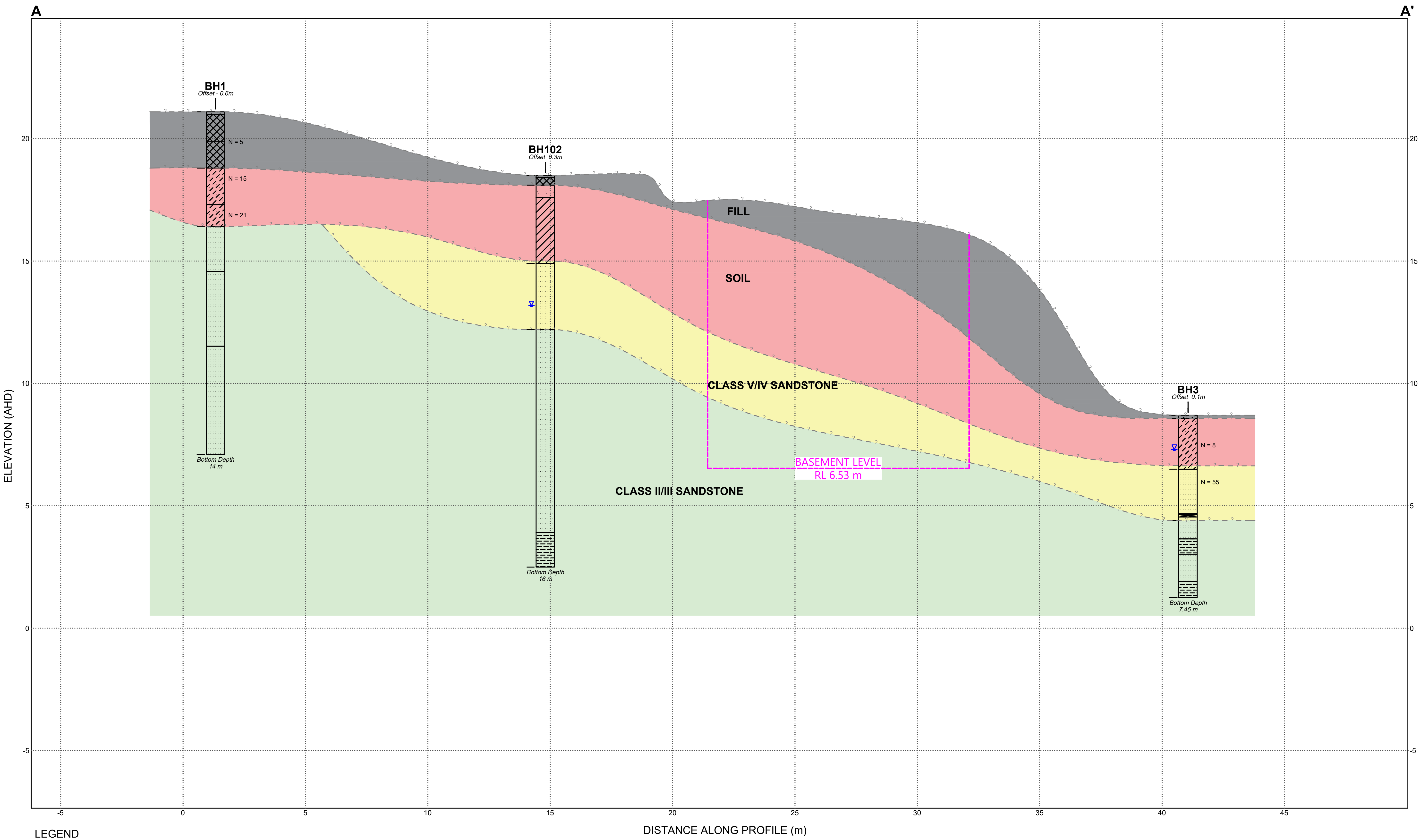
Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Richard Cole Architects
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:250 @ A3
DATE: 20.09.2023

TITLE: **Test Location Plan**
Proposed Mixed Use Development
231 Whale Beach Road, Whale Beach

N

PROJECT No: 45636.02
DRAWING No: 1
REVISION: 0



LEGEND

☒ Core Loss
▨ Clay
▨ Clayey Sand
△ Concrete
▦ Filling
⬚ Sand
⬚ Sandstone
☰ Shale

NOTES:
1. 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.
2. Summary logs only and should be read in conjunction with detailed logs.

TESTS / OTHER
N - Standard penetration test value
-·-·- - Interpreted geotechnical boundary
▽ - Water level

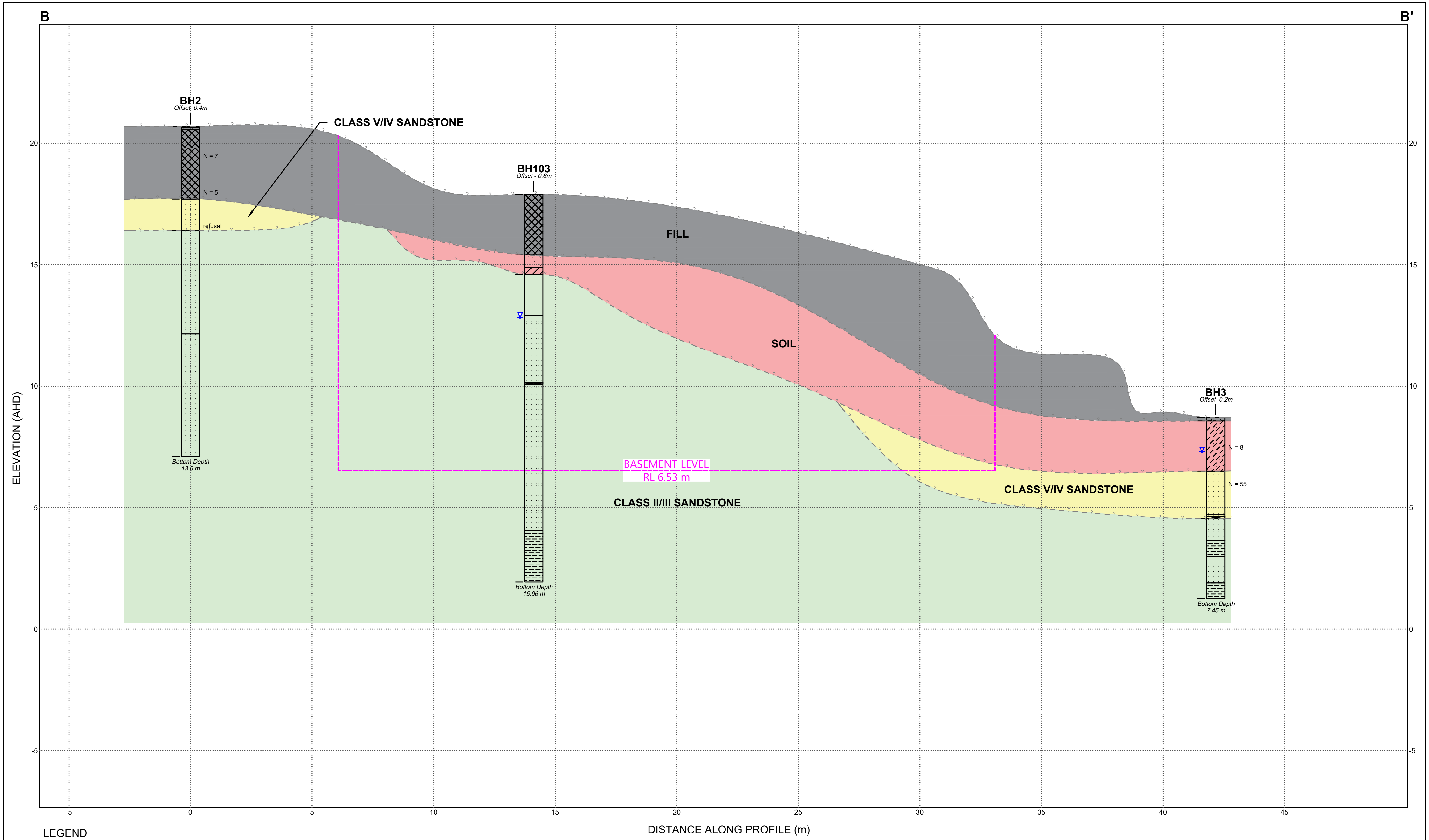
0 3
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Richard Cole Architects
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:150 (H) 1:150 (V) @ A3
DATE: 20.09.2023

TITLE: **Interpreted Geotechnical Cross-Section A-A'**
Proposed Mixed-Use Development
231 Whale Beach Road, Whale Beach

PROJECT No: 45636.02
DRAWING No: 2
REVISION: 0



LEGEND

- ☒ Core Loss
- ▨ Clay
- ▨ Clayey Sand
- ▨ Concrete
- ▦ Filling
- ▤ Pavers
- ░ Sand
- ░ Sandstone
- ▤ Shale

NOTES:
1. 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.
2. Summary logs only and should be read in conjunction with detailed logs.

TESTS / OTHER
N - Standard penetration test value
-·-·- - Interpreted geotechnical boundary
▽ - Water level

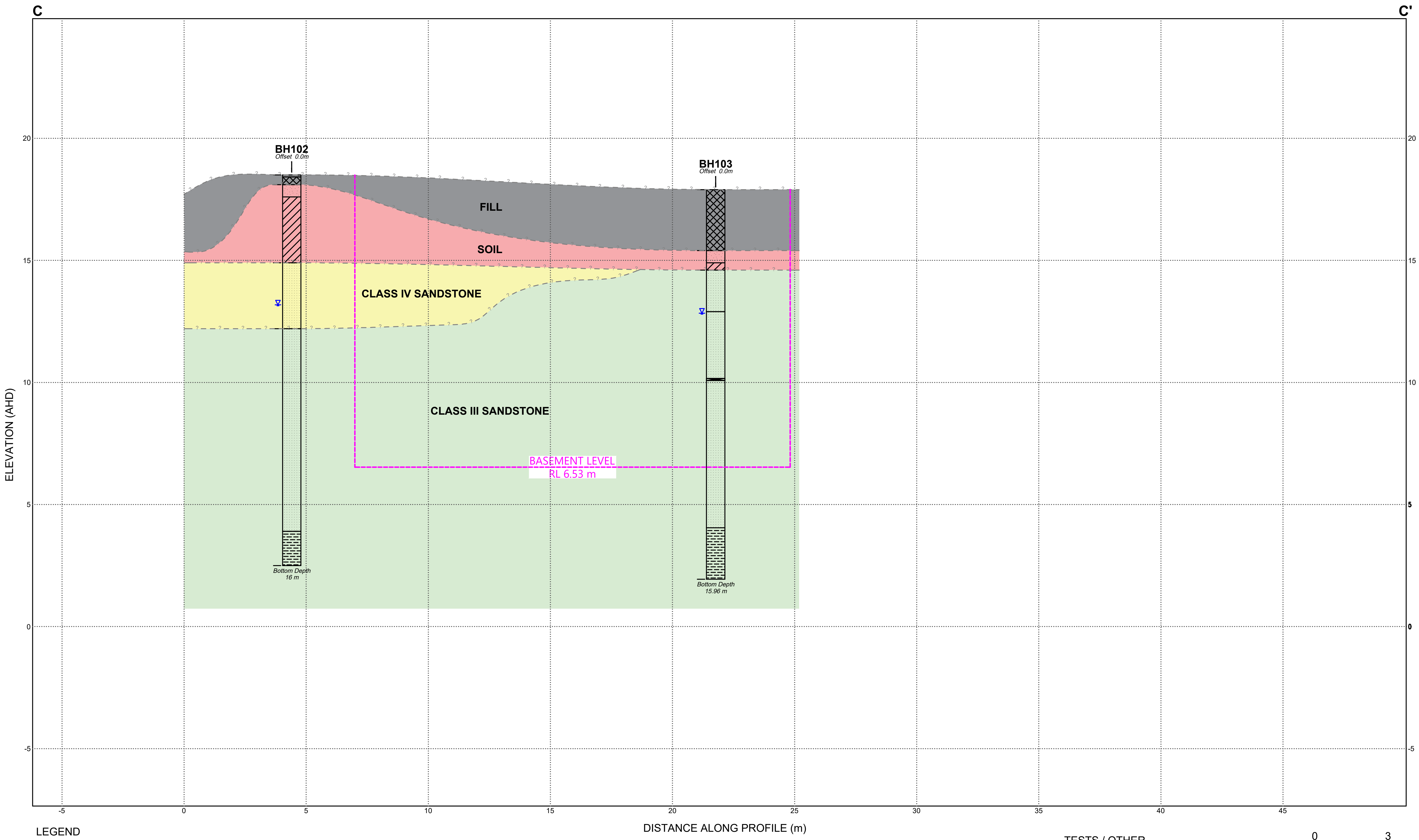
0 3
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Richard Cole Architects
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:150 (H) 1:150 (V) @ A3
DATE: 20.09.2023

TITLE: **Interpreted Geotechnical Cross-Section B-B'**
Proposed Mixed-Use Development
231 Whale Beach Road, Whale Beach

PROJECT No: 45636.02
DRAWING No: 3
REVISION: 0



LEGEND

☒ Core Loss

▨ Clay

▵ Concrete

▩ Filling

☐ Sand

☐ Sandstone

☰ Shale

NOTES:

1. 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.
2. Summary logs only and should be read in conjunction with detailed logs.

TESTS / OTHER

N - Standard penetration test value

-·-·- - Interpreted geotechnical boundary

⊻ - Water level

0 3

Horizontal Scale (metres)

Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Richard Cole Architects
OFFICE: Sydney
DRAWN BY: EC
SCALE: 1:150 (H) 1:150 (V) @ A3
DATE: 20.09.2023

TITLE: **Interpreted Geotechnical Cross-Section C-C'**
Proposed Mixed-Use Development
231 Whale Beach Road, Whale Beach

PROJECT No: 45636.02
DRAWING No: 4
REVISION: 0

Appendix C

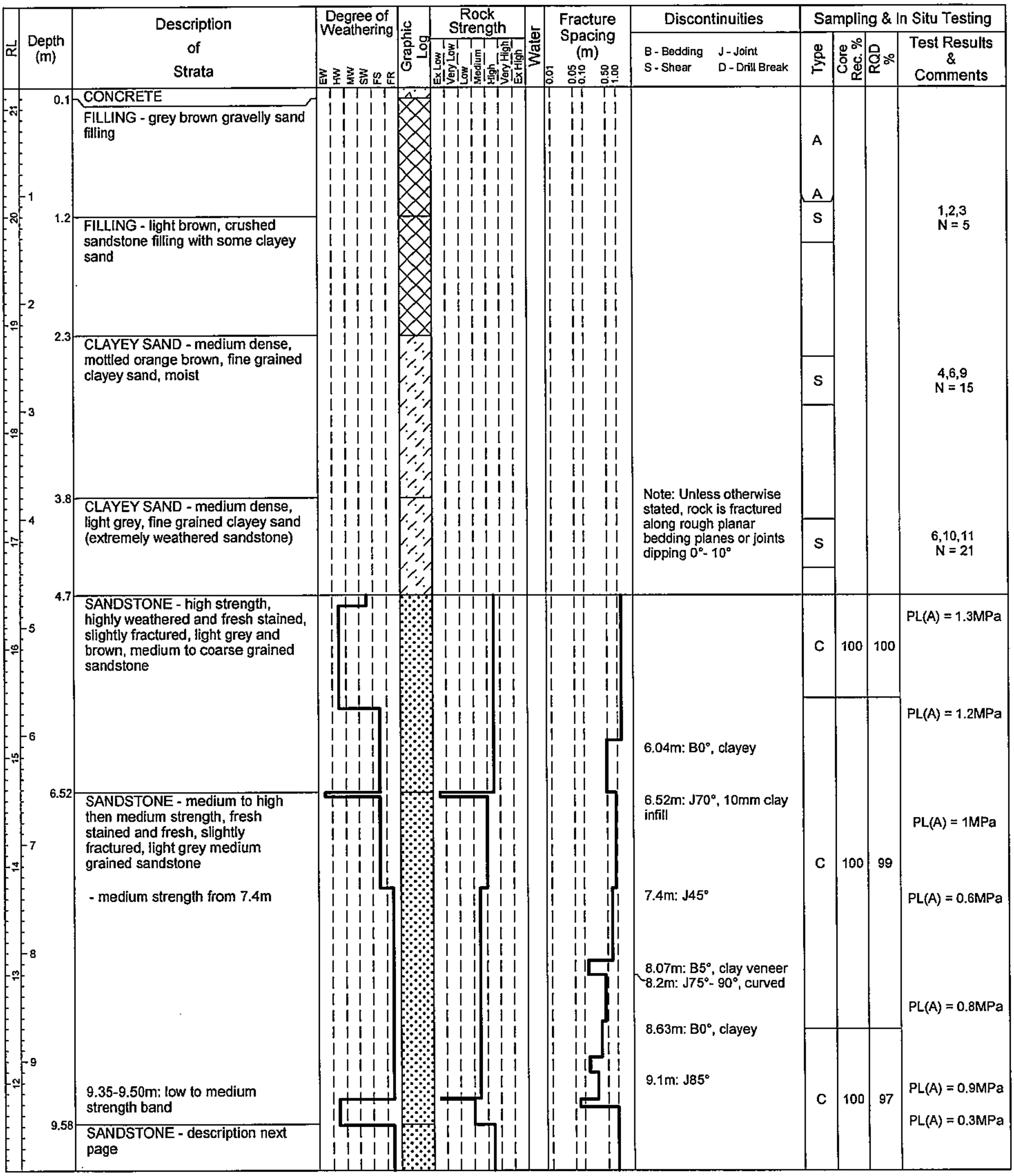
Previous Field Work Results

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 B - Bedding J - Joint S - Shear D - Drill Break	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	C	100	99	PL(A) = 1.2MPa
6.52	SANDSTONE - medium to high then medium strength, fresh stained and fresh, slightly fractured, light grey medium grained sandstone - medium strength from 7.4m	6.52m: J70°, 10mm clay infill 7.4m: J45° 8.07m: B5°, clay veneer 8.2m: J75°- 90°, curved 8.63m: B0°, clayey				PL(A) = 1MPa PL(A) = 0.6MPa PL(A) = 0.8MPa
	9.35-9.50m: low to medium strength band	9.1m: J85°	C	100	97	PL(A) = 0.9MPa PL(A) = 0.3MPa
9.58	SANDSTONE - description next page					

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

Douglas Partners
Geotechnics · Environment · Groundwater

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		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
10	11											PL(A) = 1.5MPa
9	12											
8	13								C	100	100	PL(A) = 1.8MPa PL(A) = 1.7MPa
7	14 14.0	Bore discontinued at 14.0m										
6	15											
5	16											
4	17											
3	18											
2	19											

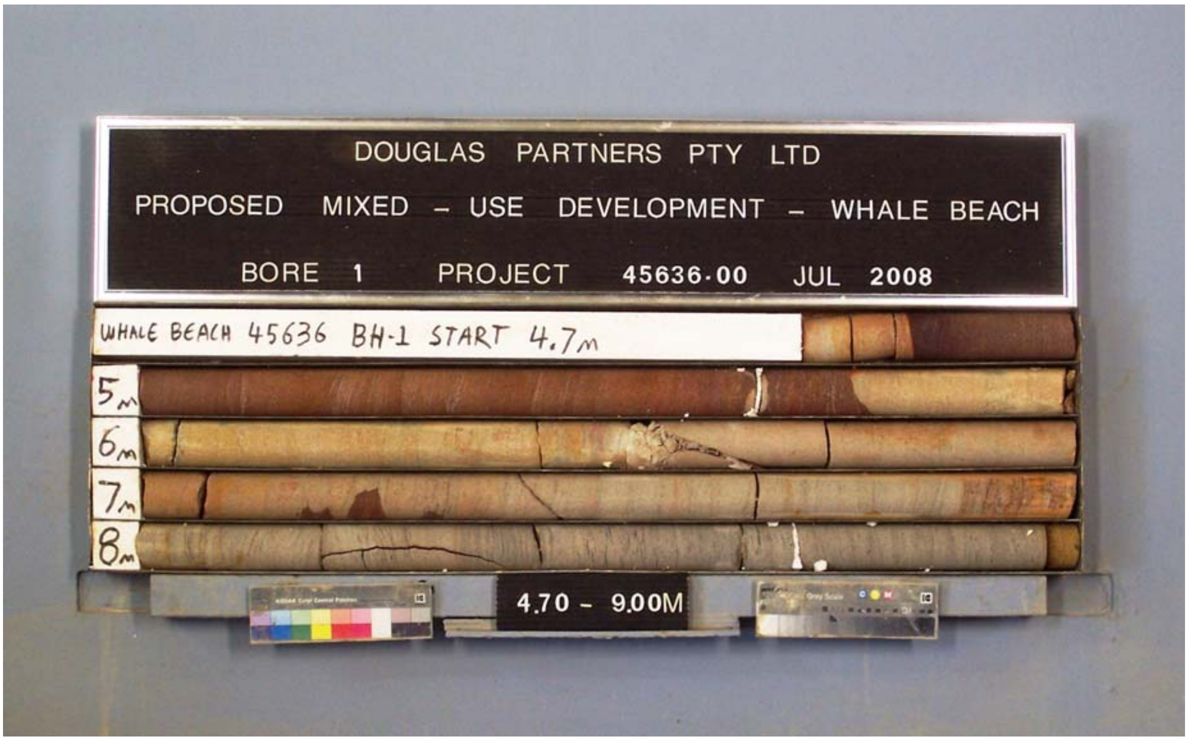
RIG: Bobcat **DRILLER:** Steve **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Diatube to 0.10m; Solid flight auger to 2.5m; Rotary to 4.7m; NMLC-Coring to 14.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

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_x	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

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

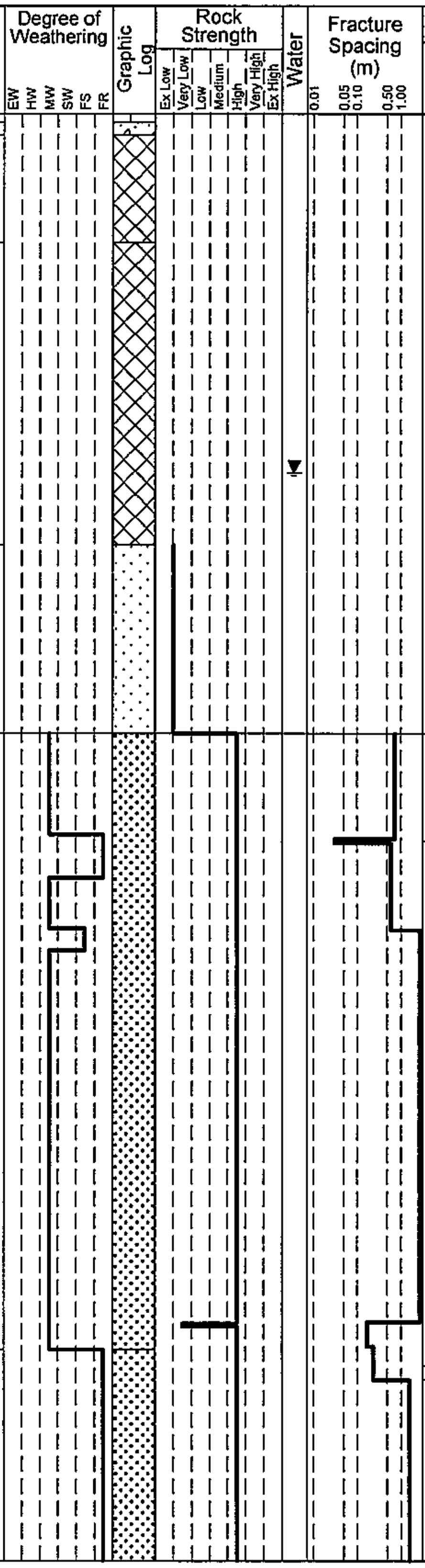

DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED – USE DEVELOPMENT – WHALE BEACH
BORE 1 PROJECT 45636·00 JUL 2008
9
10
11
12
13
9.00 –14.00M

BOREHOLE LOG

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

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

BORE No: 2
PROJECT No: 45636
DATE: 2-3/7/08
SHEET 1 **OF** 2



Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
0.06	BRICK PAVERS - 50mm thick over sand bedding						
0.15	CONCRETE - 90mm thick			A			
	FILLING - grey brown, fine to medium grained sand filling with some clay and gravel, moist			A			
0.9	FILLING - loose, dark grey, fine to medium grained sand filling with some organic debris, moist			A			
				S			3,4,3 N = 7
		▼					
				S			1,2,3 N = 5
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°				
				S			14,25/40mm refusal
4.3	SANDSTONE - high strength, moderately weathered, slightly fractured and unbroken, brown medium to coarse grained sandstone						PL(A) = 1.1MPa
			5.03m: B5°, clayey 5.05m: B5°, 15mm clay	C	100	97	
			5.66m: B10°, clay veneer				PL(A) = 1.2MPa
							PL(A) = 1.7MPa
							PL(A) = 2MPa
				C	100	100	
							PL(A) = 1.3MPa
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				PL(A) = 1.4MPa
							PL(A) = 1.5MPa
				C	100	100	

RL: 20, 19, 18, 17, 16, 15, 14, 13, 12, 11

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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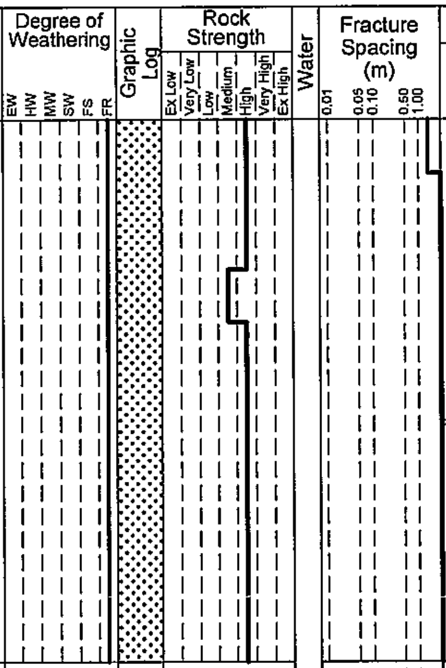
BOREHOLE LOG

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	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
10	11	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey, medium grained sandstone with some carbonaceous laminations *(continued)* 11.0-11.35m: medium strength, fine grained sandstone	10.36m: J10°, carbonaceous band	C	100	100	PL(A) = 1.2MPa PL(A) = 0.8MPa
9	12		>>	C	100	100	PL(A) = 1.5MPa
8	13						PL(A) = 1.5MPa
	13.6	Bore discontinued at 13.6m					
7	14						
6	15						
5	16						
4	17						
3	18						
2	19						
1							

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 2 PROJECT 45636.00 JUL 2008
WHALE BEACH 45636 BH2 START 4.3m
5m
6m
7m
8m
4.30 - 9.00M

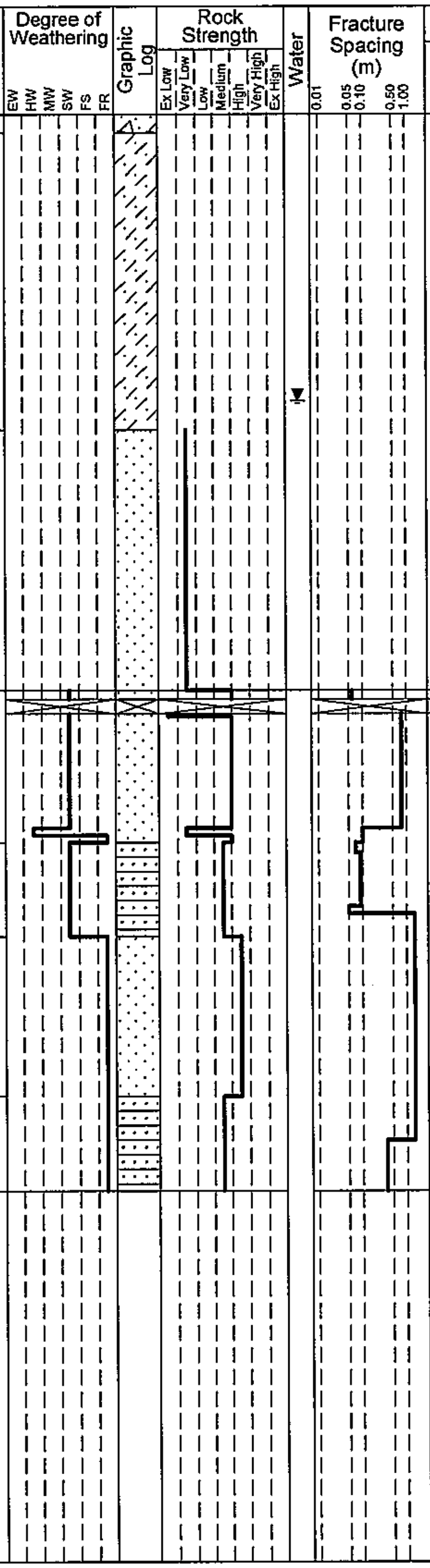

DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 2 PROJECT 45636.00 JUL 2008
9
10
11
12
13
9.00 - 13.60M

BOREHOLE LOG

CLIENT: Leslie Cassar
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	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.13	CONCRETE					
	CLAYEY SAND - loose, light grey brown, fine to medium grained clayey sand, moist		A 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					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	C	100	91	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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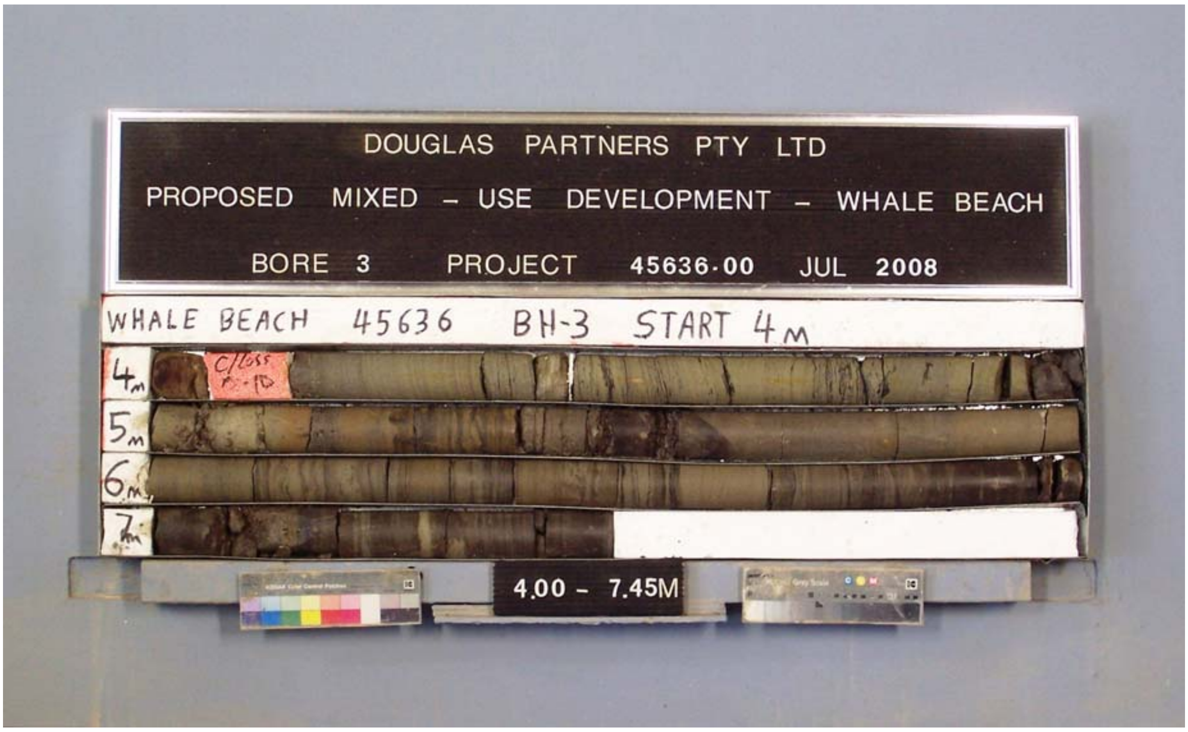
DOUGLAS PARTNERS PTY LTD
PROPOSED MIXED - USE DEVELOPMENT - WHALE BEACH
BORE 3 PROJECT 45636-00 JUL 2008
WHALE BEACH 45636 BH-3 START 4M
4m
C/LOSS
5m
6m
7m
4.00 - 7.45M

Appendix D

Current Field Work Results

BOREHOLE LOG

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

SURFACE LEVEL: 17.8 AHD
EASTING: 345087
NORTHING: 6279727
DIP/AZIMUTH: 90°/--

BORE No: BH101
PROJECT No: 45636.02
DATE: 15/9/2023
SHEET 1 OF 1

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 F - Fault	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
		FILL/ SAND: fine to medium, brown, moist		▨					A			
		Below 0.4m: trace blue-metal gravel		▨					A			
17	1	Below 1.0m: with clay nodules		▨								
		At 1.3m: possible asbestos fragment		▨					A			
16	1.7	Bore discontinued at 1.7m Refusal on concrete										

RIG: Hand tools **DRILLER:** Tightsite **LOGGED:** TM **CASING:** Uncased
TYPE OF BORING: Hand auger to 1.7m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

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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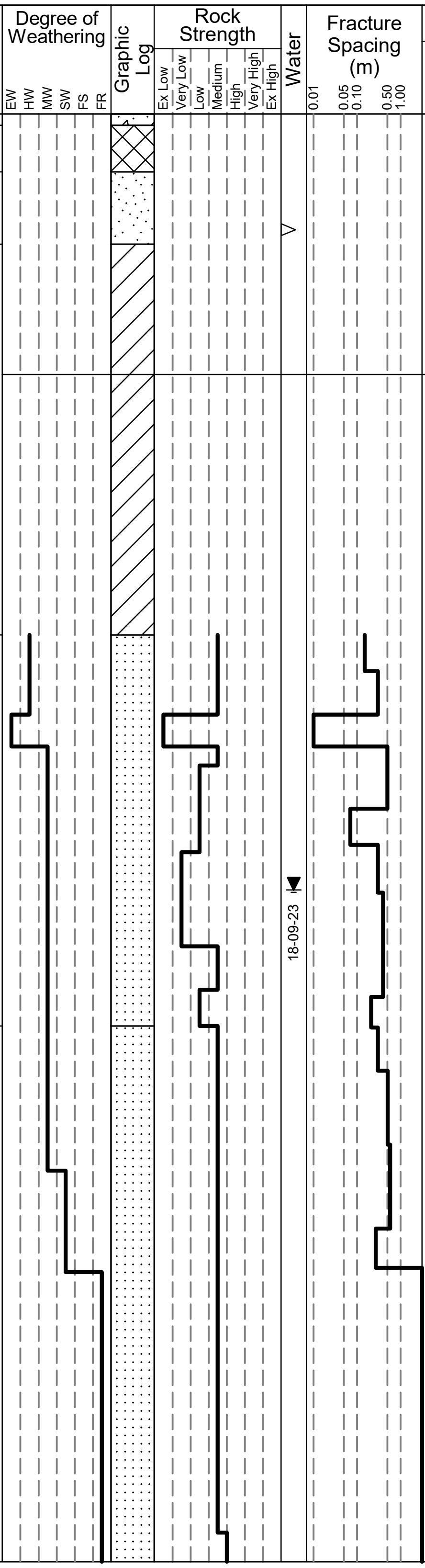
BOREHOLE LOG

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

SURFACE LEVEL: 18.5 AHD
EASTING: 345086
NORTHING: 6279727
DIP/AZIMUTH: 90°/--

BORE No: BH102
PROJECT No: 45636.02
DATE: 14/9/2023
SHEET 1 OF 2

Depth (m)	Description of Strata
0.08	LEAN MIX CONCRETE
0.4	FILL/ SAND: fine to coarse, grey-brown, trace blue-metal igneous and sandstone gravel, moist
0.9	SAND SP: fine to medium, grey, with clay, moist, colluvial Below 0.8m: wet
	CLAY CI: medium plasticity, red-brown mottled yellow-brown, w<PL, apparently stiff, residual
3.6	SANDSTONE: fine to medium grained, pale grey and yellow-brown, very low to medium strength, highly weathered, fractured, Hawkesbury Sandstone
6.3	SANDSTONE: fine to medium, pale grey, medium to high strength, moderately weathered then fresh, slightly fractured to unbroken, Hawkesbury Sandstone



Discontinuities

B - Bedding J - Joint
S - Shear F - Fault

3.85m: B0, pl, ro, cly co
4.15m: Ds 220mm
4.9m: J 70-90 (x3), pl, ro, cly inf 2-5mm
5.38m: J60, pl, ro, cln
5.7m: Ds 30mm
6.1m: Cs 5mm
6.31m: Cs 10mm
6.61m: Ds 70mm & fe stn
7.12m: B5, pl, ro, fe stn
7.7m: B15, pl, ro, fe stn
7.97m: B15, pl, ro, fe stn

Sampling & In Situ Testing

Type	Core Rec. %	RQD %
A		
A		
A		
C	100	0
C	100	0
C	100	0
C	100	90
C	100	100
C	100	100

Test Results & Comments

PL(A) = 0.45
PL(A) = 0.14
PL(A) = 0.09
PL(A) = 0.57
PL(A) = 0.1
PL(A) = 0.42
PL(A) = 0.44
PL(A) = 0.85
PL(A) = 0.97

RIG: Hand tools **DRILLER:** Tightsite **LOGGED:** TM **CASING:** HQ to 1.8m

TYPE OF BORING: Diatube to 0.08m, Hand auger to 1.8m, NMLC coring to 16.00m

WATER OBSERVATIONS: Groundwater seepage at 0.8m

REMARKS: Location coordinates are in MGA94 Zone 56. Groundwater well installed to 16.00m (screen 16.0-10.0m; blank 10.0-0.0; gravel 15.96-5.0m; bentonite 5.0-2.0m; backfill to GL; gatic at surface)

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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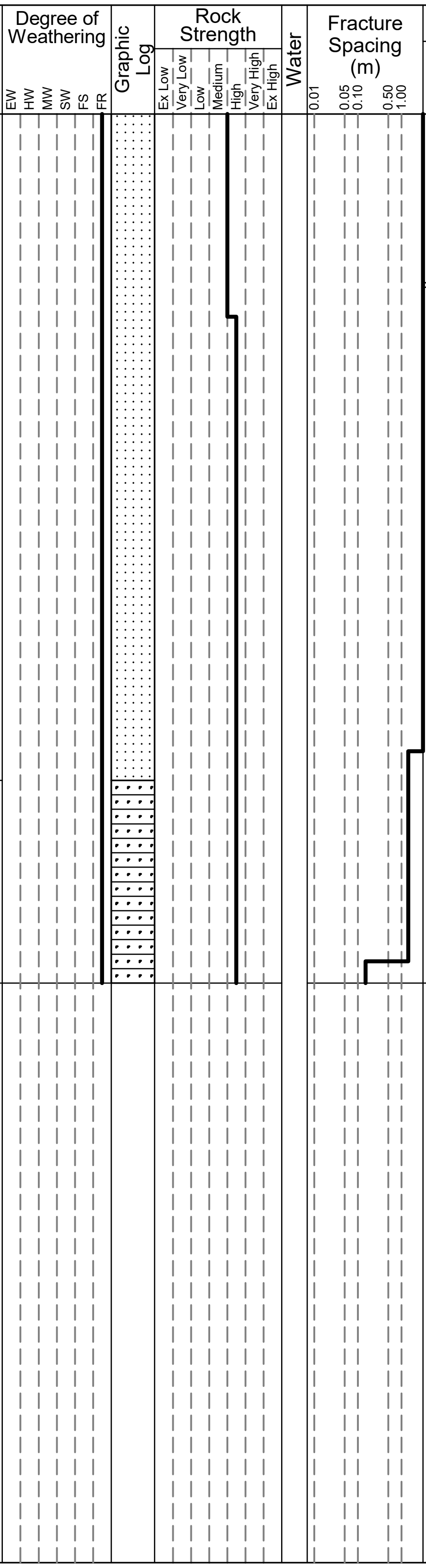
BOREHOLE LOG

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

SURFACE LEVEL: 18.5 AHD
EASTING: 345086
NORTHING: 6279727
DIP/AZIMUTH: 90°/--

BORE No: BH102
PROJECT No: 45636.02
DATE: 14/9/2023
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
8		SANDSTONE: fine to medium, pale grey, medium to high strength, moderately weathered then fresh, slightly fractured to unbroken, Hawkesbury Sandstone *(continued)*		C	100	100	PL(A) = 1
	11		>>				
7	12			C	100	100	PL(A) = 1
6	13						PL(A) = 1.2
5	14			C	100	100	PL(A) = 1.3
4			14.4m: Cs 10mm				
	14.6	LAMINITE: fine grained, pale grey sandstone (50%) interbedded with dark grey siltstone (50%), high strength, fresh, unbroken, Hawkesbury Sandstone		C	100	100	PL(A) = 1.3
3			15.85m: J50, pl, sm, cln				PL(A) = 1.3
	16 16.0	Bore discontinued at 16.0m Target depth reached					
2	17						
1	18						
0	19						
-1							

RIG: Hand tools **DRILLER:** Tightsite **LOGGED:** TM **CASING:** HQ to 1.8m

TYPE OF BORING: Diatube to 0.08m, Hand auger to 1.8m, NMLC coring to 16.00m

WATER OBSERVATIONS: Groundwater seepage at 0.8m

REMARKS: Location coordinates are in MGA94 Zone 56. Groundwater well installed to 16.00m (screen 16.0-10.0m; blank 10.0-0.0; gravel 15.96-5.0m; bentonite 5.0-2.0m; backfill to GL; gatic at surface)

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: 102 PROJECT: WHALE BEACH SEPTEMBER 2023

1.80 – 6.00m

BORE: 102 PROJECT: WHALE BEACH SEPTEMBER 2023

6.00 – 11.00m

BORE: 102 PROJECT: WHALE BEACH SEPTEMBER 2023

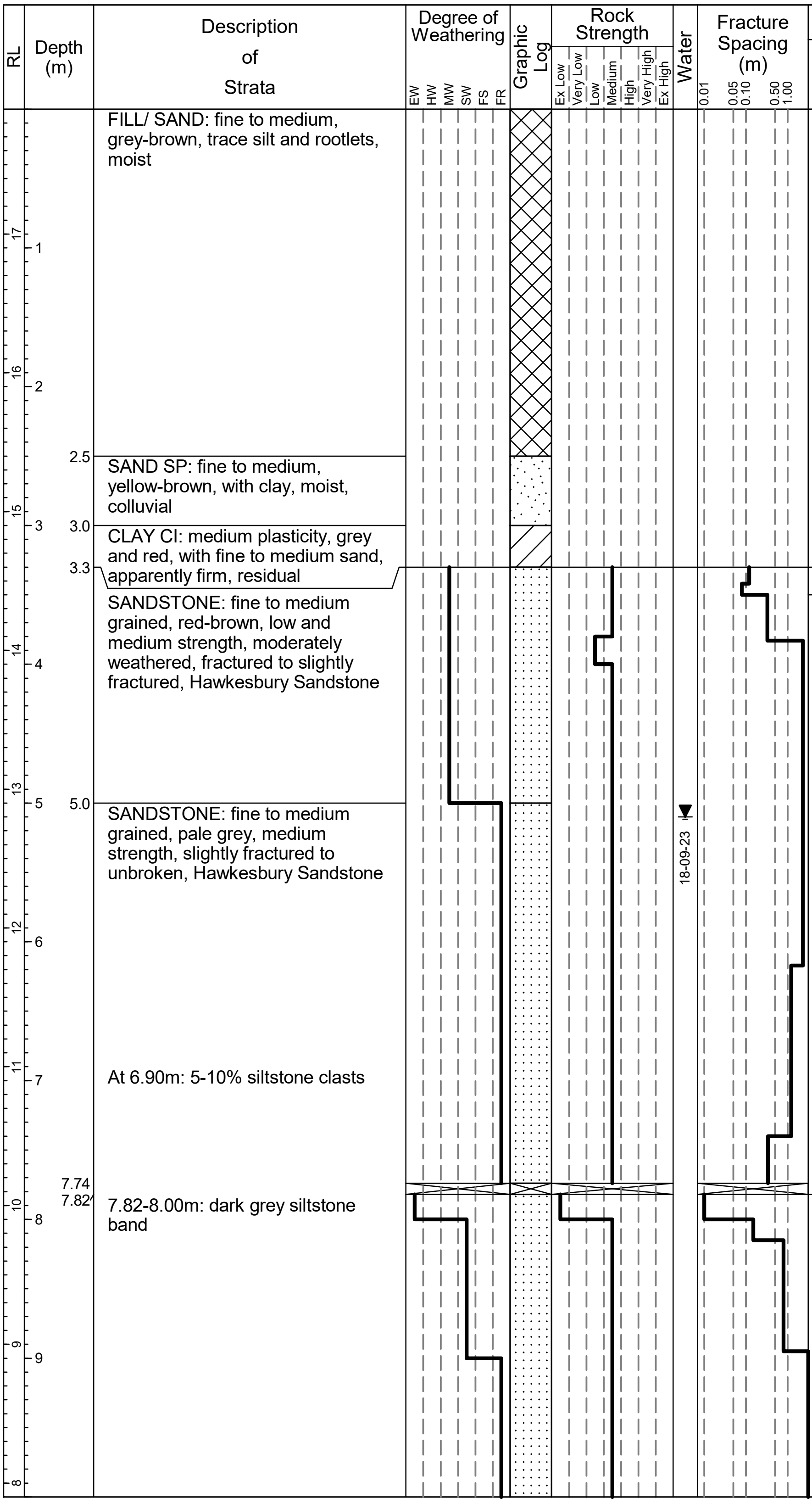
11.00 – 16.00m

BOREHOLE LOG

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

SURFACE LEVEL: 17.9 AHD
EASTING: 345083
NORTHING: 6279710
DIP/AZIMUTH: 90°/--

BORE No: BH103
PROJECT No: 45636.02
DATE: 15 - 18/9/2023
SHEET 1 OF 2



Depth (m)	Description of Strata	Water	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	FILL/ SAND: fine to medium, grey-brown, trace silt and rootlets, moist						
2.5	SAND SP: fine to medium, yellow-brown, with clay, moist, colluvial						
3.0	CLAY CI: medium plasticity, grey and red, with fine to medium sand, apparently firm, residual			A			
3.3	SANDSTONE: fine to medium grained, red-brown, low and medium strength, moderately weathered, fractured to slightly fractured, Hawkesbury Sandstone		3.42m: B15, pl, ro, fe stn, ti 3.5m: B0, pl, ro, cly co 3.83m: Cs 5mm	C	100	90	PL(A) = 0.38 PL(A) = 0.26 PL(A) = 0.42 PL(A) = 0.54
5.0	SANDSTONE: fine to medium grained, pale grey, medium strength, slightly fractured to unbroken, Hawkesbury Sandstone	18-09-23	6.17m: B5, pl, un, cbs vn	C	100	100	PL(A) = 0.48
	At 6.90m: 5-10% siltstone clasts		7.4m: J80, pl, ro, cln	C	95	85	PL(A) = 0.43 PL(A) = 0.42
7.74 7.82	7.82-8.00m: dark grey siltstone band		7.74m: CORE LOSS: 80mm 7.82m: Ds 180mm 8.15m: B0, pl, ro, fe stn				PL(A) = 0.53
			8.95m: B5, pl, ro, fe stn	C	100	100	PL(A) = 0.33
				C	100	100	PL(A) = 0.95

RIG: Hand tools **DRILLER:** Tightsite **LOGGED:** TM **CASING:** HQ to 3.3m

TYPE OF BORING: Hand auger to 3.3m, NMLC coring to 15.96m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Location coordinates are in MGA94 Zone 56. Groundwater well installed to 15.96m (screen 15.96-10.0m; blank 10.0-0.0; gravel 15.96-6.0m; bentonite 6.0-2.0m; backfill to GL; gatic at surface)

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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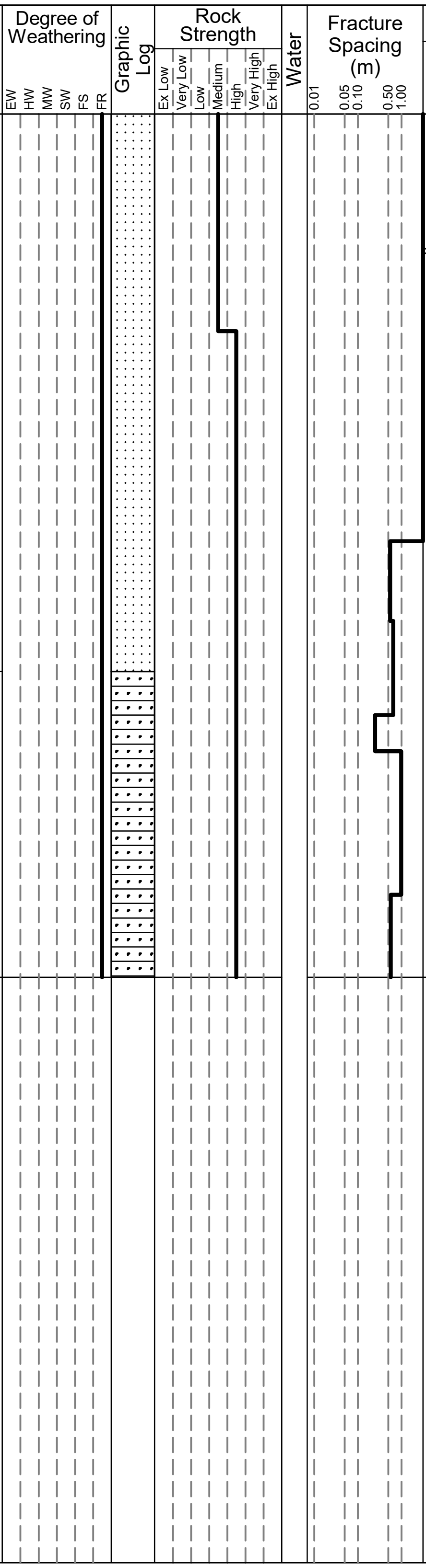
BOREHOLE LOG

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

SURFACE LEVEL: 17.9 AHD
EASTING: 345083
NORTHING: 6279710
DIP/AZIMUTH: 90°/--

BORE No: BH103
PROJECT No: 45636.02
DATE: 15 - 18/9/2023
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		7.82-8.00m: dark grey siltstone band *(continued)*		C	100	100	
7	11		>>				PL(A) = 0.73
6	12			C	100	100	PL(A) = 1.2
5	13		12.95m: J85, pl, ro, cln				PL(A) = 1.1
			13.5m: J60, pl, ro, fg 10mm	C	100	99	PL(A) = 1.5
4	13.85 14	LAMINITE: fine grained pale grey sandstone (50%) interbedded with dark grey siltstone (50%), high strength, fresh, unbroken, Hawkesbury Sandstone	14.15m: J70, pl, ro, cln 14.4m: J40, pl, ro, cly vn				PL(A) = 1.2
3	15			C	100	100	PL(A) = 1.3
			15.39m: J40, pl, ro, cly vn				
2	15.96 16	Bore discontinued at 15.96m Target depth reached					PL(A) = 2.7
1	17						
0	18						
-1	19						
-2							



RIG: Hand tools **DRILLER:** Tightsite **LOGGED:** TM **CASING:** HQ to 3.3m
TYPE OF BORING: Hand auger to 3.3m, NMLC coring to 15.96m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56. Groundwater well installed to 15.96m (screen 15.96-10.0m; blank 10.0-0.0; gravel 15.96-6.0m; bentonite 6.0-2.0m; backfill to GL; gatic at surface)

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: 103 PROJECT: WHALE BEACH SEPTEMBER 2023

3.30 – 8.00m

BORE: 103 PROJECT: WHALE BEACH SEPTEMBER 2023

8.00 – 13.00m

BORE: 103 PROJECT: WHALE BEACH SEPTEMBER 2023

13.00 – 15.96m

Permeability Testing - Rising or Falling Head Test Report

Client:	Leslie Cassar	Project No:	45636.02
Project:	Proposed Mixed-Use Development	Test date:	14-Sep-23
Location:	231 Whale Road, Whale Beach	Tested by:	TM

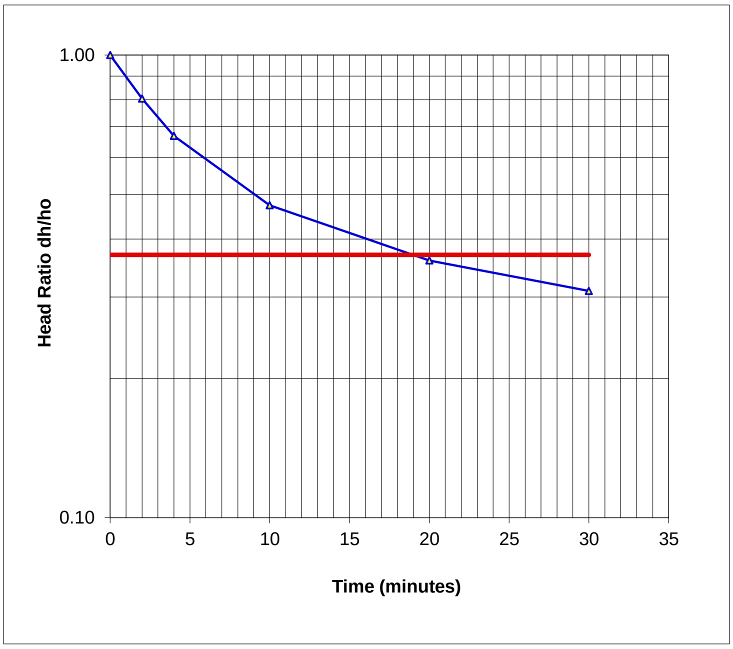
Test Location		**Test No.**	BH2	
Description:	Standpipe in borehole	Easting:	345069.1	m
Material type:	Sandstone	Northing	6279707.6	m
		Surface Level:	20.7	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	3	m
borehole diameter (2R)	76	mm	Depth to water at start of test	6.17	m
Effective Length of well screen (Le)	9.30	m	Depth of top of PVC standpipe	4.30	m
			Depth of base of PVC standpipe	13.60	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	6.17	3.17	1.000
2.0	5.55	2.55	0.804
4.0	5.12	2.12	0.669
10.0	4.50	1.50	0.473
20.0	4.14	1.14	0.360
30.0	3.98	0.98	0.309



To = 19 mins
1140 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **3.7E-07** m/sec
= **0.032** m/day

Permeability Testing - Rising or Falling Head Test Report

Client:	Leslie Cassar	Project No:	45636.02
Project:	Proposed Mixed-Use Development	Test date:	14-Sep-23
Location:	231 Whale Road, Whale Beach	Tested by:	TM

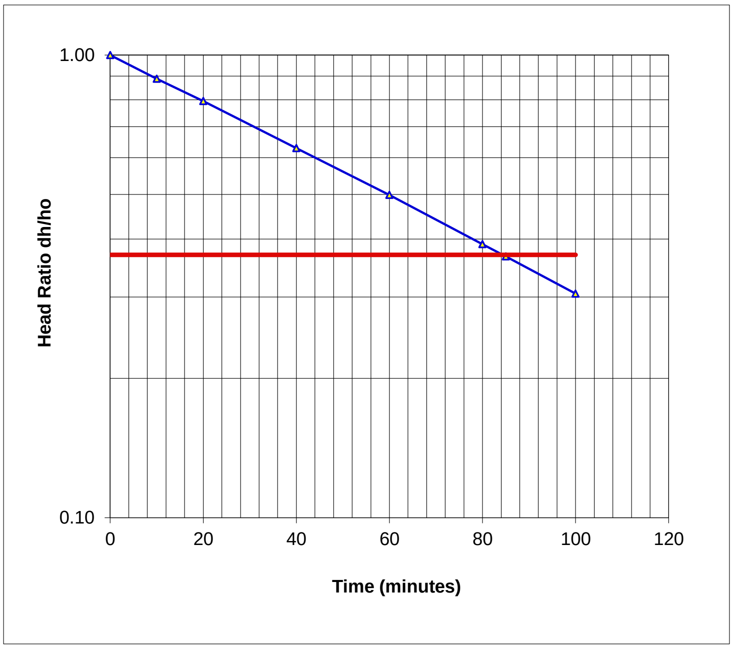
Test Location		**Test No.**	BH3	
Description:	Standpipe in borehole	Easting:	345110.3	m
Material type:	Sandstone and Laminte	Northing	6279716.6	m
		Surface Level:	8.7	m AHD

Details of Well Installation

Effective diameter (2re)	120	mm	Depth to water before test	1.4	m
borehole diameter (2R)	76	mm	Depth to water at start of test	3.99	m
Effective Length of well screen (Le)	5.30	m	Depth of top of PVC standpipe	2.20	m
			Depth of base of PVC standpipe	7.50	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	3.99	2.59	1.000
10.0	3.70	2.30	0.888
20.0	3.46	2.06	0.795
40.0	3.03	1.63	0.629
60.0	2.69	1.29	0.498
80.0	2.41	1.01	0.390
85.0	2.35	0.95	0.367
100	2.19	0.79	0.305



To = 85 mins
5100 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	k =	**3.3E-07**	m/sec
	=	**0.028**	m/day

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Permeability Testing - Rising or Falling Head Test Report

Client: Leslie Cassar
Project: Proposed Mixed-Use Development
Location: 231 Whale Road, Whale Beach

Project No: 45636.02
Test date: 19-Sep-23
Tested by: TM

Test Location
Description: Standpipe in borehole
Material type: Sandstone

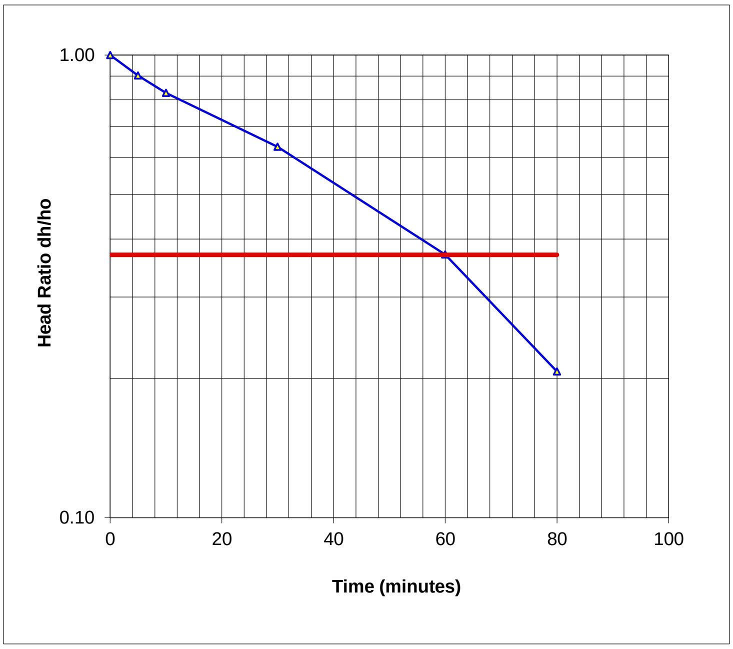
Test No. BH102
Easting: 345086.1 m
Northing 6279726.8 m
Surface Level: 18.5 m AHD

Details of Well Installation

Effective diameter (2re)	76	mm
borehole diameter (2R)	76	mm
Effective Length of well screen (Le)	10.64	m
Depth to water before test	5.36	m
Depth to water at start of test	10.19	m
Depth of top of PVC standpipe	5.00	m
Depth of base of PVC standpipe	16.00	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.0	10.19	4.83	1.000
5.0	9.72	4.36	0.903
10.0	9.36	4.00	0.828
30.0	8.42	3.06	0.634
60.0	7.15	1.79	0.371
80.0	6.36	1.00	0.207



To = 60 mins
3600 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity **k = 1.1E-07** m/sec
= 0.009 m/day

Permeability Testing - Rising or Falling Head Test Report

Client:	Leslie Cassar	Project No:	45636.02
Project:	Proposed Mixed-Use Development	Test date:	19-Sep-23
Location:	231 Whale Road, Whale Beach	Tested by:	TM

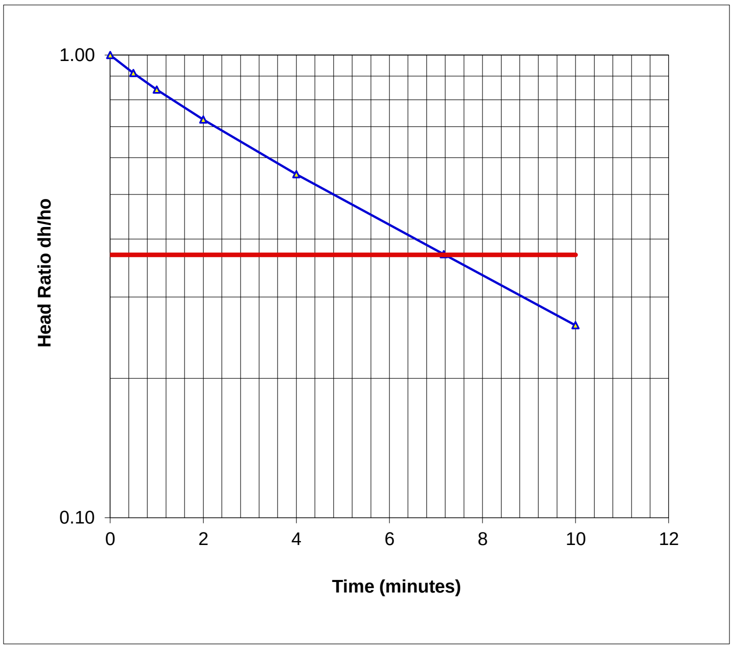
Test Location		**Test No.**	BH103	
Description:	Standpipe in borehole	Easting:	345086.1	m
Material type:	Sandstone	Northing	6279726.8	m
		Surface Level:	18.5	m AHD

Details of Well Installation

Effective diameter (2re)	76	mm	Depth to water before test	5.1	m
borehole diameter (2R)	76	mm	Depth to water at start of test	9.9	m
Effective Length of well screen (Le)	10.00	m	Depth of top of PVC standpipe	6.00	m
			Depth of base of PVC standpipe	16.00	m

Test Results

Time (min)	Depth (m)	Change in Head dH (m)	dH/Ho
0.00	9.90	4.80	1.000
0.50	9.49	4.39	0.915
1.00	9.14	4.04	0.842
2.00	8.58	3.48	0.725
4.00	7.75	2.65	0.552
7.17	6.88	1.78	0.371
10.0	6.35	1.25	0.260



To = 7.167 mins
430 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **9.4E-07** m/sec
= **0.081** m/day

Rock Descriptions

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:

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

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

Sampling Methods

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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.

The soil group symbol classifications are given as follows based on two major soil divisions:

- Coarse-grained soils
- Fine-grained soils

Major Divisions				Description	
				Group Symbol*	Typical Name
COARSE-GRAINED SOILS	More than 65% by dry mass, (excluding that larger than 63 mm) is greater than 0.075 mm	GRAVEL	More than 50% of coarse grains are greater than 2.36 mm	GW	Well graded gravels and gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
		GRAVELLY SOILS		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clay gravels, gravel-sand-clay mixtures.
		SAND	More than 50% of coarse grains are less than 2.36 mm	SW	Well graded sands and gravelly sands, little or no fines.
				SP	Poorly graded sands and gravelly sands, little or no fines.
		SANDY SOILS		SM	Silty sand, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures.

* For coarse grained soils where the fines content is between 5% and 12%, the soil shall be given a dual classification eg GP-GM.

FINE-GRAINED SOILS	More than 35% by dry mass, (excluding that larger than 63 mm) is less than 0.075 mm	Liquid Limit less than 35%	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity
		35% <LL< 50%	CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
			Pt	Peat muck and other highly organic soils.

Soil Descriptions

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, 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, 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, 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.

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).

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
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

■	Asphalt
▨	Road base
▨	Concrete
▨	Filling

Soils

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

Sedimentary Rocks

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

Metamorphic Rocks

▨	Slate, phyllite, schist
▨	Gneiss
▨	Quartzite

Igneous Rocks

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

Appendix E

Photo Plates

Photo 1 – View of cracking along the external footpath

Photo 2 – View of cracking along the external stairs I

Photo 3 – View of the cracking in the balcony at the eastern side of the site

Photo 4 – View of the property from Surf Road

Douglas Partners Geotechnics / Environment / Groundwater	Project: 221060.00	Site Photographs
	Date: 13 March 2023	33 Burnell Street, Russell Lea

Appendix F

Australian Geoguides for Slope Management and Maintenance

Australian
Geomechanics
Society

Extract from

Australian Geomechanics

Journal and News of the Australian Geomechanics Society
Volume 42 No 1 March 2007

Extract containing:

“The Australian GeoGuides for Slope Management and Maintenance”

Ref: AGS (2007e)

Landslide Risk Management

ENGINEERS
AUSTRALIA

AusIMM
AUSTRALASIAN INSTITUTE OF MINING & METALLURGY

ISSN 0818-9110

THE AUSTRALIAN GEOGUIDES
FOR SLOPE MANAGEMENT AND MAINTENANCE

AGS Landslide Taskforce, Slope Management and Maintenance Working Group

The Australian Geomechanics Society (AGS) presents on the following pages a guideline on slope management and maintenance, as part of the landslide risk management guidelines developed under the National Disaster Funding Program (NDMP). This Guideline is aimed at home owners, developers and local councils, but also has applicability to a larger audience which includes builders and contractors, consultants, insurers, lawyers, government departments and in fact any person, or organisation, with a responsibility for the management or maintenance of a slope. The objective is to inform those with little or no knowledge of geotechnical engineering about landslides.

Each GeoGuide is a stand-alone document, which is formatted so that it can be printed on two sides of a single A4 sheet. It is expected that the set of GeoGuides will increase with time to cover a range of topics. As things stand:

- **GeoGuide LR1** is an introductory sheet that should be read by all users, since it explains what the LR (landslide risk) series is about and defines terms.
- **GeoGuides LR2, 3 and 4** explain why landslides occur and provide information on different types of landslide.
- **GeoGuide LR5** discusses the critical part that water often plays in relation to landslide occurrence and discusses measures that can be adopted to limit its effect.
- **GeoGuide LR6** refers to retaining walls and their maintenance.
- **GeoGuide LR7** puts the concept of landslide risk into an everyday context, so users can relate a particular landslide risk to other risks that they know they are prepared to take, sometimes on a daily basis.
- **GeoGuide LR8** retains the ideas of good and poor hillside construction practice originally provided by an AGS sub-committee in 1985.
- **GeoGuide LR9** concentrates specifically on effluent and surface water disposal, which is an important topic in some development areas.
- **GeoGuide LR10** is specifically aimed at those who have property on the coast and could be susceptible to coastal erosion processes.
- **GeoGuide LR11** provides information about the benefits of keeping records on inspection and maintenance activities and provides a proforma record sheet for users.

It is recognised that the GeoGuides are likely to be upgraded from time to time. Feedback on use and suggested changes should be sent to the National Chair of the Australian Geomechanics Society. The latest versions of the GeoGuides will be downloadable from the AGS website www.australiangemechanics.org

Through the NDMP, Australian governments (at Commonwealth, State and Local Government levels) are also funding the development of a Landslide Zoning Guideline (AGS 2007a), and a Practice Note Guideline (AGS 2007c) to which interested readers seeking in-depth information should refer.

ACKNOWLEDGEMENTS

These guidelines have been prepared by The Australian Geomechanics Society with funding from the National Disaster Mitigation Program, the Sydney Coastal Councils Group, and The Australian Geomechanics Society.

The Australian Geomechanics Society established a Working Group within a Landslide Taskforce to develop the guidelines. The development of the guidelines was managed by a Steering Committee. Membership of the Working Group, Taskforce and Steering Committee is listed in the Appendix.

Drafts of these GeoGuides have been subject to review by members of the AGS Landslide Taskforce, members of the geotechnical profession and local government.

REFERENCES

AGS (2007a) Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Management. Australian Geomechanics Society, *Australian Geomechanics*, Vol 42, No1.

AGS (2007c). Practice Note Guidelines for Landslide Risk Management. Australian Geomechanics Society. *Australian Geomechanics*, Vol 42, No1,

AGS (2007e). The Australian GeoGuides for slope management and maintenance –. Australian Geomechanics Society. *Australian Geomechanics*, Vol 42, No 1, - this paper.

AUSTRALIAN GEOGUIDE LR1 (INTRODUCTION)

INTRODUCTION TO LANDSLIDE RISK

Photographs courtesy of Greg Kotze and Tony Phillips

AUSTRALIAN GEOGUIDES

The **Australian GeoGuides (LR series)** are a set of information sheets on the subject of landslide risk management and maintenance, published by the Australian Geomechanics Society (AGS). They provide background information intended to help people without specialist technical knowledge understand the basic issues involved. Topics covered include:

LR1 - Introduction
LR2 - Landslides
LR3 - Landslides in Soil
LR4 - Landslides in Rock
LR5 - Water & Drainage
LR6 - Retaining Walls
LR7 - Landslide Risk
LR8 - Hillside Construction
LR9 - Effluent & Surface Water Disposal
LR10 - Coastal Landslides
LR11 - Record Keeping

The GeoGuides explain why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local authority approval (if required) to remove, or reduce, the risk they represent.

Preparation of the GeoGuides has been funded by Australian governments through the National Disaster Mitigation Program (NDMP). This is a national program aimed at identifying and addressing natural disaster risk priorities across Australia. Technical input has been provided by experienced geotechnical engineers, engineering geologists and local government and government agency representatives from around Australia.

BACKGROUND

A number of landslides and cliff collapses occurred in Australia in the 1980's and 1990's in which lives were lost. Of these the Thredbo landslide probably received the most publicity, but there were several others. During this period the AGS issued a number of advisory notes to practitioners in relation to the assessment of landslide risk and its reduction. Building on these notes, and responding to changes in technology, a technical paper known as AGS2000 was prepared. It was followed in 2002 by an intensive nation-wide educational campaign attended by a large number of interested professionals from government departments and private industry. This resulted in an increased awareness of the risks associated with unstable slopes and a changed approach in many government departments responsible for regional planning, domestic development, roads, railways and the maintenance of natural features such as cliffs.

STATUS OF THE GEOGUIDES

The GeoGuides reflect the essence of good practice as perceived by a large number of geotechnical engineers, engineering geologists and other practitioners such as local government planners. **The GeoGuides are generic and do not, and cannot, constitute advice in relation to a specific situation. This must be sought from a geotechnical practitioner with first hand knowledge of the site.** It is expected that some local councils will refer to the GeoGuides and their companion publications in planning and building legislation. Check with your local council to see how it regards these documents. Companion publications to the GeoGuides are:

- AGS (2007a) Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Management Australian Geomechanics Society, *Australian Geomechanics*, Vol 42, No1 and its associated commentary (AGS 2007b).
- AGS (2007c). Practice Note Guidelines for Landslide Risk Management. Australian Geomechanics Society. *Australian Geomechanics*, Vol 42, No1 2007, and its associated "Commentary" (AGS 2007d).

Copies of the above documents are available on the AGS website www.australiangeomechanics.org

AUSTRALIAN GEOGUIDE LR1 (INTRODUCTION)

TERMINOLOGY

Terminology tends to change with time and place and with the context in which it is used. The terms listed below have the following meanings in the GeoGuides:

Consequence	the outcome, or potential outcome, arising from the occurrence of a landslide expressed quantitatively, or qualitatively, in terms of loss, disadvantage, damage, injury, or loss of life.
Discontinuity	in relation to the ground is a crack, a bedding plane (a boundary between strata) or fault (a plane along which the ground has sheared) which forms a plane of weakness and reduces the overall strength of the ground.
Equilibrium	the condition when the forces on a mass of soil or rock in the ground, or on a retaining structure, are equal and opposite.
Factor of safety (FOS)	theoretically the forces available to prevent a part of the ground, or a retaining structure, from moving divided by those trying to move it. A FOS of one or less indicates that failure is likely to occur, but not how likely it is. To allow for unknowns and to limit movements engineers always aim to achieve a FOS significantly larger than one.
Failure	when part of the ground experiences movement as a result of the out of balance forces on it. Failure of a retaining structure means it is no longer able to fulfil its intended function.
Geotechnical practitioner	when referred to in the Australian GeoGuides (LR series), is a professional geotechnical engineer, or engineering geologist, with chartered status in a recognised national professional institution and relevant training, experience and core competencies in landslide risk assessment and management. In some government departments, technical officers are specifically trained to undertake some of the functions of a geotechnical practitioner.
Hazard	a condition with the potential for causing an undesirable consequence. In relation to landslides this includes the location, size, speed, distance of travel and the likelihood of its occurrence within a given period of time.
Landslide	the movement, or the potential movement, of a mass of rock, debris, or earth down a slope.
Likelihood	a qualitative description of probability, or frequency, of occurrence.
Partial saturation	the condition in the ground above the water table where both air and water are present as well as soil, or rock.
Perched water table	a water table above the true water table supported by a low permeability stratum.
Permeability	a measure of the ability of the ground to allow water to flow through it.
Risk	a measure of the probability and severity of an adverse effect to life, health, property or the environment.
Slip failure	landslide.
Stable	the condition when failure will not occur. Over geological time no part of the ground can be considered stable. Over short periods (eg the life of a structure) stability implies a very low likelihood of failure.
Retaining structure	anything built by humans which is intended to support the ground and inhibit failure.
Structure	in relation to rock, or soil, means the spacing, extent, orientation and type of discontinuities found in the ground at a particular location.
Tension crack	a distinct open crack that normally develops in the ground around a landslide and indicates actual, or imminent , failure.
Water table	the level in the ground below which it is saturated and the voids are filled with water.

Photograph courtesy of Phil Flentje

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

LANDSLIDES

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian Landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book "Guideline Document Landslide Hazards" published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board's website www.abcb.gov.au .

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fail again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both "potential" and "actual" landslides must be taken very seriously. They present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with serious consequences. Wetting up of the ground (which may involve a rise in ground water table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- open cracks, or steps, along contours
- ground water seepage, or springs
- bulging in the lower part of the slope
- hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can "run-out" from above, "regress" from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else's land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 - Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10°- 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway
Steep	18°- 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27°- 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks etc.
Extreme	45°- 64°	1 on 0.5	Need rope access to climb slope
Cliff	64°- 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

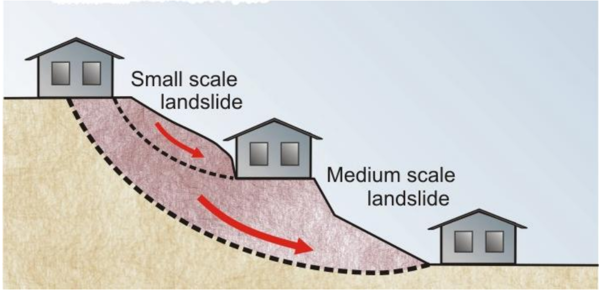

Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

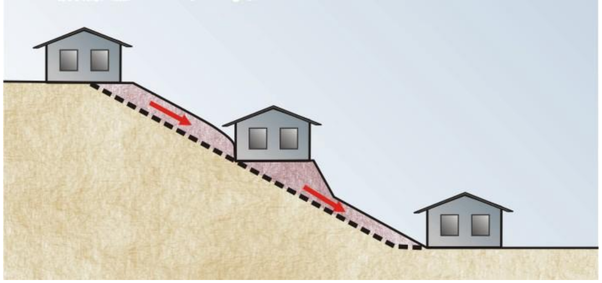

Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain apparently unchanged for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

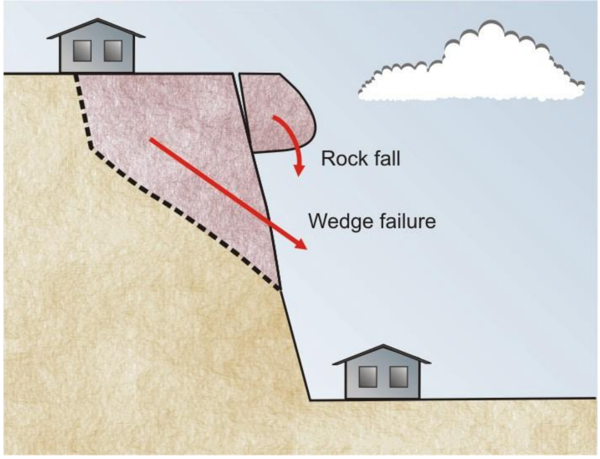

Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

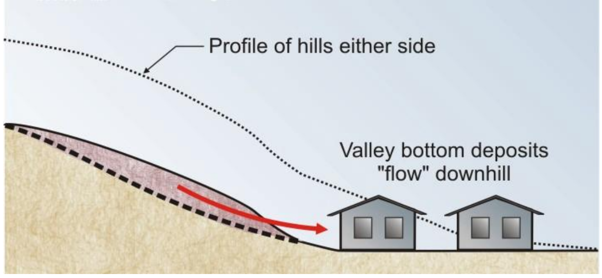

Figure 4

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

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

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

AUSTRALIAN GEOGUIDE LR3 (LANDSLIDES IN SOIL)

LANDSLIDES IN SOIL

Landslides occur on soil slopes and the consequences can include damage to property and loss of life. Soil slopes exist in all parts of Australia and can even occur in places where rock outcrops can be seen on the surface. If you live on, or below, a soil slope it is important to understand why a landslide might occur and what you can do to reduce the risk it presents.

It is always worth asking the question *"why is this slope here?"*, because the answer often leads to an understanding of what might happen in the future. Slopes are usually formed by weathering (breakdown) and erosion (physical movement) of the natural ground - the "parent material". Many factors are involved including rain, wind, chemical change, temperature variation, plant growth, animal activity and our own human enthusiasm for development. The general process is outlined in Figure 1.

The upper levels of the parent material progressively weather over thousands, or millions, of years, losing strength. This can result in a surface layer which looks similar to the parent material (although its colour has probably changed) but has the strength of a soil - this is called "residual soil". At some stage the weathered surface layer is exposed to the elements and fragments are transported down the slope. In this context a fragment could be a single sand grain, a boulder, or a landslide. The time scale could be anything from a few seconds to many thousands of years. The transported fragments often collect on the lower slopes and form a new soil layer that blankets the original slope - "colluvium". If material reaches a river or the sea it is deposited as "alluvium" or as a "marine deposit". With appropriate changes in river and sea level this material can again find itself on the surface to commence another cycle of weathering and erosion. In places often, but not only, near the coast, this can include sand sized fragments which form beaches and are sometimes blown back onto the land to form dunes.

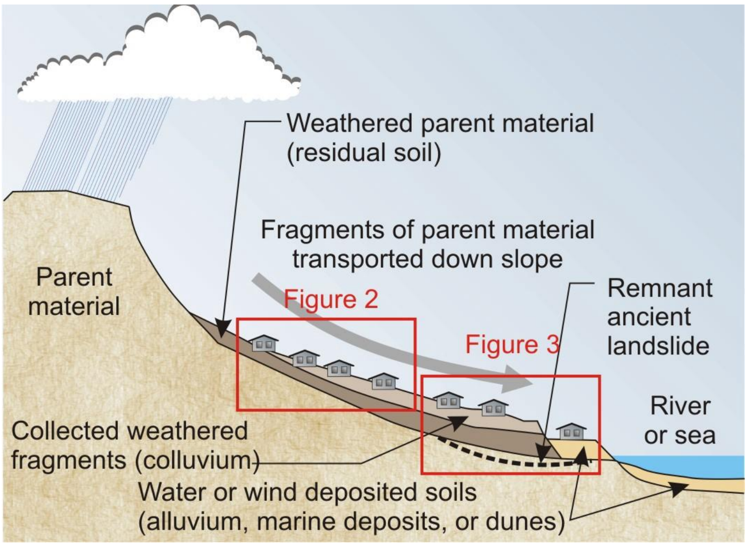

Figure 1

Landslides can occur almost anywhere on a soil slope. Slides can be rotational, translational, or debris flows (see GeoGuide LR2) and may have a number of causes.

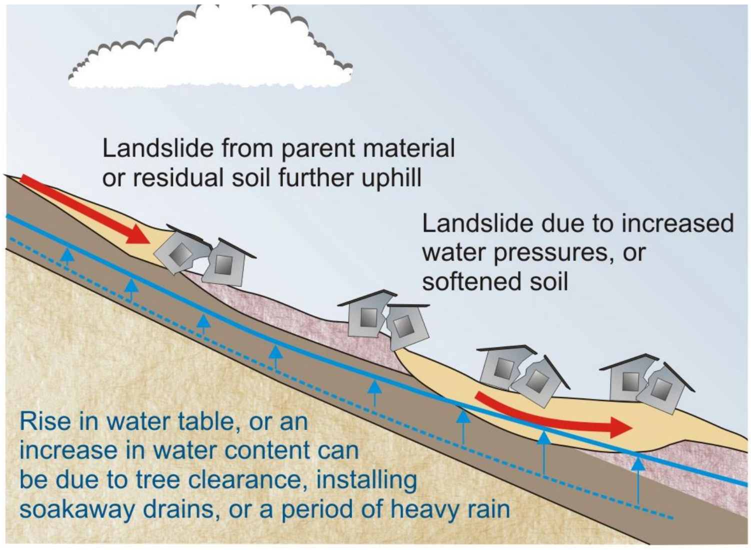

Figure 2

AUSTRALIAN GEOGUIDE LR3 (LANDSLIDES IN SOIL)

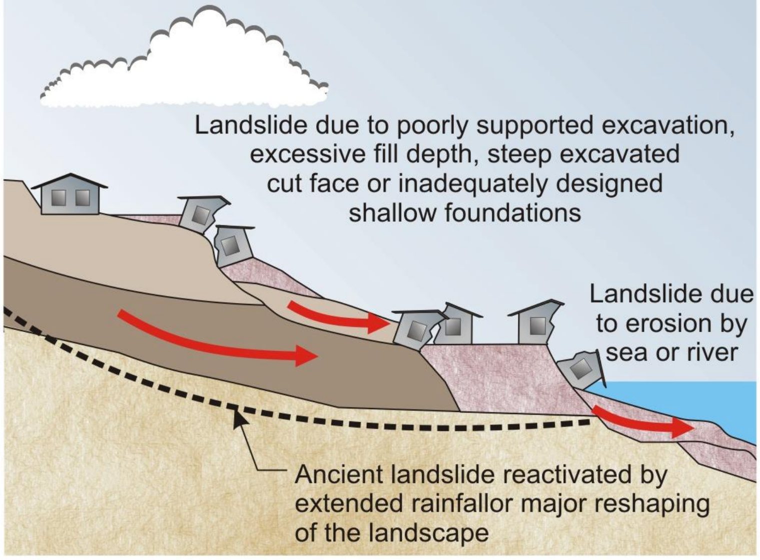

Figure 3

Some of the more common causes of landslides in soil are:

1) Falls of the parent material or residual soil from above, due to natural weathering processes (Figure 2).
2) Increased moisture content and consequent softening of the soil, or a rise in the water table. These can be due to excessive tree clearance, ill-considered soak-away drainage or septic systems, or heavy rainfall (Figure 2).
3) Excavation without adequate support, increased surface load from fill placement, or inadequately designed shallow foundations (Figure 3).
4) Natural erosion at the toe of the slope due to scour by a river or the sea (Figure 3).
5) Re-activation of an ancient landslide (Figure 3).

Most soil slopes appear stable, but they all achieved their present shape through a process of weathering and erosion and are often sensitive to minor changes in the factors that affect their stability. As a general rule, human activities only improve the situation if they have been designed to do so. Once this idea is understood, it is probably easy to see why the following basic rules are so important and should not be ignored without seeking site specific advice from a geotechnical practitioner:

- Do not clear trees unnecessarily.
- Do not cut into a slope without supporting the excavated face with an engineer designed structure.
- Do not add weight to a slope by placing earth fill or constructing buildings with inadequately designed shallow foundations (Note: in certain circumstances weight is added to the toe of a slope to inhibit landslide movement, but this must be carried out in accordance with a proper engineering design).
- Do not allow water from storm water drains, or from septic waste or effluent disposal systems to soak into the ground where it could trigger a landslide.

More information in relation to good and poor hillside construction practice is given in GeoGuide LR8. With appropriate engineering input it is often possible to reduce the likelihood, or consequences, of a landslide and so reduce the risk to property and to life. Such measures can include the construction of properly designed storm water and sub-soil drains, surface protection (GeoGuide LR5) and retaining walls (GeoGuide LR6). **Design should be undertaken by a geotechnical practitioner and will normally require local council approval.**

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

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
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The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR4 (LANDSLIDES IN ROCK)

LANDSLIDES IN ROCK

Rocks have been formed by many different geological processes and may have been subjected to intense pressure, large scale distortion, extreme temperature and chemical change. As a result there are many different rock types and their condition varies enormously. Rock strength varies and is often significantly reduced by the presence of discontinuities (GeoGuide LR1). You may think that rock lasts forever, but in reality it weathers under the combined effects of water, wind, chemical change, temperature variation, plant growth and animal activity and erodes with time. Rock is often the parent material that ends up forming soil slopes (GeoGuide LR3). Inevitably different rocks have different physical and chemical characteristics and they weather and erode to form different types of soil.

Weathering can lead to landslides (GeoGuide LR2) on rock slopes. The type of landslide depends on the nature of rock, the way it has weathered and the presence or absence of discontinuities. It is hard to generalise, though normally a specific combination of discontinuities and material types will be the determining factor and these are often underground and out of sight. Typical examples are provided in the figures 1 to 4. A geotechnical practitioner can assess the landslide risk and propose appropriate maintenance measures. This often entails making geological observations over an area significantly larger than the site and a review of available background information, including records of known landslides and aerial photographs. Depending on the amount of information available, geotechnical investigation may or may not be needed. Every site is different and every site has to be assessed individually.

It is impossible to predict exactly when a landslide will occur on a rock slope, but failure is normally sudden and the consequences can be catastrophic.

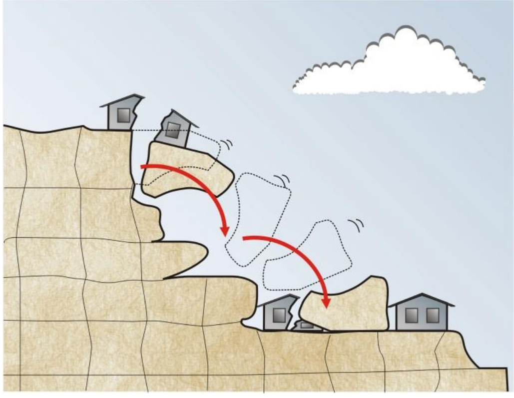

Figure 1 - Failure of an undercut block

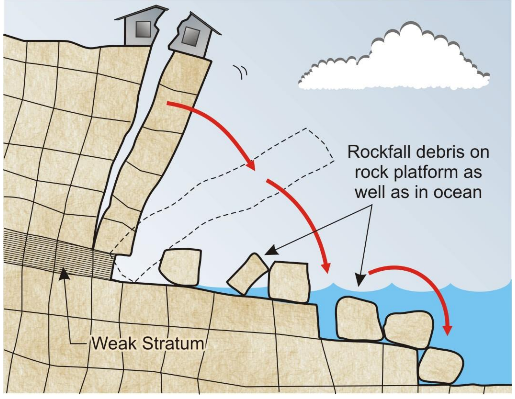

Figure 2 - Toppling failure

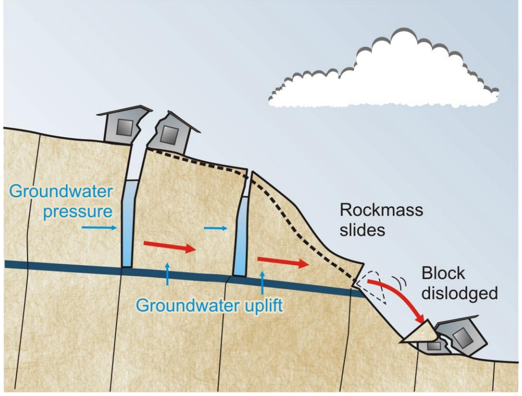

Figure 3 - Block slide on weak layer

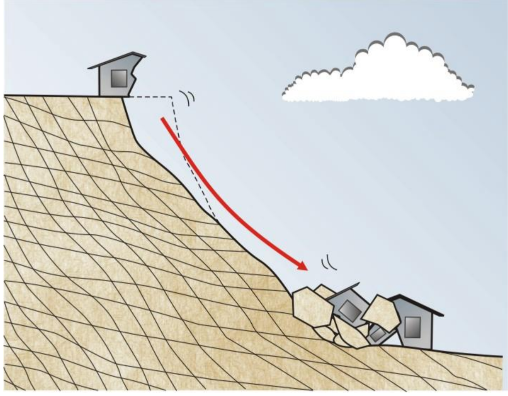

Figure 4 - Wedge failure along discontinuities

If the landslide risk is assessed as being anything other that Low, or Very Low, (GeoGuide LR7) it may be possible to carry out work aimed at reducing the level of risk.

The most common options are:

1) Trimming the slope to remove hazardous blocks of rock.
2) Bolting, or anchoring, to fix hazardous blocks in position and prevent movement.
3) Installation of catch fences and other rockfall protection measures to limit the impact of rockfalls.
4) Deep drainage designed to limit changes in the ground water table (GeoGuide LR5).

Although such measures can be effective, they need inspection and on-going maintenance (GeoGuide LR11) if they are to be effective for periods equivalent to the life of a house. **Design should be undertaken by a geotechnical practitioner and will normally require local council approval.** It should be appreciated that it may not be viable to carry out remedial works in all circumstances: for example where the landslide is on someone else's property, where the cost is out of proportion to the value of the property, or where the risk inherent in carrying out the work is actually greater than the risk of leaving things as they are. In situations such as these, development may be considered inappropriate.

AUSTRALIAN GEOGUIDE LR4 (LANDSLIDES IN ROCK)

ROCK SLOPE HAZARD REDUCTION MEASURES

Removal of loose blocks - may be effective but, depending on rock type, ongoing erosion can result in more blocks becoming unstable within a matter of years. Routine inspection, every 5 or so years, may be required to detect this.

Rock bolts and rock anchors (Figure 5) - can be installed in the ground to improve its strength and prevent individual blocks from falling. Rock bolts are usually tightened using a torque wrench, whilst rock anchors carry higher loads and require jacking. Both can be designed to be "permanent" using stainless steel, or sheathing, to inhibit corrosion, but the cost can be up to 10 times that of the "temporary" alternative. You should inspect rock bolts and rock anchors for signs of water seepage, rusting and deterioration around the heads at least once every 5 years. If you notice any of these warning signs, have them checked by a geotechnical practitioner. It is recommended that you keep copies of design drawings and maintenance records (GeoGuide LR11) for the anchors on your site and pass them on to the new owner should you sell.

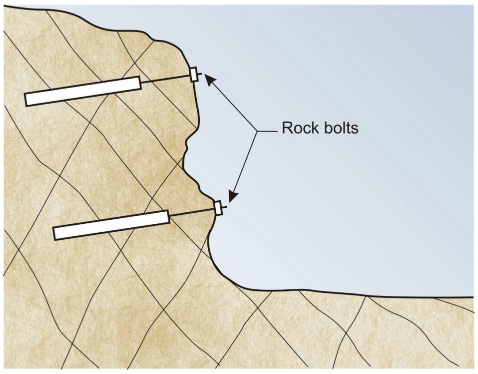

Figure 5

Rock fall netting, catch fences and catch pits (Figure 6) - are designed to catch or control falling rocks and prevent them from damaging nearby property. You should inspect them at least once every 5 years, and after major falls, and arrange for fallen and trapped rocks to be removed if they appear to be filling up. Check for signs of corrosion and replace steel elements and fixings before they lose significant strength.

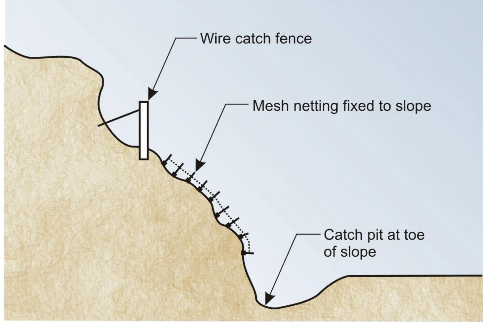

Figure 6

Cut-off drains (Figure 7) - can be used to intercept surface water run-off and reduce flows down the cliff face. Suitable drains are often excavated into the rock, or constructed from mounds of concrete, or stabilised soil, depending on conditions. Drains must be laid to a fall of at least 1% so they drain adequately. Frequent inspection is needed to ensure they are not blocked and continue to function as intended.

Clear trees and large bushes (Figure 7) - from slopes since roots can prize boulders from the face increasing the landslide hazard.

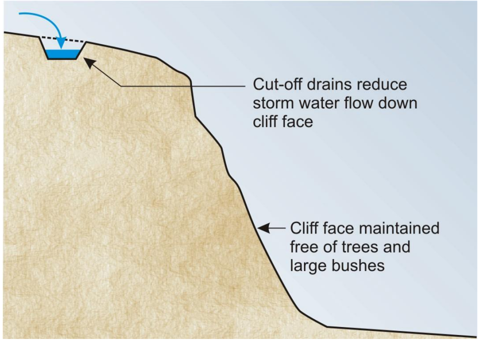

Figure 7

Natural cliffs and bluffs - often present the greatest hazard and yet are easily overlooked, because they have "been there forever". They can exist above a building, road, or beach, presenting the risk of a rock falling onto whatever is below. They also sometimes support buildings with a fine view to the horizon. Cliffs should be observed frequently to ensure that they are not deteriorating. You may find it convenient to use binoculars to look for signs of exposed "fresh" rock on the face, where a recent fall has occurred, or to go to the foot of the cliff from time to time to see if debris is collecting. A thorough inspection of a cliff face is often a major task requiring the use of rope access methods and should only be undertaken by an appropriately qualified professional. If tension cracks are observed in the ground at the top of a cliff take immediate action, since they could indicate imminent failure. **If you have any concerns at all about the possibility of a rock fall seek advice from a geotechnical practitioner.**

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

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AUSTRALIAN GEOGUIDE LR5 (WATER & DRAINAGE)

WATER, DRAINAGE & SURFACE PROTECTION

One way or another, water usually plays a critical part in initiating a landslide (GeoGuide LR2). For this reason, it is a key factor to be controlled on sites with more than a low landslide risk (GeoGuide LR7).

Groundwater and Groundwater Flow

The ground is permeable and water flows through it as illustrated in Figure 1. When rain falls on the ground, some of it runs along the surface ("surface water run-off") and some soaks in, becoming groundwater. Groundwater seeps downwards along any path it can find until it meets the water table: the local level below which the ground is saturated. If it reaches the water table, groundwater either comes to a halt in what is effectively underground storage, or it continues to flow downwards, often towards a spring where it can seep out and become surface water again. Above the water table the ground is said to be "partially saturated", because it contains both water and air. Suctions can develop in the partially saturated zone which have the effect of holding the ground together and reducing the risk of a landslide. Vegetation and trees in particular draw large quantities of water out of the ground on a daily basis from the partially saturated zone. This lowers the water table and increases suctions, both of which reduce the likelihood of a landslide occurring.

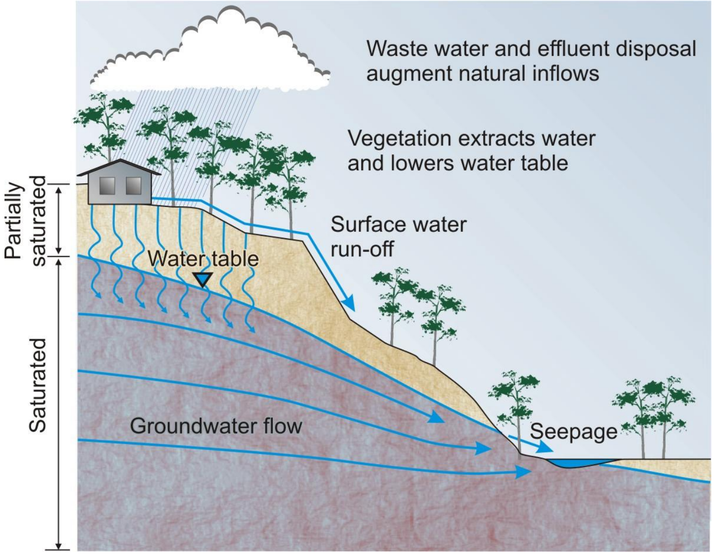

Figure 1 - Groundwater flow

Groundwater Flow and Landslides

The landslide risk in a hillside can be affected by increase in soak-away drainage or the construction of retaining walls which inhibit groundwater flow. The groundwater is likely to rise after heavy rain, but it can also rise when human interference upsets the delicate natural balance. Activities such as felling trees and earthworks can lead to:

- a reduction in the beneficial suctions in the partially saturated zone above the water table.
- increased static water pressures below the water table,
- increased hydraulic pressures due to groundwater flow,
- loss of strength, or softening, of clay rich strata,
- loss of natural cementing in some strata,
- transportation of soil particles.

Any of these effects, or a combination of them, can lead to landslides like those illustrated in GeoGuides LR2, LR3 and LR4.

Limiting the Effect of Water

Site clearance and construction must be carefully considered if changes in groundwater conditions are to be limited. GeoGuide LR8 considers good and poor development practices. Not surprisingly much of the advice relates to sensible treatment of water and is not repeated here. Adoption of appropriate techniques should make it possible to either maintain the current ground water table, or even cause it to drop, by limiting inflow to the ground.

If drainage measures and surface protection are relied on to keep the risk of a landslide to a tolerable level, it is important that they are inspected routinely and maintained (GeoGuide LR11).

The following techniques may be considered to limit the destabilising effects of rising groundwater due to development and are illustrated in Figure 2.

AUSTRALIAN GEOGUIDE LR5 (WATER & DRAINAGE)

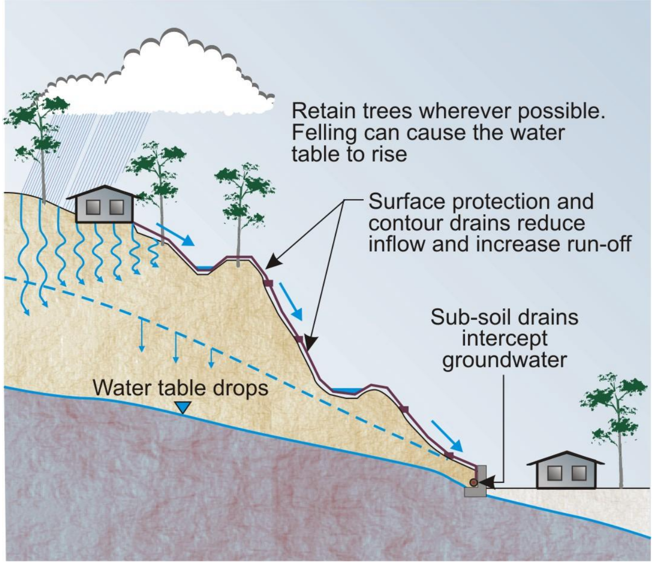

Figure 2 - Techniques used to control groundwater flow

Surface water drains (dish drains, or table drains) - are often used to prevent scour and limit inflow to a slope. Other than in rock, they are relatively ineffective unless they have an impermeable lining. You should clear them regularly, and as required, and not less than once a year. If you live in an area with seasonal rainfall, it is best to do this near the end of the dry season. If you notice that soil or rock debris is falling from the slope above, determine the source and take appropriate action. This may mean you have to seek advice from a geotechnical practitioner.

Surface protection - is sometimes used in addition to surface water drainage to prevent scour and minimise water inflow to a slope. You should inspect concrete, shotcrete or stone pitching for cracking and other signs of deterioration at least once a year. Make sure that weepholes are free of obstructions and able to drain. If the protection is deteriorating, you should seek advice from a geotechnical practitioner.

Sub-soil drains - are often constructed behind retaining walls and on hillsides to intercept groundwater. Their function is to remove water from the ground through an appropriate outlet. It is important that subsoil drains are designed to complement other measures being used. They should be laid in a sand, or gravel, bed and protected with a graded stone or geotextile filter to reduce the chance of clogging. Sub-soil drains should always be laid to a fall of at least 1 vertical on 100 horizontal. Ideally the high end should be brought to the surface, so it can be flushed with water from time to time as part of routine maintenance procedures.

Deep, underground drains - are usually only used in extreme circumstances, where the landslide risk is assessed as not being tolerable and other stabilisation measures are considered to be impractical. They work by permanently lowering the water table in a slope. They are not often used in domestic scale developments, but if you have any on your site be aware that professional maintenance is essential. If they are not maintained and stop working, the water table will rise and a landslide may even occur during normal weather conditions. Both an increase or a reduction in the normal flow from deep drains could indicate a problem if it appears to be unrelated to recent rainfall. If changes of this sort are observed, you should have the drains and your site checked by a geotechnical practitioner.

Documentation - design drawings and specifications for geotechnical measures intended to minimise landslide risk can be of great assistance to a geotechnical specialist, or structural engineer, called in to inspect and report on them. Copies of available documentation should be retained and passed to the new owner when the property is sold (GeoGuide LR11). You should also request details of an appropriate maintenance program for drainage works from the designer and keep that information with other relevant documentation and maintenance records.

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 LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
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AUSTRALIAN GEOGUIDE LR6 (RETAINING WALLS)

RETAINING WALLS

Retaining walls are used to support cuts and fills. Some are built in the open and backfill is placed behind them (gravity walls). Others are inserted into the ground (cast *in situ* or driven piles) and the ground is subsequently excavated on one side. Retaining walls, like all man-made structures, have a finite life. Properly engineered walls should last 50 years, or more, without needing significant repairs. However, not all walls fit this category. Some, particularly those built by inexperienced tradesmen without engineering input, can deflect and even fail because they are unable to withstand the pressures that develop in the ground around them or because the materials from which they are built deteriorate with time. **Design of retaining walls more than 900mm high should be undertaken by a geotechnical practitioner or structural engineer and normally require local council approval.**

Retaining walls have to withstand the weight of the ground on the high side, any water pressure forces that develop, any additional load (surcharge) on the ground surface and sometimes swelling pressures from expansive clays. These forces are resisted by the wall itself and the ground on the low side. Engineers calculate the forces that the retained ground, the water, and the surcharge impose on a wall (the disturbing force) as well as the maximum force that the wall and ground on the low side can provide to resist them (the restoring force). The ratio of the restoring force to the disturbing force is called the "factor of safety" (GeoGuide LR1). Permanent retaining walls designed in accordance with accepted engineering standards will normally have a factor of safety in the range 1.5 to 2.

Never add surcharge to the high side of a wall (e.g. place fill, erect a structure, stockpile bulk materials, or park vehicles) unless you know the wall has been designed with that purpose in mind.

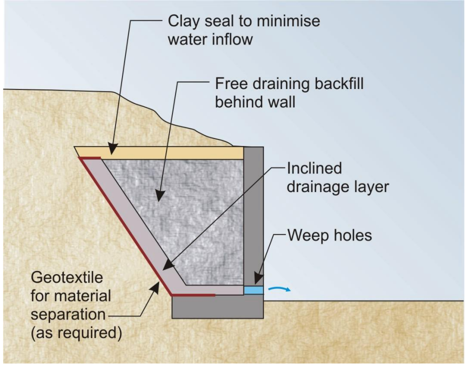

Figure 1- Typical formed concrete wall

Never more than lightly water plants on the high side of a retaining wall.

Never excavate at the toe of a retaining wall.

Any of these actions will reduce the factor of safety of the wall and could lead to failure. If in doubt about any aspect of an existing retaining wall, or changes you would like to make near one, seek advice from a geotechnical practitioner, or a structural engineer. This GeoGuide sets out basic inspection requirements for retaining walls and identifies some common signs that might indicate all is not well. GeoGuide LR11 provides information about records that should be kept.

GRAVITY WALLS

Gravity walls are so called because they rely on their own weight (the force of gravity) to hold the ground behind in place.

Formed concrete and reinforced blockwork walls (Figure 1) **-** should be built so the backfill can drain. They should be inspected at least once a year. Look for signs of tilting, bulging, cracking, or a drop in ground level on the high side, as any of these may indicate that the wall has started to fail. Look for rust staining, which may indicate that the steel reinforcement is deteriorating and the wall is losing structural strength ("concrete cancer"). Ensure that weep holes are clear and that water is able to drain at all times, as high water pressures behind the wall can lead to sudden and catastrophic failure.

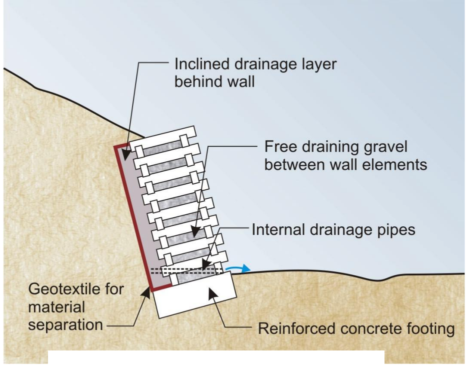

Figure 2 -Typical crib

Concrete "crib" walls (Figure 2) **-** should be filled with clean gravel, or "blue metal" with a nominated grading. Sometimes soil is used to reduce cost, but this is undesirable, from an engineering perspective, unless internal drainage is incorporated in the wall's construction. Without backfill drainage, a soil filled crib wall is likely to have a lower factor of safety than is required. Crib walls should be inspected as for formed concrete walls. In addition, you should check that material is not being lost through the structure of the wall, which has large gaps through it.

Timber "crib" walls - should be checked as for concrete crib walls. In addition, check the condition of the timber. Once individual elements show signs of rotting, it is necessary to have the wall replaced. If you are uncertain seek advice from a geotechnical practitioner, or a structural engineer.

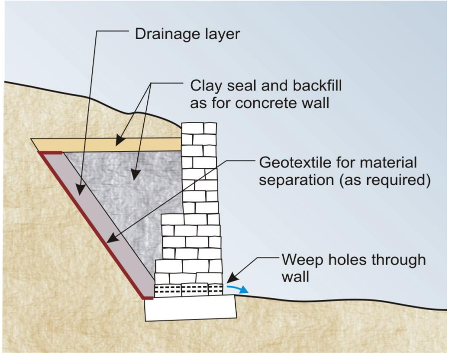

Figure 3 -Typical masonry wall

Masonry walls: natural stone, brick, or interlocking blocks (Figure 3) - more than about 1m high, should be wider at the bottom than at the top and include specific measures to permit drainage of the backfill. They should be checked as for formed concrete walls. Natural stone walls should be inspected for signs of deterioration of the individual blocks: strength loss, corners becoming rounded, cracks appearing, or debris from the blocks collecting at the foot of the wall.

AUSTRALIAN GEOGUIDE LR6 (RETAINING WALLS)

Old Masonry walls (Figure 4) - Many old masonry retaining walls have not been built in accordance with modern design standards and often have a low "factor of safety" (GeoGuide LR1). They may therefore be close to failure and a minor change in their condition, or loading, could initiate collapse. You need to take particular care with such structures and seek professional advice sooner rather than later. Although masonry walls sometimes deflect significantly over long periods of time collapse, when it occurs, is usually sudden and can be catastrophic. Familiarity with a particular situation can instil a false sense of confidence.

Reinforced soil walls (Figure 5) **-** are made of compacted select fill in which layers of reinforcement are buried to form a "reinforced soil zone". The reinforcement is all important, because it holds the soil "wall" together. Reinforcement may be steel strip, or mesh, or a variety of geosynthetic ("plastic") products. The facing panels are there to protect the soil "wall" from erosion and give it a finished appearance.

Most reinforced soil walls are proprietary products. Construction should be carried out strictly in accordance with the manufacturer's instructions. Inspection and maintenance should be the same as for formed concrete and concrete block walls. If unusual materials such as timber, or used tyres, are used as a facing it should be checked to see that it is not rotting, or perishing.

OTHER WALLS

Cantilevered and anchored walls (Figure 6) **-** rely on earth pressure on the low side, rather than self-weight, to provided the restoring force and an adequate factor of safety. These walls may comprise:

- a line of touching bored piers (contiguous bored pile wall) or
- sprayed concrete panels between bored piers (shotcrete wall) or
- horizontal timber or concrete planks spanning between upright timber or steel soldier piles or
- steel sheet piles.

Depending on the form of construction and ground conditions, walls in excess of 3 m height normally require at least one row of permanent ground anchors.

INSPECTION

All walls should be inspected at least once a year, looking for tilting and other signs of deterioration. Concrete walls should be inspected for cracking and rust stains as for formed concrete gravity walls. Contiguous bored pile walls can have gaps between the piles - look for loss of soil from behind which can become a major difficulty if it is not corrected. Timber walls should be inspected for rot, as for timber crib walls. Steel sheet piles should be inspected for signs of rusting. In addition, you should make sure that ground anchors are maintained as described in GeoGuide LR4 under the heading "Rock bolts and rock anchors".

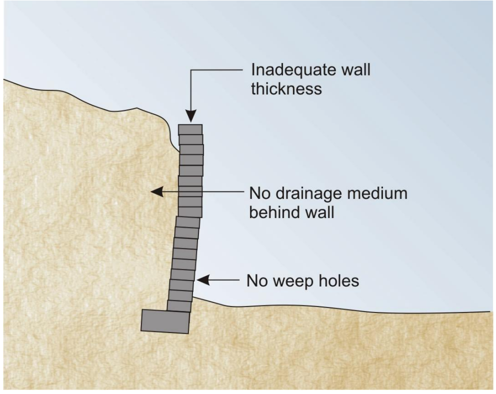

Figure 4 - Poorly built masonry wall

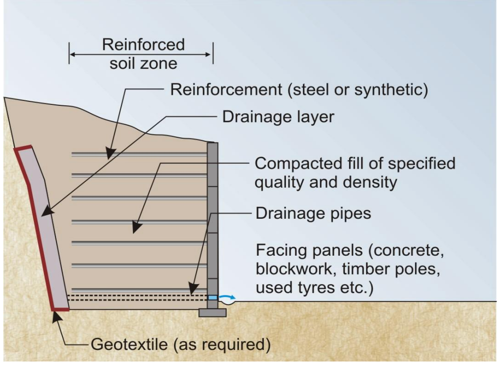

Figure 5 - Typical reinforced soil wall

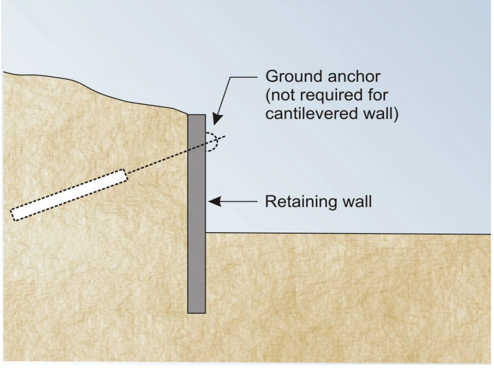

Figure 6 - Typical cantilevered or anchored wall

One of the most important issues for walls is that their internal drainage systems are operational. Frequently verify that internal drainage pipes and surface interception drains around the wall are not blocked nor have become inoperative.

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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

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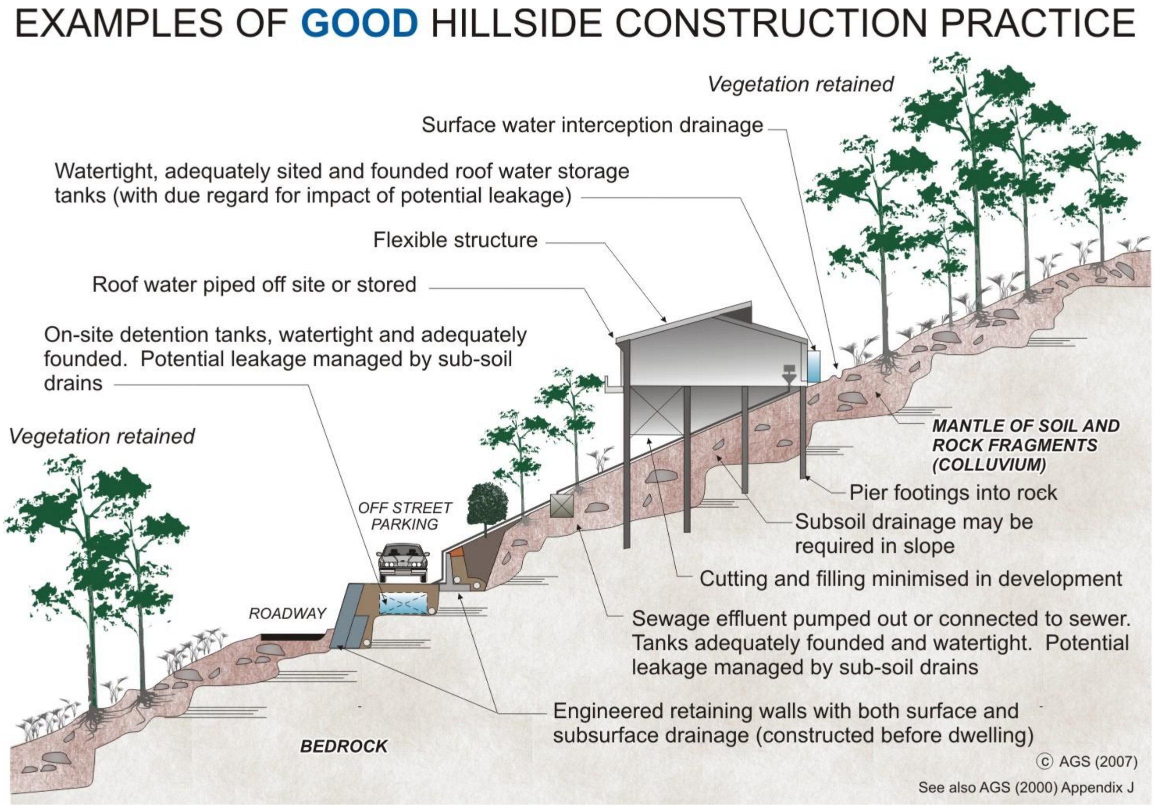
AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

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.

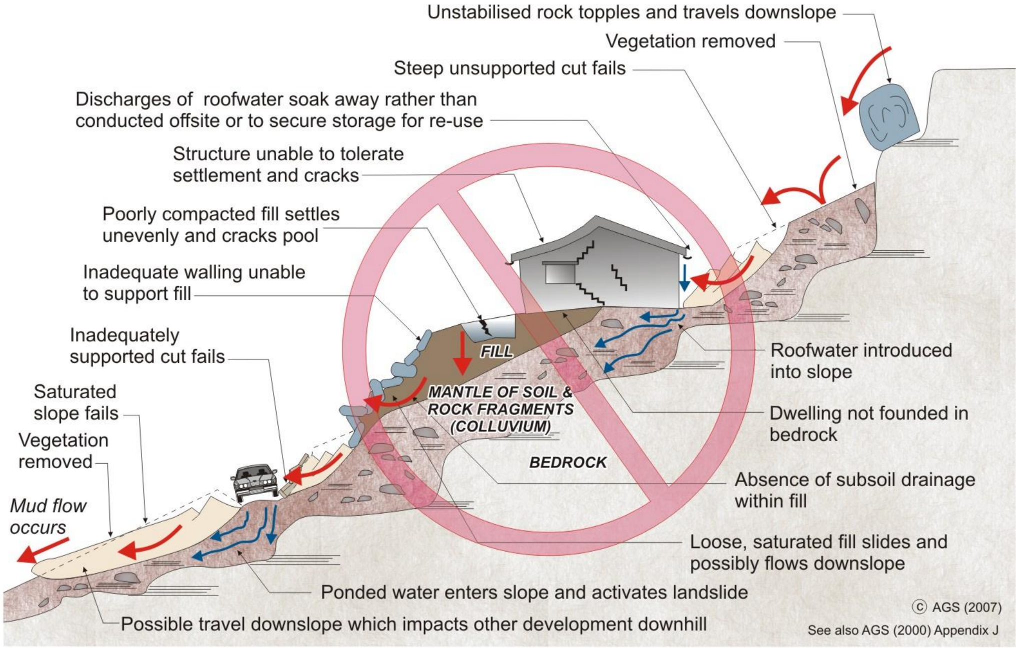
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing 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

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AUSTRALIAN GEOGUIDE LR9 (EFFLUENT DISPOSAL)

EFFLUENT AND SURFACE WATER DISPOSAL

EFFLUENT AND WASTEWATER

All households generate effluent and wastewater. The disposal of these products and their impact on the environment are key considerations in the planning of safe and sustainable communities. Cities and townships generally have reticulated water, sewer and stormwater systems, which are designed to deliver water and dispose of effluent and wastewater with minimal impact on the environment. However, many smaller communities and metropolitan fringe suburbs throughout Australia are un-sewered. Some of these are located in hillside or coastal settings where landslides present a hazard.

Processes by which wastewater can affect slope stability

As explained in GeoGuides LR3 and LR5, groundwater variations have a significant impact on slope stability. Inappropriate disposal of effluent and wastewater may result in the ground becoming saturated. The result is equivalent to a localised rise of the groundwater table and may have the potential to cause a landslide (GeoGuides LR2, LR5 and LR8).

On-site effluent disposal

In un-sewered areas disposal of effluent must be achieved through suitable methods. These methods usually involve containment within the boundaries of the site ("on-site disposal"). State environment protection agencies and local government authorities can usually provide advice on suitable disposal systems for your area. Such systems may include:

- *Septic systems,* which involve a storage/digestion tank for solids, with disposal of the liquid effluent via absorption trenches and beds, leach drains, or soak wells. Such systems are best suited to areas not prone to landslides.
- *Aerobic treatment units* which incorporate an individual household treatment plant to aid breakdown of the waste into a higher quality effluent. Such effluent is further treated and disposed of by surface or sub-surface irrigation, sub-soil dripper, or shallow leach drain system.
- *Nutrient retentive leaching systems* which utilise septic tanks to process the solid and liquid wastes in conjunction with discharge of the effluent through sand filters, media filters, mound systems and nutrient retentive leaching systems, which strip the effluent of nutrients.

Toilet (and sometimes kitchen) waste is known as *black water.* Other, less contaminated, wastewater streams from showers, baths and laundries are known as *grey water*. *Grey water re-use systems* allow a household to conserve water from bathrooms, kitchens and laundries, for re-use on gardens and lawns.

Recommendations for effluent disposal

In areas prone to landslide hazard, it is recommended that whatever effluent disposal system is employed, it should be designed by a qualified professional, familiar with how such a system can impact on the local environment. Local council, and in some instances state environment protection agency, approval is usually required as well. Many local authorities require a site assessment report, which covers all relevant issues. If approved, the report's recommendations must be incorporated in the system design. Reduction in the volume of effluent is beneficial so composting toilets and highly rated (i.e. low consumption) water appliances are recommended. It should be noted that in some state and local government jurisdictions there are restrictions on the alternative measures that can be applied. Consideration should be given to applying treated wastewater to land at low rates and over as large an area as possible. Further guidance can be found in Australian Standard AS/NZS 1547:2000 On-site domestic wastewater management.

Effluent disposal fields should be sited with due consideration to the overall landscape and the individual characteristics of the property. Some guidance is provided. In particular, effluent fields should be located downslope of the building, away from stormwater, or *grey water*, discharge areas and where there is minimal potential for downstream pollution. Set backs and buffer distances vary from state to state and local requirements should be adhered to. All systems require regular maintenance and inspection. Efficient operation of the system must be a priority for property owners/occupiers to ensure safe and sustainable communities. Responsibility for maintenance rests with owners.

SURFACE WATER DRAINAGE

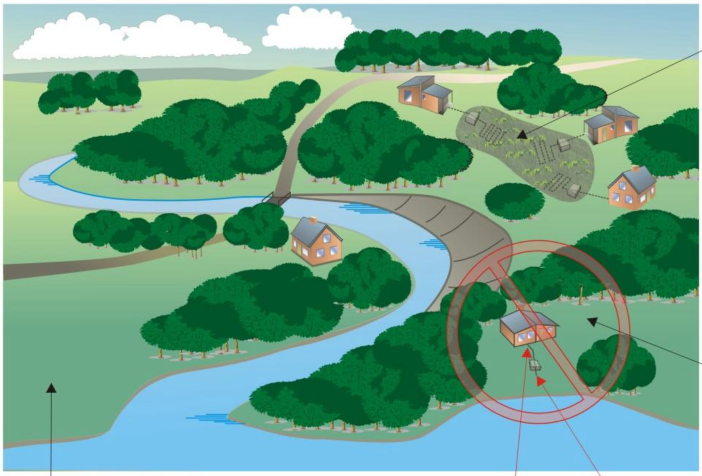
Attention to on-site surface water management is also important. Runoff from developments, including buildings, decks, access tracks and hardstand areas should be collected and discharged away from the development and other effluent disposal fields. Particular care must be given to the design of overflows on water tanks, as this is often overlooked. Discharge from any development should be spread out as much as possible, unless it can be directed to an existing natural water course. Ponding of water on hillsides and the concentration of water flows on slopes must be avoided.

It is recommended that a specific drainage plan and strategy should be developed in conjunction with the effluent disposal system for sites with a high potential for slope instability. Maintenance of the surface water drainage system is as important as maintenance of the effluent disposal system and again the responsibility rests with owners.

AUSTRALIAN GEOGUIDE LR9 (EFFLUENT DISPOSAL)

Avoid concave slopes, depressions and benches

Locate disposal field preferably on downhill side of the house with trenches following the contour, manage landslide risk if this is an issue



Land application area size is determined by soil dependent loading rate

Disposal area planted with shallow rooting grasses and shrubs

Keep access and buildings away from disposal field to retain full soil absorption and evaporation capabilities.

Avoid areas of high groundwater

Disposal field better located on flatter area and away from the water

Special design considerations are required for floodprone land

Disposal trench should be constructed so that landslide risk is tolerable. Seek professional advice if in doubt

Disposal trench too close to waters edge

Reduce effluent volumes through highly rated appliances and grey water re-use systems

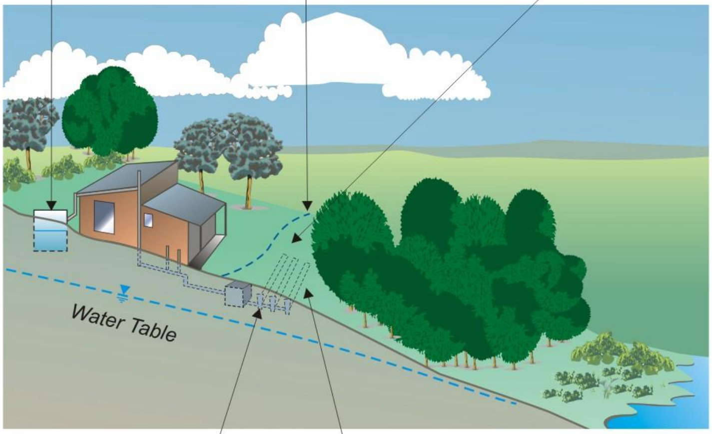
Avoid concentrations of surface water and direct away from effluent fields

Other effluent disposal systems can include soak wells, surface/spray irrigation, drip irrigation and subsurface drippers

Locate underground household water storage uphill and away from disposal field

Direct rainfall runoff away from disposal field with a cut-off drain

Disposal field set back from property boundary in accordance with local provisions



Ensure overflow at water tank is spread broadly across slope

Retain vegetation where possible and plant area with grasses and shrubs to improve operation of disposal field

Disposal system located away from surface waters. Check local provisions

Ensure point of application is above the highest seasonal water table

Locate disposal field (if that is what is required) along the contours of the slope in accordance with local provisions and landslide risk assessment

Note: Adapted from EPA Vic. Publication 451 (March 1996) "Code of Practice - Septic Tanks", which was sourced from Vic. Department of Planning and Loddon-Campaspe Regional Planning Authority.

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AUSTRALIAN GEOGUIDE LR10 (COASTAL LANDSLIDES)

LANDSLIDES IN THE COASTAL ENVIRONMENT

Coastal Instability

The coast presents a particularly dynamic environment where change is often the norm. Hazards exist in relation to both cliffs and sand dunes. The coast is also the most heavily populated part of Australia and always regarded as "prime" real estate, because of the views and access to waterways and beaches.

Photo courtesy Greg Kotze

Waves, wind and salt spray play a significant part, causing dunes to move and cliff-faces to erode well above sea level. Our response is often to try to neutralise these effects by doing such things as dumping rock in the sea, building groynes, dredging, or carrying out dune stabilisation. Such works can be very effective, but ongoing maintenance is usually needed and total reconstruction may be necessary after a relatively short working life.

Of particular significance are extreme events that cause destruction on a scale that ignores our efforts at coastal protection. Records show that cliffs have collapsed, taking with them backyards which had been relied upon as a buffer between a house and the ocean. Sand dunes have also been washed away resulting in the dramatic loss of homes and infrastructure. As with most landslide issues, even though such events may be infrequent, they could happen tomorrow. It is easy to be lulled into a false sense of security on a calm day.

In coastal areas, typical landslide hazards (GeoGuides LR1 to LR4) are compounded by coastal erosion which, over time, undercuts cliffs and eventually results in failure. In the case of sand dunes, dune erosion and dune slumping have equally dramatic effects. Coastal locations are subject to particular processes relating to fluctuating water tables, inundation under storm tides and direct wave attack. Large sections of our more sandy coastline are receding under present sea conditions. The hazards are progressive and likely to be exacerbated through climate change.

Coastal Development

If you own, or are responsible for, a coastal property it is important that you understand that, where the shore line is receding, there is a greater landslide risk than would be the case on a similar site inland. The view may make the risk worthwhile, but does not reduce it.

Coastal Landslides

Coastal landslides are little different from other landslides in that the signs of failure (GeoGuides LR2) and the causes (LR3, LR4 & LR5) are largely the same. The main difference relates to the overriding influence of wave impact, tidal movement, salt spray and high winds.

Cliff failures

In addition to the processes that produce cliff instability on inland cliffs, coastal cliffs are also subjected to repeated cycles of wetting and drying which can be accompanied by the expansive effect of salt crystal growth in gaps in the rocks. These processes accelerate the deterioration of coastal cliffs. At the base of cliffs, direct wave attack and the impact of boulders moved by wave action causes undercutting and hence instability of the overall face. Figure 2 of GeoGuide LR4 provides an example. Whilst the processes leading to coastal cliff collapse may take years, failure tends to be catastrophic and with little warning. In many cases, waves produced by large oceanic storms are the trigger assisted by rainfall to produce collapse. These are also the conditions in which you are more likely to be inside your home and oblivious to unusual noises or movements associated with imminent failure.

Sand dune escarpment and slope failures

An understanding of coastal processes is essential when determining beach erosion potential. Waves produced by large oceanic storms can erode beaches and cut escarpments into dunes. These may be of relatively short duration, when beach re-building happens after the storm, but can be a permanent feature where long term beach recession is taking place. In many locations, houses and infrastructure are sited on or immediately behind coastal dunes. After an escarpment has eroded, those assets may be lost or damaged by subsequent slumping of the dune. It is important that, on erodible coastal soils, the potential for landward incursion of an erosion escarpment is determined. Having done this, the likelihood of slope instability can be established as part of the landslide risk management process. Injury, death and structural damage have occurred around the Australian coast from collapsing sand escarpments.

Photo courtesy DNR NSW

AUSTRALIAN GEOGUIDE LR10 (COASTAL LANDSLIDES)

The large scale and potentially high speed of coastal erosion processes means that major civil engineering work and large cost is normally involved in their control. The installation of rock bolts (LR4), drainage (LR5), or retaining walls (LR6) on a single house site may be necessary to provide local stability, but are unlikely to withstand the attack of a large storm on a beach or cliff-line.

BUILDING NEAR CLIFFS AND HEADLANDS

Coastal cliffs and headlands exist because the rock that they are made from is able to resist erosion. Even so, cliff-faces are not immune and will continue to collapse (Figure 1) by one or other of the mechanisms shown on GeoGuide LR4. If you live on a coastal cliff, you should undertake inspection and maintenance as recommended in LR4 and the other GeoGuides, as appropriate. The top of the cliff, its face, and its base should be inspected frequently for signs of recent rock falls, opening of cracks, and heavy seepage which might indicate imminent failure. Since the sea can remove fallen rocks rapidly, inspections should be made shortly after every major storm as a matter of course. **If collapses are occurring seek advice from an appropriately experienced geotechnical practitioner. Advise you local council if you believe erosion is rapid or accelerating.**

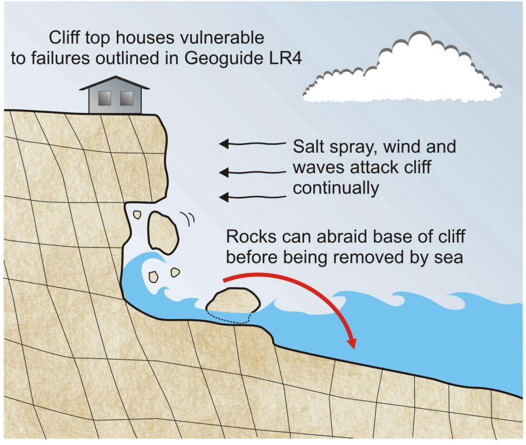

Figure 1

Building on Coastal Dunes

Any excavation in a natural dune slope is inherently unstable and must be supported and maintained (GeoGuide LR6). Dunes are particularly susceptible to ongoing erosion by wind and wave action and extreme changes can occur in a single storm. Whilst vegetation can help to stabilise dunes in the right circumstances, unfortunately a single storm has the potential to cut well into dunes and, in some cases, remove an entire low lying dune system or shift the mouth of a river. **As for cliffs, it is appropriate to observe the effects of major storms on the coastline. If erosion is causing the coastline to recede at an appreciable rate, seek advice from suitably experienced geotechnical and coastal engineering practitioners and bring it to the attention of the local council.**

Photo courtesy DNR NSW

CLIMATE CHANGE

The coastal zone will experience the most direct physical impacts of climate change. A number of reviews of global data indicate a general trend of sea level rise over the last century of 0.1 - 0.2 metres. Current rates of global average sea level rise, measured from satellite altimeter data over the last decade, exceed 3 mm/year and are accelerating. The most authoritative and recent (at the time of writing) report on climate change (IPCC, 2007) predicts a global average sea level rise of between 0.2 and 0.8 metres by 2100, compared with the 1980 - 1999 levels (the higher value includes the maximum allowance of 0.2 m to account for uncertainty associated with ice sheet dynamics).

In addition to sea level rise, climate change is also likely to result in changes in wave heights and direction, coastal wind strengths and rainfall intensity, all of which have the capacity to impact adversely on coastal dunes and cliff-faces. A Guideline for responding to the effects of climate change in coastal areas was published by Engineers Australia in 2004.

References

Engineers Australia 2004 'Guidelines for responding to the effects of climate change in coastal and ocean engineering." The National Committee on Coastal and Ocean Engineering , Engineers Australia , updated 2004.

IPCC (2007) *Climate Change 2007: The Physical Science Basis.* Summary for Policy Makers. Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC).

Nielsen, A.F., Lord D.B. and Poulos, H.G. (1992). 'Dune Stability Considerations for Building Foundations', *Aust. Civil Eng. Transactions* CE No.2, 167-174.

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- GeoGuide LR9 - Effluent & Surface Water Disposal
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AUSTRALIAN GEOGUIDE LR11 (RECORD KEEPING)

RECORD KEEPING

It is strongly recommended that records be kept of all construction, inspection and maintenance activities in relation to developments on sloping blocks. In some local authority jurisdictions, maintenance requirements form part of the building consent conditions, in which case they are mandatory.

CONSTRUCTION RECORDS

If at all possible, you should keep copies of drawings, specifications and construction (i.e. "as built") records, particularly if these differ from the design drawings. The importance of these documents cannot be over-emphasised. If a geotechnical practitioner comes to a site to carry out a landslide risk assessment and is only able to see the face of a retaining wall, the heads of some ground anchors, or the outlets of a number of sub-soil drains, it may be necessary to determine how these have been built and how they are meant to work before completing the assessment. This could involve drilling through the wall to determine how thick it is, or probing the length of the drains, or even ignoring the anchors altogether, because it is uncertain how long they are. Such "investigation" of something that may only have been built a few years before is, at best, a waste of time and money and, at worst, capable of coming up with a misleading answer which could affect the outcome of the assessment. Documentary information of this sort often proves to be invaluable later on, so treat it with as much importance as the title deeds to your property.

INSPECTION AND MAINTENANCE RECORDS

If you follow the recommendations of the Australian GeoGuides it is likely that you will either carry out periodic inspections yourself, or you will engage a geotechnical practitioner to do them for you. The collected records of these inspections will provide a detailed history of changes that might be occurring and will indicate, better than your own memory, whether things are deteriorating and, if so, at what rate. Unfortunately, without some form of written record, all information is usually lost each time a property is sold. It is recommended that a prospective purchaser should have a pre-purchase landslide risk assessment carried out on a hillside site, in much the same way that they would commission a structural assessment, or a pest inspection, of the building. If the vendor has kept good records, then the assessment is likely to be quicker and cheaper, and the outcome more reliable, than if none are available. Each site is different, but noting the following would normally constitute a reasonable record of an inspection/maintenance undertaken:

- date of inspection/maintenance and the name and professional status of the person carrying it out
- description of the specific feature (eg. cliff face, temporary rock bolt, cast *in situ* retaining wall, shallow leach drain system)
- sketch plans, sketches and photographs to indicate location and condition
- activity undertaken (eg. visual inspection; cleared vegetation from drain; removed fallen rock about 500 mm diameter)
- condition of the feature and any matters of concern (e.g. weep holes damp and flowing freely; rust on anchor heads getting worse; shotcrete uncracked and no sign of rust stains; ground saturated around leach field)
- specific outcomes (eg. no action necessary; geotechnical practitioner called in to advise on the state of the anchors; cliff face to be trimmed following the most recent rock fall; leach field to be rebuilt at new location)

A proforma record is provided overleaf for convenience. Photographs and sketches of specific observations can prove to be very useful and should be included whenever possible. Geotechnical practitioners may devise their own site specific inspection/maintenance records.

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AUSTRALIAN GEOGUIDE LR11 (RECORD KEEPING)

INSPECTION/MAINTENANCE RECORD

(Tick boxes as appropriate and add information as required) Date..

Site location (street address / lot & DP numbers / map reference / latitude and longitude)

....

FEATURE	Inspected	Maintained	Tested	By Owner	By Professional
Slopes & surface protection:					
☐ Natural slope/cliff ☐ Cut/fill slope	☐	☐	☐	☐	☐
☐ Surface water drains	☐	☐	☐	☐	☐
☐ Shotcrete ☐ Stone pitching ☐ Other	☐	☐	☐	☐	☐
Retaining walls:					
☐ Cast in situ concrete ☐ Concrete block	☐	☐	☐	☐	☐
☐ Masonry (natural stone) ☐ Masonry (brick, block)	☐	☐	☐	☐	☐
☐ Cribwall (concrete) ☐ Cribwall (timber)	☐	☐	☐	☐	☐
☐ Anchored wall ☐ Reinforced soil wall	☐	☐	☐	☐	☐
☐ Sub-soil drains ☐ Weep holes	☐	☐	☐	☐	☐
Ground improvement:					
☐ Rock bolts	☐	☐	☐	☐	☐
☐ Ground anchors ☐ Soil nails	☐	☐	☐	☐	☐
☐ Deep subsoil drains	☐	☐	☐	☐	☐
Effluent and storm water disposal systems:					
☐ Effluent treatment system	☐	☐	☐	☐	☐
☐ Effluent disposal field	☐	☐	☐	☐	☐
☐ Storm water disposal field	☐	☐	☐	☐	☐
Other:					
☐ Netting ☐ Catch fence ☐ Catch pit	☐	☐	☐	☐	☐
☐ --	☐	☐	☐	☐	☐
☐ --	☐	☐	☐	☐	☐
☐ --	☐	☐	☐	☐	☐

Observations/Notes (Add pages/details as appropriate)

....

....

....

Attachments: ☐ Sketch(es) ☐ Photograph(s) ☐ Other (eg measurements, test results)

Record prepared by .... (name): ....(signature)

Contact details: Phone:....................................... E-mail:...

Professional Status (in relation to landslide risk assessment):...

APPENDIX

AUSTRALIAN GEOMECHANICS SOCIETY

STEERING COMMITTEE

Andrew Leventhal, GHD Geotechnics, Sydney, Chair
Robin Fell, School of Civil and Environmental Engineering, UNSW, Sydney, Convenor Guidelines on Landslide Susceptibility, Hazard and Risk Working Group
Tony Phillips, Consultant, Sydney, Convenor Slope Management and Maintenance Working Group
Bruce Walker, Jeffery and Katauskas, Sydney, Convenor Practice Note Working Group
Geoff Withycombe, Sydney Coastal Councils Group, Sydney

WORKING GROUP - Guidelines on Slope Management and Maintenance
Tony Phillips, Tony Phillips Consulting, Sydney, Convenor
Henk Buys, NSW Roads and traffic Authority, Parramatta
John Braybrooke, Douglas Partners, Sydney
Tony Miner, A.G. Miner Geotechnical, Geelong

LANDSLIDE TASKFORCE

Laurie de Ambrosis, GHD Geotechnics, Sydney
Mark Eggers, Pells Sullivan Meynink, Sydney
Max Ervin, Golder Associates, Melbourne
Angus Gordon, retired, Sydney
Greg Kotze, GHD, Sydney
Arthur Love, Coffey Geotechnics, Newcastle
Alex Litwinowicz, GHD Geotechnics, Brisbane
Tony Miner, A.G. Miner Geotechnical, Geelong
Fiona MacGregor, Douglas Partners, Sydney
Garry Mostyn, Pells Sullivan Meynink, Sydney
Grant Murray, Sinclair Knight Merz, Auckland
Garth Powell, Coffey Geotechnics, Brisbane
Ralph Rallings, Pitt and Sherry, Hobart
Ian Stewart, NSW Roads and Traffic Authority, Sydney
Peter Tobin, Wollongong City Council, Wollongong
Graham Whitt, Shire of Yarra Ranges, Lillydale