Aaron Kendall
C/- Thodey Design

martens consulting engineers

# Preliminary Geotechnical Investigation: 104A Wakehurst Parkway, Elanora Heights, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P1705950JR02V02
July 2023

**Limitations Statement**

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**Document and Distribution Status**

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| 1 | Preliminary Geotechnical Investigation | Final | 09.08.2018 | 1H, 1P, 1E | 1P | | |
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**All enquiries regarding this project are to be directed to the Project Manager.**

# Contents

# 1 Development and Investigation Scope

The proposed development details and investigation scope are summarised in Table 1.

**Table 1:** Summary of proposed development and investigation scope.

| Item | Details |
|---|---|
| Property Address | 104 Wakehurst Parkway, Elanora Heights, NSW |
| Lot / DP | Lot 11, DP 1014199 |
| LGA | Northern Beaches Council (NBC) – formerly Pittwater Council |
| Background | We understand from a brief by the client that the existing property will be subdivided into two different properties as follows:<br>o 104 Wakehurst Parkway – southern portion of the property (front lot), which contains the existing development.<br>o 104A Wakehurst Parkway – northern portion of the property (rear lot), which contains no building currently ('the site').<br>This report has been prepared for 104A Wakehurst Parkway ('the site') only. Another report has been prepared for 104 Wakehurst Parkway by MA, details of which are presented in MA report reference P1705950JR01V01, dated July 2018 (MA, 2018). This report includes details of boreholes undertaken for the proposed development in the front lot (MA, 2018). |
| Site Area | 1187.6 $m^2$ (TD, 2018) |
| Proposed Development | The proposal plans (TD, 2023) indicate that the development at the site will include construction of a new two storey residential dwelling, including a ground level garage. This will likely require bulk excavation of up to approximately 6 m. |
| Council Mapped Hazard Risk | The site is located within Potential Geotechnical Landslip Hazard Area H1 (LEP, 2014). |
| Assessment Purpose | This preliminary geotechnical investigation has been prepared to address NBC requirements for a Development Application (DA). |
| Investigation Scope of Work<br>(2 July 2018) | o Review of DBYD survey plans.<br>o A site walkover survey.<br>o Two boreholes (BH101B and BH102B) up to 1.5 mBGL (refer Attachment B, and associated explanatory notes in Attachment H).<br>o Three Dynamic Cone Penetrometer (DCP) tests (DCP101B to DCP103B) up to 2.45 mBGL (refer Attachment C).<br>Two boreholes (BH101A and BH102A, included in Attachment B) and four DCPs (DCP101A to DCP104A, included in Attachment C) were conducted in the front lot (i.e. 104 Wakehurst Parkway), results of which have been considered in this reporting, where integral to our assessment.<br>All investigation locations are shown in Figure 1, Attachment A. |

# 2 Findings

## 2.1 General Site Details

General site details are summarised in Table 2.

**Table 2:** Summary of general site details based on desktop review, site walkover and site investigations.

| Item | Comment |
|---|---|
| Topography | Within highly undulating terrain, at the base of a south facing cliff, with an overall grade of approximately 22%. |
| Typical slopes, aspect, elevation | Site slopes vary as follows:<br>o Southern portion: approximately 10 % to the south.<br>o Central portion: approximately 50 % to the south.<br>o Northern portion: a near vertical sandstone cliff.<br>Site elevation ranges between approximately 8.0 mAHD at the southern site boundary and 38.0 mAHD at the top of rock cliff face. Area behind the cliff, up to the northern site boundary, has not been surveyed. |
| Expected geology | *Sydney 1:100,000 Geological Sheet 9130*, 1st edition indicates the following expected geology across the property:<br>Northern portion of the site: Hawkesbury Sandstone comprising medium to coarse grained quartz sandstone, very minor shale and laminite lenses.<br>Southern portion of the site: Newport Formation comprising interbedded laminite, shale and quartz to lithic quartz sandstone and minor red claystone.<br>We expect, from geological map and site observations, that the transition between Hawkesbury Sandstone and Newport Formation is located near the toe of the sandstone cliff. However, the exact location of this transition is unclear. |
| Existing development | A metal shed and an approximately 1 m high sandstone gabion retaining wall between the southern and central site portions, which is in a good and stable condition. |
| Vegetation | o From the southern boundary up to the existing retaining wall: grass and shrubs<br>o From the existing retaining wall up to the rock cliff face: dense vegetation comprising grass, shrubs, bushes and scattered young and mature trees. |
| Drainage | Via overland flow to the south |

## 2.2 Surface and Subsurface Conditions

Inferred medium to high strength sandstone cobbles, boulders and detached blocks were observed across the site. These included large sandstone boulders (> 5 m$^3$), partially buried upslope of the retaining wall as well as two boulders downslope of the retaining wall. Small to medium boulders (< 0.5 m$^3$), mainly unburied, were scattered across the surface upslope of the retaining wall.

Investigation revealed the following generalised subsurface units likely underlie the site:

Unit A: Colluvial loose to dense silty clayey sand, containing sandstone gravels / cobbles / boulders (possibly very low to low medium strength), likely originating from the higher-lying Hawkesbury Sandstone cliff.

Unit B: Residual inferred medium dense and dense clayey sand.

Unit C: Weathered and inferred very low to low strength rock, expected to be part of Newport Formation typically comprising interbedded laminite, shale and quartz to lithic quartz sandstone and minor red claystone.

Table 3 summarises the generalised subsurface profile inferred from site observations, boreholes and DCP test results, to investigation termination depths. Alluvial soil, encountered within the southern portion of the property (i.e. front lot), is expected not to be present within the site. An inferred geological cross section is provided as Figure 2, Attachment A.

**Table 3:** Generalised inferred subsurface profile at investigation locations.

| Unit [1] | Depth (mBGL) [2] | | |
|---|---|---|---|
| | BH101B / DCP101B | BH102B / DCP102B | DCP103B |
| A [5] | 0.0 – 2.0 | 0.0 – 1.1 | 0.0 – 2.0 |
| B [5] | 2.0 – 2.45 [3] | 1.1 – 1.35 [4] | 2.0 – 2.45 [3] |
| C | > 2.45 | > 1.35 | > 2.45 |

**Notes:**

1 Refer to borehole logs for more detailed material descriptions at tested locations.

2 Indicative depth range below ground level, to investigation termination depth, which may vary across site depending on site and local geological conditions.

3 DCP test refusal depth. For the purpose of this report DCP refusal is assumed on top of weathered rock (Unit C).

4 Push tube refusal on weathered and inferred very low to low strength rock.

5 Deeper colluvial and / or residual soil profile may be encountered upslope of the retaining wall up to the rock cliff.

We note that, considering the presence of rock boulders across the site, investigation refusal may be the result of boulders buried within the colluvial soil profile. Possible presence of cobbles/boulders or depth to top of rock should be confirmed / revised following further assessment prior to / during construction stage, as necessary.

## 2.3 Groundwater

Groundwater inflow was not encountered during drilling of the boreholes up to 1.35 mBGL. However, ephemeral perched groundwater may be encountered in the soil and / or rock profile originating from infiltration of surface water during prolonged or intense rainfall events. Groundwater typically causes softening and weathering of materials over time and, together with increase in lateral shear stresses as a result of the ephemeral perched groundwater, could induce shallow and deep slope failures.

Should further information on permanent site groundwater levels be required, additional investigation would need to be carried out (i.e. installation of groundwater monitoring wells).

# 3 Geotechnical Assessment

## 3.1 Geotechnical Landslip Risk Assessment

### 3.1.1 Hazard Assessment

Evidence of former slope movement at the site includes:

1. Shallow (< 2.5 m thick) colluvial soil profile across the lower southern portion of the site. This profile is likely the result of at least two slides:
    i. Large landslide originating near the base of the cliff extending into the southern portion of the property (i.e. 104 Wakehurst Parkway); expected to be a relict landslide.
    ii. Medium landslide originating in over-steepened slopes near the base of the cliff as a result of the original large landslide (see i above) and extending up to the southern site boundary; this slide is also considered to be relict.
2. Large sandstone boulders, originating from the cliff face. Rock weathering, vegetation growth from rock defects and soil inundation indicates these rock falls occurred > 100 years ago.
3. Small to medium sandstone boulders across the northern steeper slope at the base of the cliff. We expect these have fallen / slid recently (within last 100 years).

### 3.1.2 Hazard Risk Assessment

A geotechnical hazard risk assessment for the proposed works has been completed in accordance with the qualitative risk matrices provided in Section 7 of the Australian Geomechanics Society's *Landslide Risk Management Guidelines* (2007). We have considered three main geotechnical hazards, including shallow rotational slide, deep rotational slide and rock fall. These and associated risks are described in the following sections and in Attachment D.

The proposed development is assessed to constitute a high risk to life and property, resulting from assessed geotechnical hazards. A reduction of risk to low may be achieved, subject to adoption of the treatment measures below, recommendations presented in this report and good hillslope engineering practices. Areas likely to be impacted by slope instability risks have been highlighted on the geological cross section

(Figure 2, Attachment A). A description of good hillslope engineering practices is provided as Attachment E.

### 3.1.3 Recommended Treatment Measures

Recommended treatment measures to reduce risks to life and property include:

- Maintain vegetation cover or re-plant deep rooted vegetation after slope treatments.
- Ensure adequate surface and subsurface drainage is provided upslope of the development and behind all retaining walls.
- Do not over-steepen existing grades without suitable shoring support.
- Identify and stabilise / remove existing boulders / detached rock blocks upslope of development area.
- Design and install appropriate long-term rock cliff stabilisation / support systems.
- Do not place excessive load onto retaining walls or existing and final sloping surfaces unless designed for.
- Ensure the retaining walls associated with the garage level will support surcharge loading from sloping ground and colluvium behind the wall and will resist / support any future shallow and deep colluvial soil slides.
- Ensure building and retaining wall footings are founded in rock, designed with appropriate and sufficient socket in rock.
- Install inclinometer upslope of the development to identify and monitor possible slope movements.
- Regularly inspect slopes and the rock cliff upslope of the development to identify evidence of potential slides and / or rock falls. This should be carried out by an experienced geotechnical engineer.

Subject to adopting these treatments, our geotechnical recommendations provided in Sections 3.3 and in Attachment F and adopting good hillslope engineering practices, provided as Attachment E, we consider the proposed development constitutes a low and acceptable risk to life and property. However, we note that it will be the decision of stakeholders to accept the risks.

Geotechnical Risk Management Policy for Pittwater Forms 1 and 1a are provided as Attachment G.

### 3.2 Preliminary Material Strength Properties

Preliminary material strength properties, estimated from field test results in conjunction with borehole derived soil profile data as well as engineering assumptions, are summarised in Table 4.

**Table 4:** Preliminary estimated material properties.

| Unit [1] | $\gamma_{in\text{-}situ}$ [2] ($kN/m^3$) | Φ' [3] (deg) | E' [4] (MPa) | $K_s$ [5] (MPa/m) |
|---|---|---|---|---|
| A | 16-18 | 25-27 | 3-5 | 3-5 |
| B | 18-19 | 28 | 7 | 7 |
| C | 22-23 | 28 | 50 | 20 |

**Notes:**

1 Refer to borehole logs in Attachment B for material description details.
2 Range of in-situ unit weight estimate, based on visual assessment only (± 2 $kN/m^3$).
3 Average effective internal friction angle (± 2 °) estimate assuming drained conditions; may be dependent on rock defect conditions.
4 Effective elastic modulus estimate (±10 %).
5 Modulus of subgrade reaction (vertical).

### 3.3 Geotechnical Recommendations

The following preliminary geotechnical recommendations are provided for the proposed development. Further general geotechnical recommendations are presented in Attachment F.

1. Rock cliff stabilisation: A detailed assessment, including rock mapping, of the rock cliff must be carried out by an experienced geotechnical engineer to assess rock conditions and identify steeply dipping joints and other rock defects and discontinuities that may have an adverse effect on rock stability. This must be followed by the design of appropriate long-term rock cliff stabilisation / support features. An experienced contractor should be engaged for rock stabilisation / support installations and works should be carried out under supervision of a geotechnical engineer.

2. Rock boulder / block stabilisation / removal: Condition of loose boulders and detached rock blocks, located upslope and / or within the proposed development area, should be further assessed by a geotechnical engineer during construction to determine potential adverse impacts on the proposed development and stabilisation / removal requirements. Care should be taken not to dislodge surrounding materials during removal of boulders or blocks.

3. Excavation and support: Excavation could potentially destabilise rock boulders within the colluvial profile. All exposed soil, boulders and weathered rock must be temporarily and permanently supported / retained to maintain excavation stability and limit the risk of slope instability. Appropriate support and / or excavation methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer. The upslope retaining wall associated with the garage level must be designed to support surcharge loading from sloping ground behind the wall and resist / support any future shallow and deep slides.

4. Earth Pressure Coefficients: Retaining wall design may adopt preliminary active, at rest and passive earth pressure coefficients of 0.40, 0.55 and 2.50, respectively, for level ground behind the walls.

5. Site works: Stockpiling of any excavation spoil should be limited during construction to prevent increasing the risk of slope instability, including moving loose boulders and detached blocks.

6. Ground vibrations: We do not recommend the use of rock hammer due to possible negative impacts on slope instability resulting from resultant excessive rock vibrations. Should rock hammering be adopted, we recommend limiting the hammer size to ensure peak particle velocities (PPV) caused by construction equipment or resulting from excavation at the site is below 2 mm/s (AS 2187.2, 2006, Appendix J). A higher value of 5 mm/s may be considered, subject to further assessment by a geotechnical engineer. Vibration monitoring should be undertaken in accordance with AS 2187.2 (2006).

   Care will be required to limit structural distress to neighbouring structures and settlement of very loose and loose foundation materials caused by construction plant / excavation induced ground vibrations.

7. Dilapidation surveys: dilapidation surveys of adjacent structures may be carried out prior to excavation and following completion of the development to clearly identify damage caused by the excavation process.

8. Footings and Foundations: Foundation loading associated with all new structures should be transmitted to the rock. Shallow pad footings or piers, depending on foundation level, may be designed adopting an allowable end bearing capacity of 350

kPa for rock, subject to an embedment of at least 0.5 m. Estimates of safe end bearing capacity and shaft friction for piers founding in rock are 700 kPa and 60 kPa, respectively, subject to an embedment of at least 1.0 m into Class IV rock (P.J.N Pells et al (1998) Foundations on Sandstone and Shale in the Sydney Region). For uplift resistance, we recommend reducing allowable shaft friction by 50% and checking against 'piston' and 'cone' pull-out mechanisms in accordance with AS2159 (2009).

Provided bearing capacity values should be confirmed by a geotechnical engineer on site during construction, as detailed in Section 4.2.

Top of rock was assumed for this report to correspond with investigation refusal depth and is based on limited site investigations and observations. We note that DCP refusal could also be caused by cobbles and / or boulders. Presence of boulders and depth to top of rock should be confirmed / revised by further assessment prior to / during construction stage, as necessary. Additional geotechnical investigation is required to confirm bedrock is present.

9. Drainage requirements: Appropriate surface and subsurface drainage measures should be provided upslope of the development and behind all retaining walls to divert overland flows and ephemeral perched water away from structures and discharge into council stormwater systems downslope of the site. Groundwater inflow, if encountered during rock excavation, is expected to be limited and can be managed by sump and pump methods.

10. Site Classification: The site is classified as a "P" site in accordance with AS 2870 (2011).

# 4 Proposed Additional Works

## 4.1 Works Prior to Construction Certificate

We recommend the following additional geotechnical assessments are carried out to develop the final design and prior to construction:

1. Detailed assessment of near vertical rock cliff to assess rock conditions and long term stabilisation / support requirements.
2. Condition assessment of boulders / blocks located upslope and / or within the proposed development area, for stabilisation / removal requirements.
3. Assessment of rock conditions including rock coring and point load testing of collected rock samples to assess rock strength.
4. Further geotechnical investigation in steeper upslope areas to assess thickness of colluvial / residual soil profile and rock conditions and refine risk assessment results. Subject to the results of further assessment of the steeper slope conditions, installation of inclinometers may be considered to enable identification and monitoring slope movements.
5. Review of the final design by a senior geotechnical engineer, if design carried out by a structural engineer, to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.
6. Review of construction staging plan by a senior geotechnical engineer.

## 4.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 5).

**Table 5**: Recommended inspection / monitoring requirements during site works.

| Scope of Works | Frequency/Duration | Who to Complete [3] |
|---|---|---|
| Inspect excavation retention (shoring, retaining wall, rock bolt) installations and monitor associated performance to assess need for additional support requirements. | Daily / As required [2] | Builder / MA [1] |
| Inspect rock cliff retention installation / stabilisation works to assess need for additional support requirements. | As required [2] | Builder / MA [1] |
| Inspect any exposed rock boulders / blocks to assess need for additional support requirements. | As required [2] | Builder / MA [1] |
| Monitor groundwater seepage from excavation faces, if encountered, to assess stability of exposed materials, suitability of proposed drainage and additional drainage requirements. | When encountered | Builder / MA [1] |
| Monitor excavation-induced vibrations, if required. | Daily at on-set of excavation and as agreed thereafter [2] | MA [1] |
| Monitor settlement and lateral deflection of retaining structures to identify potential excavation impacts on neighbouring properties. | Daily at on-set of excavation and as agreed thereafter | Builder / MA [1] |
| Inspect exposed material at foundation / subgrade level to verify suitability as foundation / lateral support / subgrade. | Prior to reinforcement set-up and concrete placement | MA [1] |
| Monitor sedimentation downslope of excavated areas. | During and after rainfall events | Builder |
| Monitor sediment and erosion control structures to assess adequacy and for removal of built up spoil. | After rainfall events | Builder |
| Inspect upslope of the development, including the rock cliff, to ensure no evidence of slide and / or rock fall. | Daily / As required [2] | MA [1] |
| Monitor inclinometer. | As required [2] | MA [1] |

**Notes:**

[1] MA = Martens and Associates engineer

[2] MA inspection frequency to be determined based on initial inspection findings in line with construction program.

[3] If unsuitable conditions are identified during construction, MA should be contacted immediately for further assessment / advice.

## 4.3 Post Construction Monitoring and Inspections

We recommend the following is inspected and monitored following construction completion (Table 6).

**Table 6**: Recommended inspection / monitoring requirements after completion of construction works.

| Scope of Works | Frequency/Duration | Who to Complete |
|---|---|---|
| Inspect upslope of the development, including the rock cliff, to ensure no evidence of slide and / or rock fall. | At least twice a year for the first 5 years, once a year for the following 5 years and as necessary [2] thereafter. | MA [1] |
| Monitor inclinometer. | | MA [1] |

**Notes:**

[1] MA = Martens and Associates engineer

[2] MA inspection frequency to be determined based on previous findings.

# 5 References


Australian Geomechanics Society (2007) *Practice Note Guidelines For Landslide Risk Management 2007,* Journal and News of the Australian Geomechanics Society Volume 42 No 1 March 2007.

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C.M.S Pty Ltd (2013) *Survey Plan*, Ref No: 476C, Drawing No: 476Cdetail1.dwg, dated 13 September 2013 (CMS, 2013).

Herbert C. (1983) *Sydney 1:100 000 Geological Sheet 9130*, 1st edition, Geological Survey of New South Wales, Sydney.

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Standards Australia Limited (1997) AS 1289.6.3.2:1997, *Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer test*, SAI Global Limited.

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Standards Australia Limited (2011) AS 2870:2011, *Residential slabs and footings*, SAI Global Limited.

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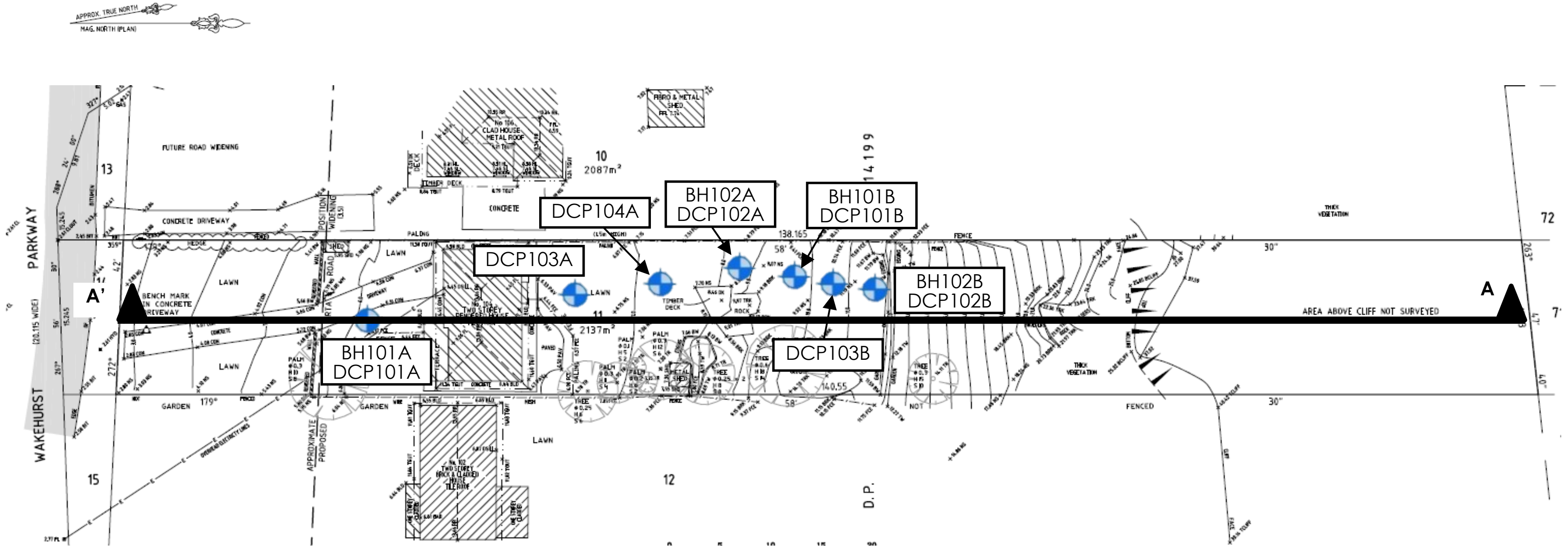
# 6 Attachment A – Figures




**Key:**

Indicative borehole and / or DCP test location

Indicative geological cross section location

| Martens & Associates Pty Ltd ABN 85 070 240 890 | | Environment \| Water \| Wastewater \| Geotechnical \| Civil \| Management | |
|---|---|---|---|
| Drawn: | HN | EXISTING SITE SURVEY AND GEOTECHNICAL TESTING PLAN<br>104A Wakehurst Parkway, Elanora Heights, NSW<br>Source: CMS, 2013 | Drawing:<br>FIGURE 1 |
| Approved: | RE | | |
| Date: | 16.07.2018 | | |
| Scale: | NA | | Job No. P1705950JR02V02 |

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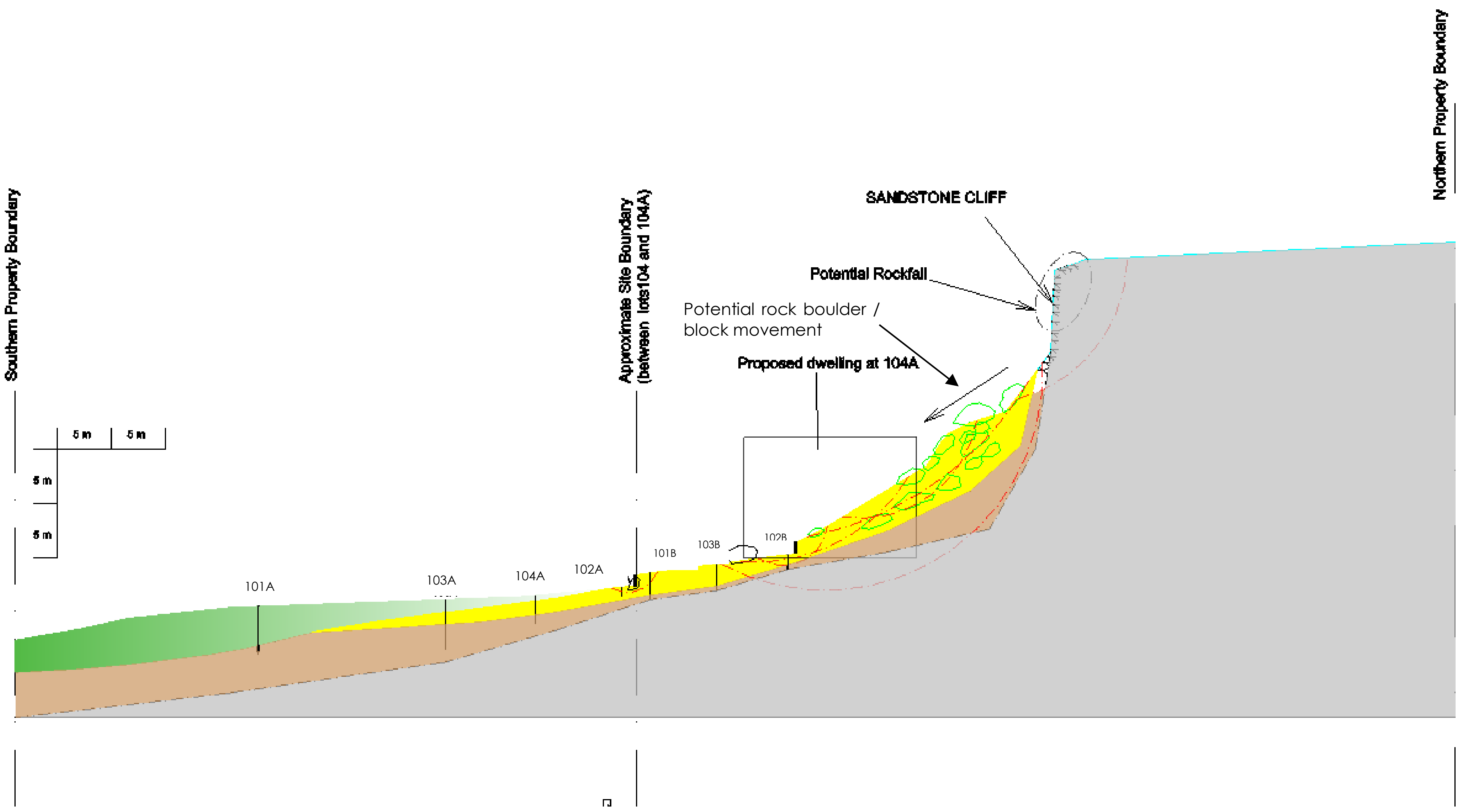


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**Key:**

- Indicative topsoil / alluvial soil profile (Units A & B)
- Indicative colluvial profile (Unit C)
- Indicative residual soil profile (Unit D)
- Indicative rock profile (Unit E)
- Indicative shallow and deep slide

| **Martens & Associates Pty Ltd** ABN 85 070 240 890 | | **Environment \| Water \| Wastewater \| Geotechnical \| Civil \| Management** | |
|---|---|---|---|
| Drawn: | HN | **INFERRED GEOLOGICAL CROSS SECTION SHOWING POSSIBLE INSTABILITY RISKS** **104A Wakehurst Parkway, Elanora Heights, NSW** | Drawing: **FIGURE 2** |
| Approved: | RE | | |
| Date: | 16.07.2018 | | |
| Scale: | NA | | Job No: P1705950JR02V02 |

# 7 Attachment B – Borehole Logs

| CLIENT | Aaron Kendall c/- Thodey Design | COMMENCED | 02/07/2018 | COMPLETED | 02/07/2018 | REF BH101A |
|---|---|---|---|---|---|---|
| PROJECT | Preliminary Geotechnical Investigation | LOGGED | HN | CHECKED | RE | Sheet 1 OF 1 |
| SITE | 104 Wakehurst Parkway, Elanora Heights, NSW | GEOLOGY | Newport Formation | VEGETATION | Grass | PROJECT NO. P1705950 |
| EQUIPMENT | 2WD ute-mounted hydraulic drill rig | EASTING | | RL SURFACE | 6 m | DATUM: AHD |
| EXCAVATION DIMENSIONS | Ø100 mm x 4.10 m depth | NORTHING | | ASPECT | South | SLOPE: <2% |

| METHOD | PENETRATION RESISTANCE | WATER | DEPTH (metres) | DEPTH / RL | SAMPLE OR FIELD TEST | RECOVERED | GRAPHIC LOG | USCS / ASCS CLASSIFICATION | SOIL/ROCK MATERIAL DESCRIPTION | MOISTURE CONDITION | CONSISTENCY DENSITY | STRUCTURE AND ADDITIONAL OBSERVATIONS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| AD/V | L | Not Encountered | | 6.00 | | | | ML | TOPSOIL: Sandy SILT; low plasticity; dark grey. | M | F | TOPSOIL |
| | | | | 0.15 / 5.85 | | | | SM | Silty SAND; fine to medium grained; dark grey. | | L | ALLUVIUM |
| | | | 0.5 | 0.50 / 5.50 | | | | | Pale grey and dark grey. | | VL | |
| | | | | 0.85 / 5.15 | | | | SP | SAND; medium grained; pale grey; trace silt. | | L | |
| | | | 1.0 | 1.10 / 4.90 | | | | | Brown and grey. | | | |
| | | | 1.5 | 1.60 / 4.40 | | | | | Brown. | | | |
| | | | 2.0 | | | | | | | | | |
| | | | 2.5 | 2.50 / 3.50 | | | | | Trace clay. | | | |
| | | | 3.0 | | | | | | | | | |
| | L-M | | 3.5 | 3.50 / 2.50 | | | | SC | Clayey SAND; fine to medium grained; brown. | | MD | RESIDUAL SOIL |
| | | | 4.0 | 4.10 | | | | | | | | |
| | | | 4.5 | | | | | | Hole Terminated at 4.10 m (Target depth reached) | | | |

EXCAVATION LOG TO BE READ IN CONJUCTION WITH ACCOMPANYING REPORT NOTES AND ABBREVIATIONS

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**Engineering Log - BOREHOLE**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| CLIENT | Aaron Kendall c/- Thodey Design | COMMENCED | 02/07/2018 | COMPLETED | 02/07/2018 | **REF BH102A** | |
| PROJECT | Preliminary Geotechnical Investigation | LOGGED | HN | CHECKED | RE | Sheet 1 OF 1 | |
| SITE | 104 Wakehurst Parkway, Elanora Heights, NSW | GEOLOGY | Newport Formation | VEGETATION | Grass | PROJECT NO. P1705950 | |
| EQUIPMENT | Pushtube | EASTING | | RL SURFACE | 8 m | DATUM | AHD |
| EXCAVATION DIMENSIONS | Ø50 mm x 0.90 m depth | NORTHING | | ASPECT | South | SLOPE | <10% |

| Drilling | | | | | Sampling | | Field Material Description | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| METHOD | PENETRATION RESISTANCE | WATER | DEPTH (metres) | DEPTH RL | SAMPLE OR FIELD TEST | RECOVERED | GRAPHIC LOG | USCS / ASCS CLASSIFICATION | SOIL/ROCK MATERIAL DESCRIPTION | MOISTURE CONDITION | CONSISTENCY DENSITY | STRUCTURE AND ADDITIONAL OBSERVATIONS |
| PT | L | Not Encountered | | 8.00<br>0.10 | | | | ML | TOPSOIL: Sandy SILT; low plasticity; dark brown; trace fine sandstone gravels. | M (<PL) | S - F | TOPSOIL |
| PT | L-M | Not Encountered | | 7.90<br>0.50 | | | | SC | Clayey SAND; coarse grained; light brown and pale grey; with fine to coarse sandstone gravels; trace silt. | D / M | L and MD | COLLUVIUM |
| PT | L<br>H | Not Encountered | 0.5 | 7.50<br>0.90 | | | | SM | Silty SAND; medium to coarse grained; dark grey; trace clay; trace sandstone gravels. | M | VL and L | |
| | | | 1.0 | | | | | | Hole Terminated at 0.90 m | | | 0.90: Push tube refusal on inferred cobbles/boulders within colluvial profile. |
| | | | 1.5 | | | | | | | | | |
| | | | 2.0 | | | | | | | | | |
| | | | 2.5 | | | | | | | | | |
| | | | 3.0 | | | | | | | | | |
| | | | 3.5 | | | | | | | | | |
| | | | 4.0 | | | | | | | | | |
| | | | 4.5 | | | | | | | | | |

EXCAVATION LOG TO BE READ IN CONJUCTION WITH ACCOMPANYING REPORT NOTES AND ABBREVIATIONS



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***Engineering Log - BOREHOLE***

| CLIENT | Aaron Kendall c/- Thodey Design | COMMENCED | 02/07/2018 | COMPLETED | 02/07/2018 | REF BH101B |
|---|---|---|---|---|---|---|
| PROJECT | Preliminary Geotechnical Investigation | LOGGED | HN | CHECKED | RE | Sheet 1 OF 1 |
| SITE | 104A Wakehurst Parkway, Elanora Heights, NSW | GEOLOGY | Hawkesbury Sandstone | VEGETATION | Grass | PROJECT NO. P1705950 |

| EQUIPMENT | Push tube | EASTING | | RL SURFACE | 9.1 m | DATUM | AHD |
|---|---|---|---|---|---|---|---|
| EXCAVATION DIMENSIONS | Ø50 mm x 1.50 m depth | NORTHING | | ASPECT | South | SLOPE | <2% |

| Drilling | | | | | Sampling | | Field Material Description | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| METHOD | PENETRATION RESISTANCE | WATER | DEPTH (metres) | DEPTH RL | SAMPLE OR FIELD TEST | RECOVERED | GRAPHIC LOG | USCS / ASCS CLASSIFICATION | SOIL/ROCK MATERIAL DESCRIPTION | MOISTURE CONDITION | CONSISTENCY DENSITY | STRUCTURE AND ADDITIONAL OBSERVATIONS |
| PT | M-H | Not Encountered | | 9.10 | | | | SM-SC | Silty Clayey SAND; fine to medium grained; brown and dark grey; with fine to coarse sandstone gravels. | D / M | L | COLLUVIUM |
| | | | 0.5 | 0.50<br>8.60 | | | | | 0.5m-0.6m: light brown. | | | |
| | | | | 0.85<br>8.25 | | | | | 0.85m-0.90m: brown and red brown. | | MD and D | |
| | | | 1.0 | 1.00<br>8.10<br>1.05<br>8.05 | | | | | 1.0m-1.05m: red brown.<br>1.05m-1.50m: dark grey. | | | |
| | | | 1.5 | 1.50 | | | | | Hole Terminated at 1.50 m | | | |
| | | | 2.0 | | | | | | | | | |
| | | | 2.5 | | | | | | | | | |
| | | | 3.0 | | | | | | | | | |
| | | | 3.5 | | | | | | | | | |
| | | | 4.0 | | | | | | | | | |
| | | | 4.5 | | | | | | | | | |

EXCAVATION LOG TO BE READ IN CONJUCTION WITH ACCOMPANYING REPORT NOTES AND ABBREVIATIONS



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**Engineering Log - BOREHOLE**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| CLIENT | Aaron Kendall c/- Thodey Design | COMMENCED | 02/07/2018 | COMPLETED | 02/07/2018 | REF BH102B | |
| PROJECT | Preliminary Geotechnical Investigation | LOGGED | HN | CHECKED | RE | Sheet 1 OF 1 | |
| SITE | 104A Wakehurst Parkway, Elanora Heights, NSW | GEOLOGY | Hawkesbury Sandstone | VEGETATION | Grass | PROJECT NO. P1705950 | |
| EQUIPMENT | Push tube | EASTING | | RL SURFACE | 11.5 m | DATUM | AHD |
| EXCAVATION DIMENSIONS | Ø50 mm x 1.35 m depth | NORTHING | | ASPECT | South | SLOPE | <5% |

| Drilling | | | | | Sampling | | Field Material Description | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| METHOD | PENETRATION RESISTANCE | WATER | DEPTH (metres) | DEPTH RL | SAMPLE OR FIELD TEST | RECOVERED | GRAPHIC LOG | USCS / ASCS CLASSIFICATION | SOIL/ROCK MATERIAL DESCRIPTION | MOISTURE CONDITION | CONSISTENCY DENSITY | STRUCTURE AND ADDITIONAL OBSERVATIONS |
| PT | L | Not Encountered | 0.5, 1.0 | 11.50 | | | | SM-SC | Silty Clayey SAND; fine to medium grained; dark brown and dark grey; with fine to coarse sandstone and ironstone gravels. | M | L / MD | COLLUVIUM |
| | M | | | 1.10 / 10.40 | | | | SC | Clayey SAND; fine to medium grained; light brown; trace sandstone gravels, inferred medium dense and dense. | | MD and D | RESIDUAL SOIL |
| | | | 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5 | 1.35 | | | | | Hole Terminated at 1.35 m | | | 1.35: Push tube refusal on inferred very low to low strength sandstone. |

EXCAVATION LOG TO BE READ IN CONJUCTION WITH ACCOMPANYING REPORT NOTES AND ABBREVIATIONS

# 8 Attachment C – DCP 'N' Counts

# Dynamic Cone Penetrometer Test Log Summary

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| | | | |
|---|---|---|---|
| **Site** | 104 Wakehurst Parkway, Elanora Heights, NSW | **DCP Group Reference** | P1705950JS01V01 |
| **Client** | Aaron Kendall | **Log Date** | 02.07.2018 |
| **Logged by** | HN | | |
| **Checked by** | RE | | |
| **Comments** | DCP's commenced 50 mm below ground level. | | |

**TEST DATA**

| Depth Interval (m) | DCP 101A | DCP 102A | DCP 103A | DCP 104A | | | | |
|---|---|---|---|---|---|---|---|---|
| **0.15** | 3 | 3 | 1 | 2 | | | | |
| **0.30** | 4 | 8 | 1 | 1 | | | | |
| **0.45** | 1 | 3 | 1 | 2 | | | | |
| **0.60** | 1 | 2 | 1 | 2 | | | | |
| **0.75** | 1 | 1 | 1 | 3 | | | | |
| **0.90** | 3 | 15 / 100 mm | 2 | 3 | | | | |
| **1.05** | 2 | Bounce at 0.9 m | 2 | 2 | | | | |
| **1.20** | 2 | | 13 | 3 | | | | |
| **1.35** | 2 | | 15 | 13 | | | | |
| **1.50** | 4 | | 7 | 15 | | | | |
| **1.65** | 2 | | 8 | 7 | | | | |
| **1.80** | 2 | | 8 | 3 | | | | |
| **1.95** | 2 | | 4 | 4 | | | | |
| **2.10** | 2 | | 3 | 12 | | | | |
| **2.25** | 2 | | 2 | 25 | | | | |
| **2.40** | 2 | | 1 | 34 | | | | |
| **2.55** | 3 | | 1 | Terminated at 2.45 m due to high 'N' counts | | | | |
| **2.70** | 3 | | 3 | | | | | |
| **2.85** | 3 | | 3 | | | | | |
| **3.00** | 4 | | 9 | | | | | |
| **3.15** | 5 | | 10 | | | | | |
| **3.30** | 5 | | 10 | | | | | |
| **3.45** | 7 | | 11 | | | | | |
| **3.60** | 13 | | 10 | | | | | |
| **3.75** | 17 | | 7 | | | | | |
| **3.90** | 14 | | 10 | | | | | |
| **4.05** | 11 | | 15 | | | | | |
| **4.20** | 12 | | 15 | | | | | |
| **4.35** | 19 | | 18 | | | | | |
| **4.50** | Terminated at 4.40 m (target depth reached) | | Terminated at 4.40 m (target depth reached) | | | | | |
| **4.65** | | | | | | | | |
| **4.80** | | | | | | | | |
| **4.95** | | | | | | | | |
| **5.10** | | | | | | | | |

# Dynamic Cone Penetrometer Test Log Summary

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| | | | |
|---|---|---|---|
| **Site** | 104A Wakehurst Parkway, Elanora Heights, NSW | **DCP Group Reference** | P1705950JS02V01 |
| **Client** | Aaron Kendall | **Log Date** | 02.07.2018 |
| **Logged by** | HN | | |
| **Checked by** | RE | | |
| **Comments** | DCP's commenced 50 mm below ground level. | | |

**TEST DATA**

| Depth Interval (m) | DCP 101B | DCP 102B | DCP 103B | | | | | |
|---|---|---|---|---|---|---|---|---|
| **0.15** | 3 | 3 | 1 | | | | | |
| **0.30** | 16 | 2 | 3 | | | | | |
| **0.45** | 20 | 8 | 3 | | | | | |
| **0.60** | 17 | 8 | 7 | | | | | |
| **0.75** | 14 | 14 | 3 | | | | | |
| **0.90** | 9 | 11 | 4 | | | | | |
| **1.05** | 11 | 10 / 50 mm | 5 | | | | | |
| **1.20** | 20 | Bounce at 1.0 m | 4 | | | | | |
| **1.35** | 10 | | 4 | | | | | |
| **1.50** | 9 | | 6 | | | | | |
| **1.65** | 11 | | 8 | | | | | |
| **1.80** | 9 | | 8 | | | | | |
| **1.95** | 7 | | 7 | | | | | |
| **2.10** | 13 | | 13 | | | | | |
| **2.25** | 11 | | 31 | | | | | |
| **2.40** | 18 | | 20 | | | | | |
| **2.55** | Bounce at 2.45 m | | Bounce at 2.45 m | | | | | |
| **2.70** | | | | | | | | |
| **2.85** | | | | | | | | |
| **3.00** | | | | | | | | |

# 9 Attachment D – Geotechnical Risk Calculation Sheet

# Slope Instability Risk - Summary Assessment

Method based on Walker *et al.* in AGS Vol 42 No. 1 March 2007

Method ST-38 Revised 08.07.09

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## PROJECT DETAILS

| | | | |
|---|---|---|---|
| Project | 104 Wakehurst Parkway, Elanora Heights, NSW | Ref. No. | P1705950JS03V01 |
| Author | HN | Reviewed | RE |
| Date Created | 11.07.2018 | | |

## RISK ASSESSMEENT

| Risk | Identify Hazard | | | Description | Likelihood [1] | Risk to Life [1] Probability | Risk to Life [1] Assessment | Risk to Property [1] Consequence | Risk to Property [1] Assessment |
|---|---|---|---|---|---|---|---|---|---|
| A | Shallow rotational slide | Possible | Insignificant | Shallow rotational slide | Possible | 7.90E-07 | Lr-A | Insignificant | VL |
| B | Deep seated rotational slide | Possible | Insignificant | Deep seated rotational slide | Possible | 6.50E-07 | Lr-A | Insignificant | VL |
| C | Rock fall | Likely | Insignificant | Rock fall | Likely | 9.50E-07 | Lr-A | Insignificant | L |
| | | | | | | | | | |

**Notes**

1. Assumes tretment measures are adopted.

**Definitions**

1. Risk to Life Assessment - Lr-A: Acceptable risk for loss of life for the person(s). Risk level suitable for new developments.
2. Risk to Life Assessment - Lr-T: Tolerable risk for loss of life for the person(s). Risk level suitable for existing structures > 10 years old. Risk level unsuitable for new developments.
3. Risk to Life Assessment - Lr-U: Unacceptable risk for loss of life for the person(s). Risk level unsuitable for new or existing (>10 years old) developments.

**Risk Level Implications**

1. VH - Very High Risk - Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce to Low. Cost could be prohibitive.
2. H - High Risk - Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Treatment will be costly.
3. M - Moderate Risk - May be tolerated in certain circumstances but requires investigation, planning and implementation to reduce risk to Low. Treatment options are practical.
4. L - Low Risk - Usually acceptable to regulators. Where treatment has been requir3ed to reduce the risk to this level, ongoing maintenance is required.
5. VL - Very Low Risk - Acceptable. Manage by normal slope maintenance procedures.

**Treatment Measures**

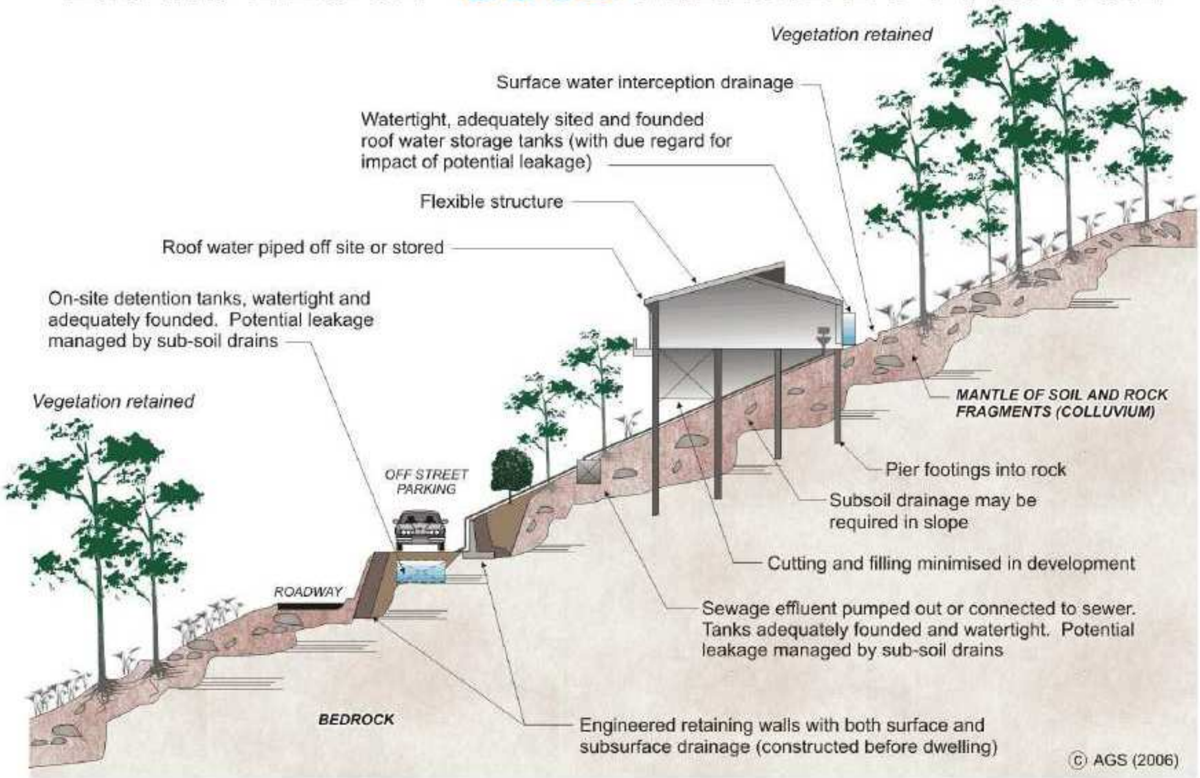
Ensure good hill slope engineering practice is adopted (examples are provided in Attachment E). Maintain vegetation cover. Do not over-steepen existing grades without suitable shoring support. Do not place excessive load onto reaining walls or existing and final sloping surfaces unless designed for. Ensure appropriate foundation and footing design. Provide / maintain appropriate surface and sub-surface drainage. Refer report text for further recommendations.

# 10 Attachment E – Hillside Construction Guidelines (AGS, 2007)

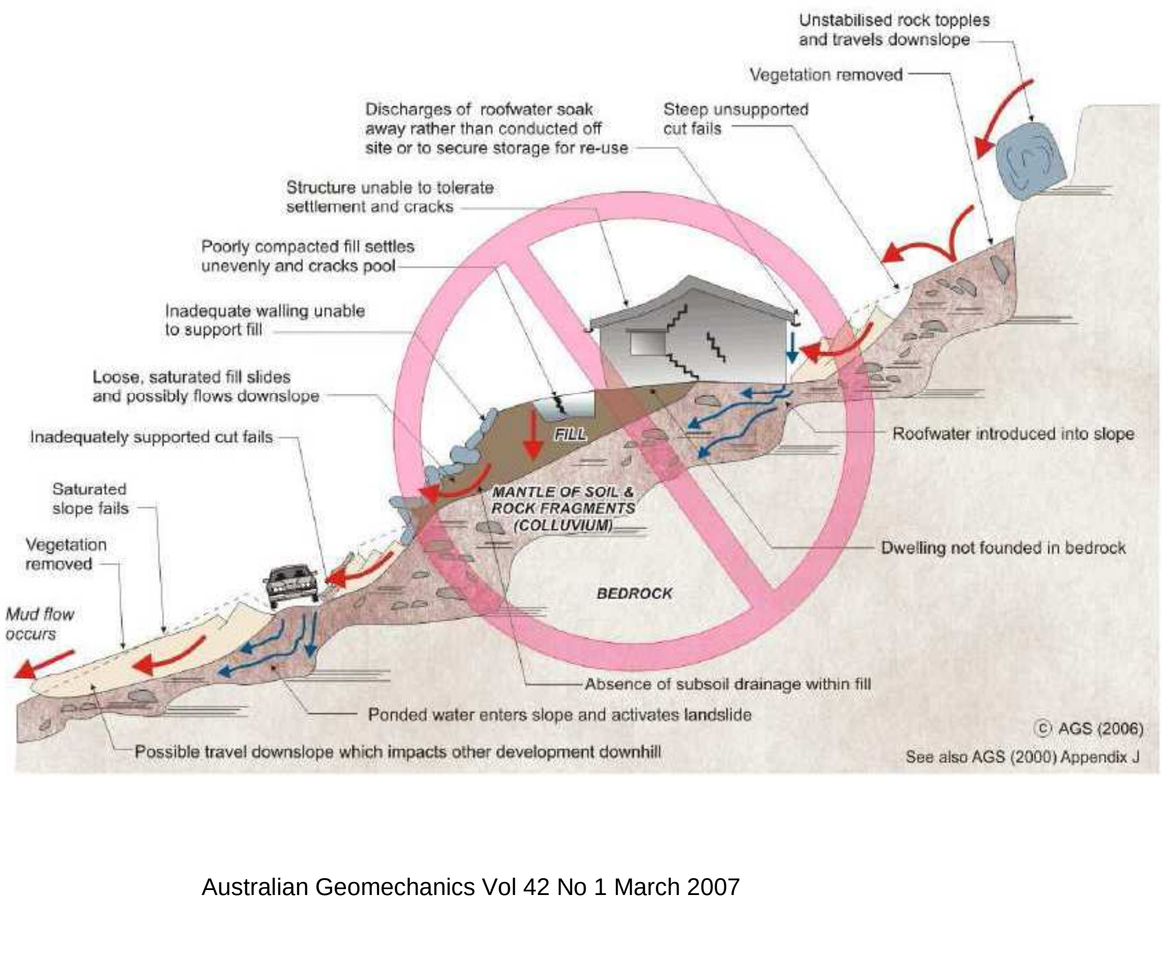
# APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

# EXAMPLES OF GOOD HILLSIDE PRACTICE




# EXAMPLES OF POOR HILLSIDE PRACTICE



# 11 Attachment F – General Geotechnical Recommendations

# Geotechnical Recommendations



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*These general geotechnical recommendations have been prepared by Martens to help you deliver a safe work site, to comply with your obligations, and to deliver your project. Not all are necessarily relevant to this report but are included as general reference. Any specific recommendations made in the report will override these recommendations.*

**Batter Slopes**

Excavations in soil and extremely low to very low strength rock exceeding 0.75 m depth should be battered back at grades of no greater than 1 Vertical (V) : 2 Horizontal (H) for temporary slopes (unsupported for less than 1 month) and 1 V : 3 H for longer term unsupported slopes.

Vertical excavation may be carried out in medium or higher strength rock, where encountered, subject to inspection and confirmation by a geotechnical engineer. Long term and short term unsupported batters should be protected against erosion and rock weathering due to, for example, stormwater run-off.

Batter angles may need to be revised depending on the presence of bedding partings or adversely oriented joints in the exposed rock, and are subject to on-site inspection and confirmation by a geotechnical engineer. Unsupported excavations deeper than 1.0 m should be assessed by a geotechnical engineer for slope instability risk.

Any excavated rock faces should be inspected during construction by a geotechnical engineer to determine whether any additional support, such as rock bolts or shotcrete, is required.

**Earthworks**

Earthworks should be carried out following removal of any unsuitable materials and in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect the condition of prepared surfaces to assess suitability as foundation for future fill placement or load application.

Earthworks inspections and compliance testing should be carried out in accordance with Sections 5 and 8 of AS3798 (2007), with testing to be carried out by a National Association of Testing Authorities (NATA) accredited testing laboratory.

**Excavations**

All excavation work should be completed with reference to the *Work Health and Safety (Excavation Work) Code of Practice (2015)*, by Safe Work Australia. Excavations into rock may be undertaken as follows:

1. Extremely low to low strength rock - conventional hydraulic earthmoving equipment.

2. Medium strength or stronger rock - hydraulic earthmoving equipment with rock hammer or ripping tyne attachment.

Exposed rock faces and loose boulders should be monitored to assess risk of block / boulder movement, particularly as a result of excavation vibrations.

**Fill**

Subject to any specific recommendations provided in this report, any fill imported to site is to comprise approved material with maximum particle size of two thirds the final layer thickness. Fill should be placed in horizontal layers of not more than 300 mm loose thickness, however, the layer thickness should be appropriate for the adopted compaction plant.

**Foundations**

All exposed foundations should be inspected by a geotechnical engineer prior to footing construction to confirm encountered conditions satisfy design assumptions and that the base of all excavations is free from loose or softened material and water. Water that has ponded in the base of excavations and any resultant softened material is to be removed prior to footing construction.

Footings should be constructed with minimal delay following excavation. If a delay in construction is anticipated, we recommend placing a concrete blinding layer of at least 50 mm thickness in shallow footings or mass concrete in piers / piles to protect exposed foundations.

A geotechnical engineer should confirm any design bearing capacity values, by further assessment during construction, as necessary.

**Shoring - Anchors**

Where there is a requirement for either soil or rock anchors, or soil nailing, and these structures penetrate past a property boundary, appropriate permission from the adjoining land owner must be obtained prior to the installation of these structures.

**Shoring - Permanent**

Permanent shoring techniques may be used as an alternative to temporary shoring. The design of such structures should be in accordance with the findings of this report and any further testing recommended by this report. Permanent shoring may include [but not be limited to] reinforced block work walls, contiguous and semi contiguous pile walls, secant pile walls and soldier pile walls with or without reinforced shotcrete infill panels. The choice of shoring system will depend on the type of structure, project budget and site specific geotechnical conditions.

Permanent shoring systems are to be engineer designed and backfilled with suitable granular material and free-draining drainage material. Backfill should be placed in maximum 100 mm thick layers compacted using a hand operated compactor. Care should be taken to ensure excessive compaction stresses are not transferred to retaining walls.

Shoring design should consider any surcharge loading from sloping / raised ground behind shoring structures, live loads, new structures, construction equipment, backfill compaction and static water pressures. All shoring systems shall be provided with adequate foundation designs.

Suitable drainage measures, such as geotextile enclosed 100 mm agricultural pipes embedded in free-draining gravel, should be included to redirect water that may collect behind the shoring structure to a suitable discharge point.

#### Shoring - Temporary

In the absence of providing acceptable excavation batters, excavations should be supported by suitably designed and installed temporary shoring / retaining structures to limit lateral deflection of excavation faces and associated ground surface settlements.

#### Soil Erosion Control

Removal of any soil overburden should be performed in a manner that reduces the risk of sedimentation occurring in any formal stormwater drainage system, on neighbouring land and in receiving waters. Where possible, this may be achieved by one or more of the following means:

1. Maintain vegetation where possible
2. Disturb minimal areas during excavation
3. Revegetate disturbed areas if possible

All spoil on site should be properly controlled by erosion control measures to prevent transportation of sediments off-site. Appropriate soil erosion control methods in accordance with Landcom (2004) shall be required.

#### Trafficability and Access

Consideration should be given to the impact of the proposed works and site subsurface conditions on trafficability within the site e.g. wet clay soils will lead to poor trafficability by tyred plant or vehicles.

Where site access is likely to be affected by any site works, construction staging should be organised such that any impacts on adequate access are minimised as best as possible.

#### Vibration Management

Where excavation is to be extended into medium or higher strength rock, care will be required when using a rock hammer to limit potential structural distress from excavation-induced vibrations where nearby structures may be affected by the works.

To limit vibrations, we recommend limiting rock hammer size and set frequency, and setting the hammer parallel to bedding planes and along defect planes, where possible, or as advised by a geotechnical engineer. We recommend limiting vibration peak particle velocities (PPV) caused by construction equipment or resulting from excavation at the site to 5 mm/s (AS 2187.2, 2006, Appendix J).

#### Waste – Spoil and Water

Soil to be disposed off-site should be classified in accordance with the relevant State Authority guidelines and requirements.

Any collected waste stormwater or groundwater should also be tested prior to discharge to ensure contaminant levels (where applicable) are appropriate for the nominated discharge location.

MA can complete the necessary classification and testing if required. Time allowance should be made for such testing in the construction program.

#### Water Management - Groundwater

If the proposed works are likely to intersect ephemeral or permanent groundwater levels, the management of any potential acid soil drainage should be considered. If groundwater tables are likely to be lowered, this should be further discussed with the relevant State Government Agency.

#### Water Management – Surface Water

All surface runoff should be diverted away from excavation areas during construction works and prevented from accumulating in areas surrounding any retaining structures, footings or the base of excavations.

Any collected surface water should be discharged into a suitable Council approved drainage system and not adversely impact downslope surface and subsurface conditions.

All site discharges should be passed through a filter material prior to release. Sump and pump methods will generally be suitable for collection and removal of accumulated surface water within any excavations.

#### Contingency Plan

In the event that proposed development works cause an adverse impact on geotechnical hazards, overall site stability or adjacent properties, the following actions are to be undertaken:

1. Works shall cease immediately.
2. The nature of the impact shall be documented and the reason(s) for the adverse impact investigated.
3. A qualified geotechnical engineer should be consulted to provide further advice in relation to the issue.

# 12 Attachment G – Geotechnical Risk Management Policy for Pittwater – Forms 1 and 1a

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

**Development Application for** Aaron Kendall
Name of Applicant
**Address of site** 104A Wakehurst Parkway, Elanora heights NSW

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

*I,* Kenneth Burgess *on behalf of* Martens and Associates
(Insert Name) (Trading or Company Name)

on this the 13/6/2023 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $10million.

I:
**Please mark appropriate box**

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

**Geotechnical Report Details:**

Report Title: Preliminary Geotechnical Assessment
Report Date: June 2023
Author: Hamed Naghibi
Author's Company/Organisation: Martens and Associates

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

Thodey Design (2023) Architectural Drawings Rev M
dated 11/4/2023

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

Signature ......KBurgess......
Name ...Kenneth Burgess...
Chartered Professional Status....CPEng....
Membership No. ....3789174....
Company...Martens and Associates

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

**Development Application for** Aaron Kendall
Name of Applicant
**Address of site** 104 A Wakehurst Parkway, Elanora Heights NSW

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

**Geotechnical Report Details:**

Report Title: Preliminary Geotechnical Investigation
Report Date: June 2023
Author: Hamed Naghibi
**Author's Company/Organisation:** Martens and Associates

**Please mark appropriate box**

- ✓ Comprehensive site mapping conducted 2 July 2018 (date)
- ✓ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ✓ Subsurface investigation required
  - No Justification ......
  - ✓ Yes Date conducted 2 July 2018
- ✓ Geotechnical model developed and reported as an inferred subsurface type-section
- ✓ Geotechnical hazards identified
  - ✓ Above the site
  - ✓ On the site
  - ✓ Below the site
  - ✓ Beside the site
- ✓ Geotechnical hazards described and reported
- ✓ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ✓ Consequence analysis
  - ✓ Frequency analysis
- ✓ Risk calculation
- ✓ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ✓ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ✓ Design Life Adopted:
  - ✓ 100 years
  - Other ...... specify
- ✓ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ✓ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ✓ Risk assessment within Bushfire Asset Protection Zone.

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

Signature ...... KBurgess
Name Kenneth Burgess
Chartered Professional Status CPEng
Membership No. 3789174
Company Martens and Associates

# 13 Attachment H – Notes About This Report

## Important Information About Your Report (1 of 2)

*These notes have been prepared by Martens to help you interpret and understand the limitations of your report. Not all are necessarily relevant to all reports but are included as general reference.*

**Engineering Reports - Limitations**

The recommendations presented in this report are based on limited investigations and include specific issues to be addressed during various phases of the project. If the recommendations presented in this report are not implemented in full, the general recommendations may become inapplicable and Martens & Associates accept no responsibility whatsoever for the performance of the works undertaken.

Occasionally, sub-surface conditions between and below the completed boreholes or other tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact Martens & Associates.

Relative ground surface levels at borehole locations may not be accurate and should be verified by on-site survey.

**Engineering Reports – Project Specific Criteria**

Engineering reports are prepared by qualified personnel. They are based on information obtained, on current engineering standards of interpretation and analysis, and on the basis of your unique project specific requirements as understood by Martens. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the Client.

Where the report has been prepared for a specific design proposal (e.g. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (e.g. to a twenty storey building). Your report should not be relied upon, if there are changes to the project, without first asking Martens to assess how factors, which changed subsequent to the date of the report, affect the report's recommendations. Martens will not accept responsibility for problems that may occur due to design changes, if not consulted.

**Engineering Reports – Recommendations**

Your report is based on the assumption that site conditions, as may be revealed through selective point sampling, are indicative of actual conditions throughout an area. This assumption often cannot be substantiated until project implementation has commenced. Therefore your site investigation report recommendations should only be regarded as preliminary.

Only Martens, who prepared the report, are fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report, there is a risk that the report will be misinterpreted and Martens cannot be held responsible for such misinterpretation.

**Engineering Reports – Use for Tendering Purposes**

Where information obtained from investigations is provided for tendering purposes, Martens recommend 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.

Martens would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

**Engineering Reports – Data**

The report as a whole presents the findings of a site assessment and should not be copied in part or altered in any way.

Logs, figures, drawings *etc* are customarily included in a Martens report and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel), desktop studies and laboratory evaluation of field samples. These data should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

**Engineering Reports – Other Projects**

To avoid misuse of the information contained in your report it is recommended that you confer with Martens before passing your report on to another party who may not be familiar with the background and purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

**Subsurface Conditions - General**

Every care is taken with the report in relation to interpretation of subsurface conditions, discussion of geotechnical aspects, relevant standards and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions - the potential will depend partly on test point (eg. excavation or borehole) spacing and sampling frequency, which are often limited by project imposed budgetary constraints.

- Changes in guidelines, standards and policy or interpretation of guidelines, standards and policy by statutory authorities.
- The actions of contractors responding to commercial pressures.
- Actual conditions differing somewhat from those inferred to exist, because no professional, no matter how qualified, can reveal precisely what is hidden by earth, rock and time.

  The actual interface between logged materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions.

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

**Subsurface Conditions - Changes**

Natural processes and the activity of man create subsurface conditions. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Reports are based on conditions which existed at the time of the subsurface exploration / assessment.

Decisions should not be based on a report whose adequacy may have been affected by time. If an extended period of time has elapsed since the report was prepared, consult Martens to be advised how time may have impacted on the project.

**Subsurface Conditions - Site Anomalies**

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

**Report Use by Other Design Professionals**

To avoid potentially costly misinterpretations when other design professionals develop their plans based on a Martens report, retain Martens to work with other project professionals affected by the report. This may involve Martens explaining the report design implications and then reviewing plans and specifications produced to see how they have incorporated the report findings.

**Subsurface Conditions – Geo-environmental Issues**

Your report generally does not relate to any findings, conclusions, or recommendations about the potential for hazardous or contaminated materials existing at the site unless specifically required to do so as part of Martens' proposal for works.

Specific sampling guidelines and specialist equipment, techniques and personnel are typically used to perform geo-environmental or site contamination assessments. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Martens for information relating to such matters.

**Responsibility**

Geo-environmental reporting relies on interpretation of factual information based on professional judgment and opinion and has an inherent level of uncertainty attached to it and is typically far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded.

To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Martens to other parties but are included to identify where Martens' responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Martens closely and do not hesitate to ask any questions you may have.

**Site Inspections**

Martens will always be pleased to provide engineering inspection services for aspects of work to which this report relates. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site. Martens is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction.

# Soil Data



## Definitions

In engineering terms, soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material does not exhibit any visible rock properties and can be remoulded or disintegrated by hand in its field condition or in water, it is described as a soil. Other materials are described using rock description terms.

The methods of description and classification of soils and rocks used in this report are typically based on Australian Standard 1726 and the Unified Soil Classification System (USCS) – refer Soil Data Explanation of Terms (2 of 3). In general, descriptions cover the following properties: strength or density, colour, moisture, structure, soil or rock type and inclusions.

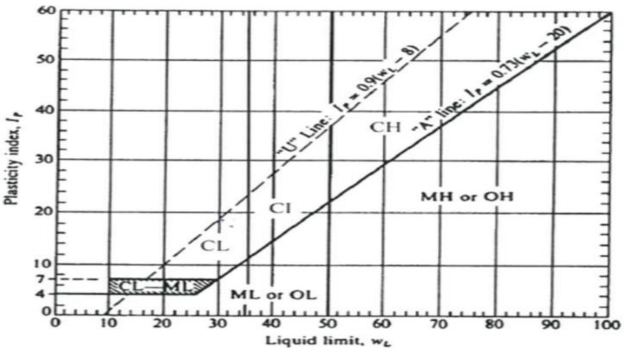
## Particle Size

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (e.g. sandy CLAY). Unless otherwise stated, particle size is described in accordance with the following table.

| Division | Subdivision | | Particle Size (mm) |
|---|---|---|---|
| Oversized | BOULDERS | | >200 |
| | COBBLES | | 63 to 200 |
| Coarse Grained Soil | GRAVEL | Coarse | 19 to 63 |
| | | Medium | 6.7 to 19 |
| | | Fine | 2.36 to 6.7 |
| | SAND | Coarse | 0.6 to 2.36 |
| | | Medium | 0.21 to 0.6 |
| | | Fine | 0.075 to 0.21 |
| Fine Grained Soil | SILT | | 0.002 to 0.075 |
| | CLAY | | < 0.002 |

## Plasticity Properties

Plasticity properties of cohesive soils can be assessed in the field by tactile properties or by laboratory procedures.




## Soil Moisture Condition

Coarse Grained (Granular) Soil:

| | |
|---|---|
| Dry (D): | Looks and feels dry. Cemented soils are hard, friable or powdery. Uncemented soils run freely through fingers. |
| Moist (M): | Feels cool and damp and is darkened in colour. Particles tend to cohere. |
| Wet (W): | As for moist but with free water forming on hands when handled. |

Fine Grained (Cohesive) Soil:

| | |
|---|---|
| Moist, dry of plastic limit[1] (w < PL): | Looks and feels dry. Hard, friable or powdery. |
| Moist, near plastic limit (w ≈ PL): | Can be moulded, feels cool and damp, is darkened in colour, at a moisture content approximately equal to the PL. |
| Moist, wet of plastic limit (w > PL): | Usually weakened and free water forms on hands when handled. |
| Wet, near liquid limit[2] (w ≈ LL) | |
| Wet, wet of liquid limit (w > LL) | |

[1] Plastic Limit (PL): Moisture content at which soil becomes too dry to be in a plastic condition.

[2] Liquid Limit (LL): Moisture content at which soil passes from plastic to liquid state.

## Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.
(Note: consistency is affected by soil moisture condition at time of measurement)

| Term | $C_u$ (kPa) | Field Guide |
|---|---|---|
| Very Soft (VS) | ≤12 | A finger can be pushed well into the soil with little effort. Sample exudes between fingers when squeezed in fist. |
| Soft (S) | >12 and ≤25 | A finger can be pushed into the soil to about 25mm depth. Easily moulded by light finger pressures. |
| Firm (F) | >25 and ≤50 | The soil can be indented about 5mm with the thumb, but not penetrated. Can be moulded by strong figure pressure. |
| Stiff (St) | >50 and ≤100 | The surface of the soil can be indented with the thumb, but not penetrated. Cannot be moulded by fingers. |
| Very Stiff (VSt) | >100 and ≤200 | The surface of the soil can be marked, but not indented with thumb pressure. Difficult to cut with a knife. Thumbnail can readily indent. |
| Hard (H) | > 200 | The surface of the soil can only be marked with the thumbnail. Brittle. Tends to break into fragments. |
| Friable (Fr) | - | Crumbles or powders when scraped by thumbnail. Can easily be crumbled or broken into small pieces by hand. |

## Density of Granular Soils

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

| Relative Density | % | SPT 'N' Value* (blows/300mm) | CPT Cone Value ($q_c$ MPa) |
|---|---|---|---|
| Very loose | ≤15 | < 5 | < 2 |
| Loose | >15 and ≤35 | 5 - 10 | 2 - 5 |
| Medium dense | >35 and ≤65 | 10 - 30 | 5 - 15 |
| Dense | >65 and ≤85 | 30 - 50 | 15 - 25 |
| Very dense | > 85 | > 50 | > 25 |

* Values may be subject to corrections for overburden pressures and equipment type and influenced by soil moisture condition at time of measurement.

## Minor Components

Minor components in soils may be present and readily detectable, but have little bearing on general geotechnical classification. Terms include:

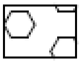
| Description of components | Proportion of component in: coarse grained soil | | | | Proportion of component in: fine grained soil | |
|---|---|---|---|---|---|---|
| | % Fines | Terminology | % Accessory coarse fraction | Terminology | % Sand/ gravel | Terminology |
| Minor | ≤5 | Trace clay / silt, as applicable | ≤15 | Trace sand / gravel, as applicable | ≤15 | Trace sand / gravel, as applicable |
| | >5,≤12 | With clay / silt, as applicable | >15,≤30 | With sand / gravel, as applicable | >5,≤30 | With sand / gravel, as applicable |
| Secondary | >12 | Prefix soil name as 'silty' or 'clayey', as applicable | >30 | Prefix soil name as 'sandy' or 'gravelly', as applicable | >30 | Prefix soil name as 'sandy' or 'gravelly', as applicable |

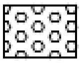
# Soil Data



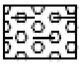
## Symbols for Soils and Other

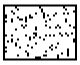
**SOILS**

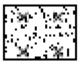
 COBBLES/BOULDERS

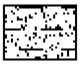
 GRAVEL (GP or GW)

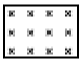
 Silty GRAVEL (GM)

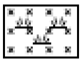
 Clayey GRAVEL (GC)

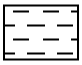
 SAND (SP or SW)

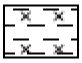
 Silty SAND (SM)

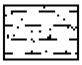
 Clayey SAND (SC)

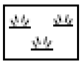
 SILT (ML or MH)

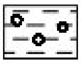
 ORGANIC SILT or CLAY (OH or OL)

 CLAY (CL, CI or CH)

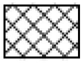
 Silty CLAY

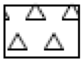
 Sandy CLAY

 PEAT (Pt)

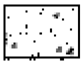
 Gravelly CLAY

**OTHER**

 FILL

 TALUS

 ASPHALT

 CONCRETE

 TOPSOIL

## Unified Soil Classification Scheme (USCS)

| **FIELD IDENTIFICATION PROCEDURES** (Excluding particles larger than 63 mm and basing fractions on estimated mass) | | | | | **USCS** | **Primary Name** |
|---|---|---|---|---|---|---|
| COARSE GRAINED SOILS More than 65 % of material less than 63 mm is larger than 0.075 mm | (A 0.075 mm particle is about the smallest particle visible to the naked eye) | GRAVELS More than half of coarse fraction is larger than 2.36 mm. | GRAVEL and GRAVEL-SAND Mixtures (≤ 5% fines) | Wide range in grain size and substantial amounts of all intermediate particle sizes; not enough fines to bind coarse grains; no dry strength | GW | GRAVEL |
| | | | | Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength | GP | GRAVEL |
| | | | GRAVEL-SILT and GRAVEL-SAND-SILT mixtures (≥12% fines) [1] | With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength; may also contain sand | GM | Silty GRAVEL |
| | | | | With excess plastic fines (for identification procedures see CL below); medium to high dry strength; may also contain sand | GC | Clayey GRAVEL |
| | | SANDS More than half of coarse fraction is smaller than 2.36 mm | SAND and GRAVEL-SAND mixtures (≤5% fines) | Wide range in grain sizes and substantial amounts of all intermediate sizes; not enough fines to bind coarse grains; no dry strength. | SW | SAND |
| | | | | Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength | SP | SAND |
| | | | SAND-SILT and SAND-CLAY mixtures (≥12% fines) [1] | With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength; | SM | Silty SAND |
| | | | | With excess plastic fines (for identification procedures see CL below); medium to high dry strength | SC | Clayey SAND |

| | | **IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM** | | | | | |
|---|---|---|---|---|---|---|---|
| | | **DRY STRENGTH (Crushing Characteristics)** | **DILATANCY** | **TOUGHNESS** | **DESCRIPTION** | **USCS** | **Primary Name** |
| FINE GRAINED SOILS More than 35 % of material less than 63 mm is smaller than 0.075 mm | (A 0.075 mm particle is about the smallest particle visible to the naked eye) | None to Low | Quick to Slow | Low | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or silt with low plasticity [2] | ML | SILT [3] |
| | | Medium to High | None to Slow | Medium | Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays | CL (or CI[4]) | CLAY |
| | | Low to Medium | Slow | Low | Organic slits and organic silty clays of low plasticity | OL | Organic SILT or CLAY |
| | | Low to Medium | None to Slow | Low to Medium | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | MH | SILT [3] |
| | | High to Very High | None | High | Inorganic clays of high plasticity, fat clays | CH | CLAY |
| | | Medium to High | None to Very Slow | Low to Medium | Organic clays of medium to high plasticity, organic silt of high plasticity | OH | Organic SILT or CLAY |
| HIGHLY ORGANIC SOILS | | Readily identified by colour, odour, spongy feel and frequently by fibrous texture | | | | Pt | PEAT |

**Notes:**

1. Between 5% and 12% - dual classification, e.g. GP-GM.
2. Low Plasticity Clay – Liquid Limit $W_L$ ≤35%; Medium Plasticity Clay – Liquid limit $W_L$ >35%, ≤50%; High Plasticity Clay - Liquid limit $W_L$ > 50%.
3. Low Plasticity Silt – Liquid Limit $W_L$ ≤50%; High Plasticity Silt - Liquid limit $W_L$ > 50%.
4. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.

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## Soil Agricultural Classification Scheme

In some situations, such as where soils are to be used for effluent disposal purposes, soils are often more appropriately classified in terms of traditional agricultural classification schemes. Where a Martens report provides agricultural classifications, these are undertaken in accordance with descriptions by Northcote, K.H. (1979) *The factual key for the recognition of Australian Soils*, Rellim Technical Publications, NSW, p 26 - 28.

| Symbol | Field Texture Grade | Behaviour of moist bolus | Ribbon length | Clay content (%) |
|---|---|---|---|---|
| S | Sand | Coherence nil to very slight; cannot be moulded; single grains adhere to fingers | 0 mm | < 5 |
| LS | Loamy sand | Slight coherence; discolours fingers with dark organic stain | 6.35 mm | 5 |
| CLS | Clayey sand | Slight coherence; sticky when wet; many sand grains stick to fingers; discolours fingers with clay stain | 6.35mm - 1.3cm | 5 - 10 |
| SL | Sandy loam | Bolus just coherent but very sandy to touch; dominant sand grains are of medium size and are readily visible | 1.3 - 2.5 | 10 - 15 |
| FSL | Fine sandy loam | Bolus coherent; fine sand can be felt and heard | 1.3 - 2.5 | 10 - 20 |
| $SCL^-$ | Light sandy clay loam | Bolus strongly coherent but sandy to touch, sand grains dominantly medium size and easily visible | 2.0 | 15 - 20 |
| L | Loam | Bolus coherent and rather spongy; smooth feel when manipulated but no obvious sandiness or silkiness; may be somewhat greasy to the touch if much organic matter present | 2.5 | 25 |
| Lfsy | Loam, fine sandy | Bolus coherent and slightly spongy; fine sand can be felt and heard when manipulated | 2.5 | 25 |
| SiL | Silt loam | Coherent bolus, very smooth to silky when manipulated | 2.5 | 25 + > 25 silt |
| SCL | Sandy clay loam | Strongly coherent bolus sandy to touch; medium size sand grains visible in a finer matrix | 2.5 - 3.8 | 20 - 30 |
| CL | Clay loam | Coherent plastic bolus; smooth to manipulate | 3.8 - 5.0 | 30 - 35 |
| SiCL | Silty clay loam | Coherent smooth bolus; plastic and silky to touch | 3.8 - 5.0 | 30- 35 + > 25 silt |
| FSCL | Fine sandy clay loam | Coherent bolus; fine sand can be felt and heard | 3.8 - 5.0 | 30 - 35 |
| SC | Sandy clay | Plastic bolus; fine to medium sized sands can be seen, felt or heard in a clayey matrix | 5.0 - 7.5 | 35 - 40 |
| SiC | Silty clay | Plastic bolus; smooth and silky | 5.0 - 7.5 | 35 - 40 + > 25 silt |
| LC | Light clay | Plastic bolus; smooth to touch; slight resistance to shearing | 5.0 - 7.5 | 35 - 40 |
| LMC | Light medium clay | Plastic bolus; smooth to touch, slightly greater resistance to shearing than LC | 7.5 | 40 - 45 |
| MC | Medium clay | Smooth plastic bolus, handles like plasticine and can be moulded into rods without fracture, some resistance to shearing | > 7.5 | 45 - 55 |
| HC | Heavy clay | Smooth plastic bolus; handles like stiff plasticine; can be moulded into rods without fracture; firm resistance to shearing | > 7.5 | > 50 |

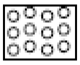
# Rock Data

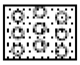


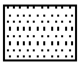
## Symbols for Rock

**SEDIMENTARY ROCK**

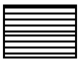
 BRECCIA

 CONGLOMERATE

 CONGLOMERATIC SANDSTONE

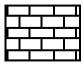
 SANDSTONE/QUARTZITE

 SILTSTONE

 MUDSTONE/CLAYSTONE

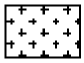
 SHALE

 COAL

 LIMESTONE

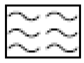
 LITHIC TUFF

**IGNEOUS ROCK**

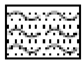
 GRANITE

 DOLERITE/BASALT

**METAMORPHIC ROCK**

 SLATE, PHYLLITE, SCHIST

 GNEISS

 METASANDSTONE

 METASILTSTONE

 METAMUDSTONE

## Definitions

Descriptive terms used for Rock by Martens are based on AS1726 and encompass rock substance, defects and mass.

*Rock Material* — The intact rock that is bounded by defects.

*Rock Defect* — Discontinuity, fracture, break or void in the material or minerals across which there is little or no tensile strength.

*Rock Structure* — The nature and configuration of the different defects within the rock mass and their relationship to each other.

*Rock Mass* — The entirety of the system formed by all of the rock material and all of the defects that are present.

## Degree of Weathering

Rock weathering is defined as the degree of decline in rock structure and grain property and can be determined in the field.

| Term | Symbol | Definition |
|---|---|---|
| Residual soil[1] | RS | Material is weathered to such an extent that it has soil properties. Mass structure, material texture, and fabric of original rock are no longer visible, but the soil has not been significantly transported. |
| Extremely weathered[1] | XW | Material is weathered to such an extent that it has soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System. Mass structure and material texture and fabric of original rock are still visible. |
| Highly weathered[2] | HW | The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the original colour of the 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[2] | MW | The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the rock is not recognisable. Rock strength shows little or no change 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 | Rock substance unaffected by weathering. No sign of decomposition of individual materials or colour changes. |

**Notes:**
1 RS and EW material is described using soil descriptive terms.
2. The term "Distinctly Weathered" (DW) may be used to cover the range of substance weathering between EW and SW

## Rock Strength

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the loading. The test procedure is described by the International Society of Rock Mechanics.

| Term (Strength) | $I_s$ (50) MPa | Uniaxial Compressive Strength MPa | Field Guide | Symbol |
|---|---|---|---|---|
| Very low | >0.03 ≤0.1 | 0.6 – 2 | May be crumbled in the hand. Sandstone is 'sugary' and friable. | VL |
| Low | >0.1 ≤0.3 | 2 – 6 | Core 150mm long x 50mm diameter may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling. | L |
| Medium | >0.3 ≤1.0 | 6 – 20 | Core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife. | M |
| High | >1 ≤3 | 20 – 60 | Core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife. Breaks with single blow from pick. | H |
| Very high | >3 ≤10 | 60 – 200 | Core 150mm long x 50mm diameter, broken readily with hand held hammer. Cannot be scratched with knife. Breaks after more than one pick strike. | VH |
| Extremely high | >10 | >200 | A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer. | EH |

# Rock Data



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## Degree of Fracturing

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude fractures such as drilling breaks (DB) or handling breaks (HB).

| Term | Description |
|---|---|
| Fragmented | The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than core diameter. |
| Highly fractured | Core lengths are generally less than 20 mm to 40 mm with occasional fragments. |
| Fractured | Core lengths are mainly 30 mm to 100 mm with occasional shorter and longer sections. |
| Slightly fractured | Core lengths are generally 300 mm to 1000 mm, with occasional longer sections and sections of 100 mm to 300 mm. |
| Unbroken | The core does not contain any fractures. |

## Rock Core Recovery

TCR = Total Core Recovery

$$= \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100\,\%$$

SCR = Solid Core Recovery

$$= \frac{\sum \text{Length of cylindrica l core recovered}}{\text{Length of core run}} \times 100\,\%$$

RQD = Rock Quality Designation

$$= \frac{\sum \text{Axial lengths of core} > 100\text{ mm long}}{\text{Length of core run}} \times 100\,\%$$

## Rock Strength Tests

▼ Point load strength Index (Is50) - axial test (MPa)

▶ Point load strength Index (Is50) - diametral test (MPa)

● Uniaxial compressive strength (UCS) (MPa)

## Defect Type Abbreviations and Descriptions

**Defect Type (with inclination given)**

| | |
|---|---|
| BP | Bedding plane parting |
| FL | Foliation |
| CL | Cleavage |
| JT | Joint |
| FC | Fracture |
| SZ/SS | Sheared zone/ seam (Fault) |
| CZ/CS | Crushed zone/ seam |
| DZ/DS | Decomposed zone/ seam |
| FZ | Fractured Zone |
| IS | Infilled seam |
| VN | Vein |
| CO | Contact |
| HB | Handling break |
| DB | Drilling break |

**Planarity**

| | |
|---|---|
| Pl | Planar |
| Cu | Curved |
| Un | Undulating |
| St | Stepped |
| Ir | Irregular |
| Dis | Discontinuous |

**Roughness**

| | |
|---|---|
| Pol | Polished |
| Sl | Slickensided |
| Sm | Smooth |
| Ro | Rough |
| VR | Very rough |

**Thickness**

| | |
|---|---|
| Zone | > 100 mm |
| Seam | > 2 mm < 100 mm |
| Plane | < 2 mm |

**Coating or Filling**

| | |
|---|---|
| Cn | Clean |
| Sn | Stain |
| Ct | Coating |
| Vnr | Veneer |
| Fe | Iron Oxide |
| X | Carbonaceous |
| Qz | Quartzite |
| MU | Unidentified mineral |

**Inclination**

Inclination of defect is measured from perpendicular to and down the core axis.
Direction of defect is measured clockwise (looking down core) from magnetic north.

## Sampling

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

Disturbed samples taken during drilling or excavation provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples may be taken by pushing a thin-walled sampling tube, e.g. $U_{50}$ (50 mm internal diameter thin walled tube), into soils and withdrawing a soil sample 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. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

## Drilling / Excavation Methods

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

Hand Excavation - in some situations, excavation using hand tools, such as mattock and spade, may be required due to limited site access or shallow soil profiles.

Hand Auger - the hole is advanced by pushing and rotating either a sand or clay auger, generally 75-100 mm in diameter, into the ground. The penetration depth is usually limited to the length of the auger pole; however extender pieces can be added to lengthen this.

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

Large Diameter Auger (e.g. Pengo) - the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling (Push Tube) - the hole is advanced by pushing a 50 - 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength *etc.* is only marginally affected.

Continuous Spiral Flight Augers - the hole is advanced using 90 - 115 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface or, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

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

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

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

## In-situ Testing and Interpretation

Cone Penetrometer Testing (CPT)

Cone penetrometer testing (sometimes referred to as Dutch Cone) described in this report has been carried out using an electrical friction cone penetrometer.

The test is described in AS 1289.6.5.1-1999 (R2013). In the test, a 35 mm diameter rod with a cone tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system.

Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the push rod centre to an amplifier and recorder unit mounted on the control truck. As penetration occurs (at a rate of approximately 20 mm per second) the information is output on continuous chart recorders. The plotted results given in this report have been traced from the original records. The information provided on the charts comprises:

(i) Cone resistance ($q_c$) - the actual end bearing force divided by the cross sectional area of the cone, expressed in MPa.

(ii) Sleeve friction ($q_f$) - the frictional force of the sleeve divided by the surface area, expressed in kPa.

(iii) Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

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

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

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

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

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

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

# Test, Drill and Excavation Methods



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

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

Standard Penetration Testing (SPT)

Standard penetration tests are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample.

The test procedure is described in AS 1289.6.3.1-2004. 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 penetration depth increments and the 'N' value is taken as the number of blows for the last two 150 mm depth increments (300 mm total penetration). 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:

(i) Where full 450 mm penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7 blows:

as 4, 6, 7
N = 13

(ii) Where the test is discontinued, short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm

as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

Dynamic Cone (Hand) Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods. Two relatively similar tests are used.

**Perth sand penetrometer (PSP)** - a 16 mm diameter flat ended rod is driven with a 9 kg hammer, dropping 600 mm. The test, described in AS 1289.6.3.3-1997 (R2013), was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

**Cone penetrometer (DCP)** - sometimes known as the Scala Penetrometer, a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm. The test, described in AS 1289.6.3.2-1997 (R2013), was developed initially for pavement sub-grade investigations, with correlations of the test results with California Bearing Ratio published by various Road Authorities.

Pocket Penetrometers

The pocket (hand) penetrometer (PP) is typically a light weight spring hand operated device with a stainless steel loading piston, used to estimate unconfined compressive strength, $q_u$, (UCS in kPa) of a fine grained soil in field conditions. In use, the free end of the piston is pressed into the soil at a uniform penetration rate until a line, engraved near the piston tip, reaches the soil surface level. The reading is taken from a gradation scale, which is attached to the piston via a built-in spring mechanism and calibrated to kilograms per square centimetre (kPa) UCS. The UCS measurements are used to evaluate consistency of the soil in the field moisture condition. The results may be used to assess the undrained shear strength, $C_u$, of fine grained soil using the approximate relationship:

$q_u = 2 \times C_u$.

It should be noted that accuracy of the results may be influenced by condition variations at selected test surfaces. Also, the readings obtained from the PP test are based on a small area of penetration and could give misleading results. They should not replace laboratory test results. The use of the results from this test is typically limited to an assessment of consistency of the soil in the field and not used directly for design of foundations.

## Test Pit / Borehole Logs

Test pit / borehole log(s) presented herein are an engineering and / or geological interpretation of the subsurface conditions. Their reliability will depend to some extent on frequency of sampling and methods of excavation / drilling. Ideally, continuous undisturbed sampling or excavation / 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 test pit / borehole logs 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 test pits / boreholes, the frequency of sampling and the possibility of other than 'straight line' variation between the test pits / boreholes.

## Laboratory Testing

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

## Ground Water

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

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

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

# Test, Drill and Excavation Methods

## Explanation of Terms (3 of 3)

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### DRILLING / EXCAVATION METHOD

| | | | | | |
|---|---|---|---|---|---|
| HA | Hand Auger | RD | Rotary Blade or Drag Bit | NQ | Diamond Core - 47 mm |
| AD/V | Auger Drilling with V-bit | RT | Rotary Tricone bit | NMLC | Diamond Core – 51.9 mm |
| AD/T | Auger Drilling with TC-Bit | RAB | Rotary Air Blast | HQ | Diamond Core – 63.5 mm |
| AS | Auger Screwing | RC | Reverse Circulation | HMLC | Diamond Core – 63.5 mm |
| HSA | Hollow Stem Auger | CT | Cable Tool Rig | DT | Diatube Coring |
| S | Excavated by Hand Spade | PT | Push Tube | NDD | Non-destructive digging |
| BH | Tractor Mounted Backhoe | PC | Percussion | PQ | Diamond Core - 83 mm |
| JET | Jetting | E | Tracked Hydraulic Excavator | X | Existing Excavation |

### SUPPORT

| | | | | | |
|---|---|---|---|---|---|
| Nil | No support | S | Shotcrete | RB | Rock Bolt |
| C | Casing | Sh | Shoring | SN | Soil Nail |
| WB | Wash bore with Blade or Bailer | WR | Wash bore with Roller | T | Timbering |

### WATER

▽ Water level at date shown

▷ Water inflow

◁ Partial water loss

◀ Complete water loss

| | |
|---|---|
| GROUNDWATER NOT OBSERVED (NO) | The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit. |
| GROUNDWATER NOT ENCOUNTERED (NX) | The borehole/test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/test pit been left open for a longer period. |

### PENETRATION / EXCAVATION RESISTANCE

| | |
|---|---|
| L | Low resistance: Rapid penetration possible with little effort from the equipment used. |
| M | Medium resistance: Excavation possible at an acceptable rate with moderate effort from the equipment used. |
| H | High resistance: Further penetration possible at slow rate & requires significant effort equipment. |
| R | Refusal/ Practical Refusal. No further progress possible without risk of damage/ unacceptable wear to digging implement / machine. |

These assessments are subjective and dependent on many factors, including equipment power, weight, condition of excavation or drilling tools, and operator experience.

### SAMPLING

| | | | | | |
|---|---|---|---|---|---|
| D | Small disturbed sample | W | Water Sample | C | Core sample |
| B | Bulk disturbed sample | G | Gas Sample | CONC | Concrete Core |

U63 Thin walled tube sample - number indicates nominal undisturbed sample diameter in millimetres

### TESTING

| | |
|---|---|
| SPT | Standard Penetration Test to AS1289.6.3.1-2004 |
| 4,7,11 | 4,7,11 = Blows per 150mm. |
| N=18 | 'N' = Recorded blows per 300mm penetration following 150mm seating |
| DCP | Dynamic Cone Penetration test to AS1289.6.3.2-1997. 'n' = Recorded blows per 150mm penetration |

**Notes:**

| | |
|---|---|
| RW | Penetration occurred under rod weight only |
| HW | Penetration occurred under hammer and rod weight only |
| 20/100mm | Where practical refusal or hammer double bouncing occurred, blows and penetration for that interval are reported (e.g. 20 blows for 100 mm penetration) |

| | |
|---|---|
| CPT | Static cone penetration test |
| CPTu | CPT with pore pressure (u) measurement |
| PP | Pocket penetrometer test expressed as instrument reading (kPa) |
| FP | Field permeability test over section noted |
| VS | Field vane shear test expressed as uncorrected shear strength (sv = peak value, sr = residual value) |
| PM | Pressuremeter test over section noted |
| PID | Photoionisation Detector reading in ppm |
| WPT | Water pressure tests |

### SOIL DESCRIPTION

| Density | | Consistency | | Moisture | |
|---|---|---|---|---|---|
| VL | Very loose | VS | Very soft | D | Dry |
| L | Loose | S | Soft | M | Moist |
| MD | Medium dense | F | Firm | W | Wet |
| D | Dense | St | Stiff | Wp | Plastic limit |
| VD | Very dense | VSt | Very stiff | Wl | Liquid limit |
| | | H | Hard | | |

### ROCK DESCRIPTION

| Strength | | Weathering | |
|---|---|---|---|
| VL | Very low | EW | Extremely weathered |
| L | Low | HW | Highly weathered |
| M | Medium | MW | Moderately weathered |
| H | High | SW | Slightly weathered |
| VH | Very high | FR | Fresh |
| EH | Extremely high | | |