The Anglican Schools Corporation (TASC)
C/- Midson Group

martens consulting engineers

Preliminary Geotechnical Assessment - Proposed St Luke's Grammar School Senior School and Sports Centre - 800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P1907215JR02V01
October 2019

Limitations Statement

The sole purpose of this report and the associated services performed by Martens & Associates Pty Ltd is to complete a preliminary geotechnical assessment in accordance with the scope of services set out by The Anglican Schools Corporation (TASC) C/- Midson Group (hereafter known as the Client). That scope of works and services were defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

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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
Site Address	800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW ('the Site')
Legal Identifier	800 Pittwater Road, Dee Why: Lot 6 DP 523299 224 Headland Road, North Curl Curl: SP 45082
Site Area	Approximately 1.5 ha (SIX Maps, 2019)
Local Government Area	Northern Beaches Council ('Council') – Former Warringah Council
Proposed Development	We understand from a brief by the client as well as proposal plans (TZGA, 2019) that the development will include alterations and additions to the existing buildings at the Site to incorporate 224 Headland Road as a sports centre and 800 Pittwater Road as a designated senior school for 600 senior school students into St Luke's Grammar School (SLGS), located at 210 Headland Road. No works are proposed to the existing building at SLGS. The proposed alterations and additions will require excavations as below: o 800 Pittwater Road: Up to approximately 6 m below ground level (mbgl) in the northern, north western, southern and western portions, some of which may extend below the existing site buildings. o 224 Headland Road: Up to approximately 19 mbgl, in the northern undeveloped portion, for a new lift shaft.
Assessment Purpose	The site is mapped on the Northern Beaches Council landslip risk map as 'Area A' (western portion of 800 Pittwater Road and southern portion of 224 Headland Road) and 'Area B' (remainder of the Site) – Refer Figure 1, Attachment A. According to Warringah Council's Local Environmental Plan (2011), *Part E: The Natural Environment:* o A geotechnical report is not normally required for 'Area A'. o A geotechnical report is required for 'Area B' for excavation depth greater than 2 m. As the site will require bulk excavations > 2 mbgl, this preliminary geotechnical assessment has been prepared to support a State Significant Development (SSD) application.
Investigation Scope of Work	Field investigations conducted on 14 June 2019 included: o A general site walkover survey. o Review of DBYD survey plans and buried service locating on site. o Four boreholes (BH101 to BH104), using a 4WD truck-mounted hydraulic drill rig, to between 0.45 mbgl and 3.3 mbgl (refer Attachment D for borehole logs, and associated explanatory notes in Attachment I). o Collection of soil samples for future reference. o Four DCP tests (DCP101 to DCP104) to between 0.4 mbgl and 2.75 mbgl (refer DCP 'N' counts in Attachment E). Investigation locations are shown in Figure 2, Attachment B.

2 General Site Details and Investigation Findings

General site details and investigation findings are summarised in Table 2.

Table 2: Summary of general site details and investigation findings.

Item	Comment
Topography	Within undulating terrain located on a spur. The eastern portion of the site (i.e. 224 Headland Road) is located on the crest of spur. Relatively steep slopes of up to approximately 50 % step down towards the western portion of 800 Pittwater Road, located on the western edge of the spur.
Typical Slopes, Aspect, Elevation	The site generally has a westerly aspect with grades as below: 800 Pittwater Road: Overall grade of approximately 10 %. 224 Headland Road: ○ Less than 5 % in the areas underneath and immediately surrounding the existing building. ○ Steep slopes of up to approximately 50 % in the undeveloped portion (i.e. northern end as well as on the natural slope below the suspended slab) Site elevation ranges between approximately 57.5 mAHD (eastern boundary) and 31.5 mAHD (western boundary).
Expected Geology and Soil Landscape	Hawkesbury Sandstone comprising medium to coarse grained quartz sandstone with very minor shale and laminite lenses (*Sydney 1:100 000 Geological Sheet 9130,* 1983). The NSW Environment and Heritage eSPADE website identifies the site as having soils of the Gymea Landscape comprising shallow to moderately deep yellow earths and earthy sands on crests and insides of benches; shallow siliceous sands on leaning edges of benches; localised gleyed podzolic soils and yellow podzolic soils on shale lenses; shallow to moderately deep siliceous sands and leached sands along drainage lines.
Existing Developments	800 Pittwater Road contains a three storey mixed-use building, including a single level basement carpark. Concrete driveway and carpark surround the building to the north, west and south. 224 Headland Road is a light industrial complex containing seven industrial units and partly suspended concrete slab driveway / carpark, supported by piles, to the west of the units. The northern end of this property is undeveloped. Longitude cracks were observed in the asphalt pavement of driveway / carpark slab, which could be a sign of potential differential settlement between suspended slab and slab on filled ground. All Site buildings are expected to have been founded in bedrock using shallow or deepened footings, depending on foundation level.
Vegetation	800 Pittwater Road: Scattered trees along the western and southern boundaries. 224 Headland Road: relatively dense vegetation, including grass, bushes and trees within the undeveloped portion (i.e. northern end as well as on the natural slope below the suspended slab.
Drainage	Via overland flow to the west into Council's stormwater network system along Pittwater Road and Harbord Road
Acid Sulfate Soil	The site has not been mapped as having acid sulfate soil risk (LIR, 2019).
Saline Soil	The site has not been mapped as having saline soil (LIR, 2019).
Subsurface Soil / Rock Units	Sandstone was exposed in cuttings for former site development and in surrounding areas, such as along the northern and eastern boundaries as well as

Item	Comment
	basement walls of 800 Pittwater Road, within the undeveloped (northern) portion of 224 Headland Road and along Pittwater Road. The rock was overlain by a thin (< 1 m) soil profile. Presence of very steep to near vertical joints was observed in exposed rock. Bedding planes were observed generally dipping towards the west angled at 10 ° to 15 ° (refer rock exposure photos in Attachment E). Investigations revealed the following key subsurface units underlie a concrete and asphalt pavement of up to 0.3 thickness across the investigated area: Unit A: Fill comprising medium dense clayey sand and sand, encountered to between approximately 0.4 mbgl (BH101) and 1.5 mbgl (BH103). This unit is likely to be encountered in the western and southern portions of 800 Pittwater Road, i.e. beneath the concrete slab as well as beneath the asphalt covered slab on ground driveway / carpark in 224 Headland Road, placed for site levelling purposes. It is assumed to have been placed under "uncontrolled" conditions for the purpose of this report. Unit B: Residual loose grading to dense clayey sand was only encountered in BH102 beneath Unit A, between approximately 1.0 mbgl and 2.9 mbgl (possibly extremely weathered rock below 2.2 mbgl). This unit is expected to overlie Unit C in some portions of the site, such as undeveloped (northern) portion as well as the natural slope below the suspended slab of 224 Headland Road. shallower residual soil profiles are expected to be present in the eastern steeper portion of the site. Residual soil thickness is likely to increase within the flatter (western) portion of the site. Unit C: Weathered and inferred low to medium strength sandstone from between approximately 0.4 mbgl (BH101) and 2.9 mbgl (BH102) up to TC-bit refusal and investigation termination depths of between 0.45 mbgl (BH101) and 3.3 mbgl (BH102). Rock level across the site is expected to step down towards the west. For the purpose of this report, sandstone below TC-bit refusal is assumed to be of medium to high strength with possible lower strength bands, which should be confirmed / revised by further assessment, as necessary.

3 Hydrogeological Assessment

3.1 NSW Department of Primary Industries Bore Search

A review of the NSW Department of Primary Industries Water (DPIW) real time groundwater bore database revealed that there are ten groundwater bores with available groundwater data within 1000 m of the site, as summarised in Table 3 (refer Figure 3, Attachment C, for groundwater bore locations relative to the site).

Table 3: DPIW real time groundwater details for ten bores within 1000 m of the site.

Bore #	Approx. Distance From Site (km)	Approximate Elevation (mAHD) [1]	Standing Water Level (mbgl)	Approx. Groundwater Level (mAHD)
GW105850	335 m (northeast)	30	1.4	28.6
GW105849	352 m (northeast)	26	1.4	24.6
GW107655	771 m (southeast)	5	3.0	2.0
GW107782	913 m (southeast)	10	1.2	8.8
GW107867	919 m (southeast)	10	2.1	7.9
GW108029	928 m (southeast)	5	2.0	3.0
GW111038	934 m (southeast)	6	2.0	4.0
GW107362	940 m (southeast)	8	2.0	6.0
GW072222	940 m (southeast)	6	1.0	5.0
GW110933	991 m (southeast)	7	1.9	5.1

Notes:

1. Approximate from Google Earth (± 5 m).

All measured groundwater levels are below proposed bulk excavation levels.

3.2 Groundwater Observations

- Groundwater inflow was not observed in the boreholes up to 3.3 mbgl (approximate RL 33.9 mAHD).
- No groundwater seepage was observed from basement rock face or rock cuttings.
- Rock staining as a result of ephemeral perched groundwater seepage from the soil / rock interface was observed along some rock cuttings.

3.3 Conclusion

Given the site topography, location and elevation, Table 3 as well as site investigation findings / observations it is considered unlikely that the proposed excavations will intercept the permanent groundwater table. However, ephemeral perched groundwater may be encountered in the soil profile or at the soil / rock interface originating from infiltration of surface water within and upslope of the site during prolonged or intense rainfall events.

Long term groundwater conditions should be confirmed with additional testing (i.e. installation of groundwater monitoring wells) prior to construction, as necessary.

4 Geotechnical Assessment

4.1 Preliminary Material Properties

Preliminary soil and rock properties, estimated from field test results in conjunction with borehole derived soil / rock profile data, as well as engineering judgement, are summarised in Table 4.

Table 4: Preliminary material properties.

Layer	Material	$\gamma_{in\text{-}situ}$ [1] (kN/m³)	UCS [2] (MPa)	Ø' [3] (deg)	E' [4] (MPa)
FILL [5]	Clayey SAND / SAND	17-18	NA [6]	NA [6]	NA [6]
RESIDUAL SOIL:	Clayey SAND (loose)	16	NA [6]	27	5
	Clayey SAND (medium dense)	18	NA [6]	32	15
	Clayey SAND (dense / possibly Class V SANDSTONE [7])	21	NA [6]	35	30
WEATHERED ROCK:	SANDSTONE (Class IV [7])	23	1 – 10	37	100-300
	SANDSTONE (Class IV / III [7]) [8]	23	10 - 30	40	300 - 500

Notes:

1. Material in-situ unit weight, based on visual assessment (±10 %).
2. Expected range of unconfined compressive strength of intact rock substance.
3. Effective internal friction angle (±2 ˚) estimate, assuming drained conditions; may be dependent on rock defect conditions.
4. Expected range of effective elastic modulus (±10 %).
5. Assumed "uncontrolled".
6. Not applicable.
7. In accordance with Pells et al., 1998. Rock classification, including rock strength, should be confirmed / revised by further rock coring,
8. Assumed below TC-bit refusal depths.

4.2 Risk of Slope Instability

No evidence of former land instability within the Site and surrounding land was observed during the site walkover survey:

- There was no evidence of subsidence or recent gross slope instability onsite.
- The existing buildings showed no significant visible cracking or settlement. A detailed structural integrity survey / dilapidation survey was not undertaken.

- Concrete / asphalt driveways exhibited only minor settlement and cracking, likely due to fill subgrade conditions.

We consider two geotechnical hazards are most likely to impact the proposed development:

- Shallow rotational slide through soils and / or weathered rock.
- Rock block falling (i.e. toppling) / failure (i.e. planar and / or wedge failure).

The proposed development is considered to constitute an acceptable risk to life and a low risk to property in accordance with qualitative risk matrices provided in Section 7 of the Australian Geomechanics Society (2007) guidelines, resulting from assessed geotechnical hazards. This is subject to the recommendations provided in Section 5 and in Attachment H and adoption of industry standard design and construction as well as good hillslope engineering practices (Attachment G).

5 Geotechnical Recommendations and Future Works

5.1 Geotechnical Recommendations

Preliminary geotechnical recommendations for the proposed development are provided below. Further general geotechnical recommendations are provided in Attachment H.

1. Excavation: Proposed excavations will encounter fill / residual soils over extremely weathered grading to medium to high strength sandstone. In light of this, we expect the following excavation equipment will be required:

    - Soils and extremely low to low strength rock: Conventional earthmoving equipment may be adopted. A 'toothed' bucket or a ripping tyne (or similar) may be required to excavate weathered rock.

    - Medium to high strength rock: Large hydraulic earthmoving equipment with ripping tyne attachment, although a rock hammer will likely be required for excavation of massive rock and for higher strength rock. Saw cutting of rock may assist excavation by rock hammer.

2. Excavation Support: Excavations in soils and extremely low to low strength rock must be temporarily and permanently battered back / shored / retained to maintain excavation stability. Medium and / or higher strength rock may remain unsupported subject to confirmation on site by a geotechnical engineer. Appropriate support and / or excavation methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer.

    Excavations in soils and extremely low to low strength rock may be temporarily battered back at:

    - 1V:2H for soil and 1V:1.5H for rock, or

    - 1V:1H, if covered with an appropriate protection facing, e.g. by shotcrete and soil nails.

    Provided temporary batters are subject to inspection and approval by a geotechnical engineer on site. Permanent batter slopes should not exceed 1V:3H.

Excavations in medium and / or higher strength rock may be carried out vertically, subject to inspection and approval by a geotechnical engineer on site.

Temporary shoring may include cantilevered or anchored soldier pile walls with shotcrete infill panels. These may be designed for inclusion in permanent support structures.

Retaining wall design may adopt active (k_a), at rest (k_0) and passive (k_p) earth pressure coefficients of 0.31, 0.47 and 3.26, respectively, for soils.

Tieback anchors must not be installed across site boundaries unless written confirmation of acceptance is obtained from neighbouring property / asset owners.

Retaining wall design should consider additional surcharge loading from live loads, new and existing structures, construction equipment, backfill compaction and static water pressures unless subsoil drainage is provided behind retaining walls.

3. Rock Support: Unstable rock wedges and columns as a result of presence of clay seams, weakly cemented (extremely weathered) seams, steeply dipping joints and other rock defects may have an adverse effect on unsupported rock face stability and construction safety. Geotechnical mapping of the excavation should be conducted in 1.5 m height increments to identify such features and allow early mitigation of risks of rock movement, such as by installation of rock bolts and / or sprayed concrete surfacing.

   If full height retaining walls are to be constructed for long term stability, then such measures are not likely to be required, however temporary measures may be required to stabilise excavations during construction.

   Rock bolts and sprayed concrete support should be specified in terms of performance requirements and installed / placed by contractors experienced in ground anchor technology and on advisement by an experienced geotechnical engineer. The actual amount of stabilisation which will be required cannot be quantified at this stage and can only be determined at the time of construction. MA can complete the necessary mapping and provide advice on support requirements.

4. Ground Vibrations: Care will be required to limit structural distress caused by rock excavation-induced vibrations to the existing structures on site as well as adjoining neighbouring structures to the south. This may be achieved by:

    a. Adopting appropriate demolition and construction methodologies, including limiting hammering of concrete during demolition and, if considered necessary, carrying out hammering as far from existing structures as possible.

    b. Rock sawing prior to the use of rock hammer, for medium and higher strength rock excavation, or to break up any concrete slabs.

    c. Monitoring ground vibrations during construction.

    We recommend limiting ground vibration peak particle velocities (PPV) at existing structures to 10 mm/s (AS 2187.2, 2006).

5. Existing Footings: Where existing building footings are likely to be impacted by the proposed alterations / additions, or capacity of the existing footings to support higher loads due to additions is assessed to be inadequate, they will require strengthening or underpinning. Methodologies for remediation works should be provided by an experienced contractor and should be reviewed and approved by a geotechnical engineer as part of the construction certification documentation process.

6. New Footings and Foundations: All new structural loads should be transmitted into the bedrock to limit differential movements across the site. Allowable end bearing capacities for design of shallow and deepened footings are presented in Section 5.2. Individual pad footings and all footings within building footprints should not span the interface between different foundation materials. Alternatively, inclusion of movement joints may mitigate impacts of differential movements.

    Footings should be designed by a suitably qualified and experienced structural or geotechnical engineer. All foundation excavations should be inspected by a geotechnical engineer to confirm encountered conditions satisfy design assumptions.

7. Groundwater / Drainage requirements: Proposed excavations are expected not to intercept the permanent groundwater table. Groundwater inflow, if encountered during rock excavation, is expected to be limited and manageable by sump and pump methods.

Drainage systems should be designed to divert overland flows and potential perched groundwater away from excavations and from behind retaining walls, and limit ponding of water in excavations and near footings.

Appropriate drainage measures should be provided behind the retaining walls to divert ephemeral seepage water away from structures and discharge into council approved discharge points downslope.

8. Site Classification: The site is classified as a class 'P' site in accordance with AS 2870 (2011), due to presence of assumed 'uncontrolled' fill across the site.

   A reclassification to "A" may be considered for lightly loaded shallow footings founding on rock. This site classification is subject to the recommendations presented in this report, the design of footings in accordance with the relevant Australian Standards and guidelines and the following conditions:

   - Provision of adequate drainage of surface and subsurface water to limit soil moisture variations impacting on foundation conditions.

   - Footings are unlikely being impacted by the presence of environments that could lead to exceptional foundation material movements, such as existing or future trees or surface water accumulation.

9. Construction Consideration: Contractor should assess the capability of their plant to excavate through medium to high and potential higher strength rock, when developing their excavation methodologies, and consider accessibility of chosen plant.

5.2 Allowable Bearing Capacities

Table 5 presents allowable bearing capacities that may be adopted for design of shallow and deepened footings.

Table 5: Allowable bearing capacities.

Unit	Shallow Footings	Piles [1]	
	ABC [2, 4]	ABC [2, 4]	ASF [3, 4]
WEATHERED ROCK: SANDSTONE (Class V)	300	700	50
WEATHERED ROCK: SANDSTONE (Class IV)	500	1500	200
ROCK: SANDSTONE (Class IV/III)	1000	3000	300

Notes:

1. Assuming bored cast in-situ pile.
2. Allowable end bearing capacity (kPa) for shallow footings embedded at least 0.3 m and piles embedded at least 0.5 m into design material type, subject to confirmation on site by a geotechnical engineer of inferred foundation conditions.
3. Allowable skin friction (kPa) below 1 m depth for bored pile in compression, assuming intimate contact between pile and foundation material. For up lift resistance, we recommend reducing ASF by 50% and checking against 'piston' and 'cone' pull-out mechanisms in accordance with AS2159 (2009).
4. ABC and ASF are given with estimated factors of safety of 3 and 2 respectively, generally adopted in geotechnical practice to limit settlement to an acceptable level for conventional building structures (< 1% of minimum footing width).

5.3 Proposed Further Works

5.3.1 Works Prior to Construction

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

1. Assessment of existing footing types and conditions as well as potential adverse impacts on them by the proposed alterations / additions.

2. Assessment of existing fill / natural batter stability conditions in the northern portion of 224 Headland Road to identify potential land instability.

3. Rock defect mapping of exposed rock to assist assessing risk of rock instability, including as a result of rock excavations.

4. To confirm rock classification or if higher end bearing pressures or better understanding of rock conditions are required, carry out rock coring and point load testing of collected rock samples to assess rock strength.

5. Assessment of existing retaining wall conditions and any possible remediation requirements.

6. Further assessment along the asphalt pavement to determine cause of pavement cracking and long term impacts on the development.

7. Review of the final design by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report. In conjunction with the results from further geotechnical investigation, a more detailed landslide risk assessment is to be carried out, considering final design details, in accordance with Council's landslide risk management guidelines.

5.3.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 6).

Table 6: Recommended inspection / monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall/anchoring) installations and exposed batters / rock excavations at maximum 1.5 m height intervals, to assess need for additional support requirements.	Daily / 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.	Daily at on-set of excavation and as agreed thereafter [2]	MA [1]
Inspect underpinning / strengthening works to ensure compliance with design assumptions and industry standards, if required.	Daily / As required [2]	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]
Inspect fill / natural batter in the northern portion of 224 Headland Road to ensure no evidence of instability as a result of construction works.	Weekly / As required [2]	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

Notes:

1. MA = Martens and Associates engineer.
2. MA inspection frequency to be determined based on initial inspection findings in line with construction program

6 References

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Tonkin Zulaikha Greer Architects (2019) St Luke's Grammar School; New Senior Campus – SEARS Application, dated 3 April 2019 (TZGA, 2019).

Stephen R. Carr Surveyor (2019) survey plan for 800 Pittwater Road, Dee Why, NSW, Plan No. 1819-10, Sheets 1 and 2, dated 16 January 2019 (SRCS, 2019).

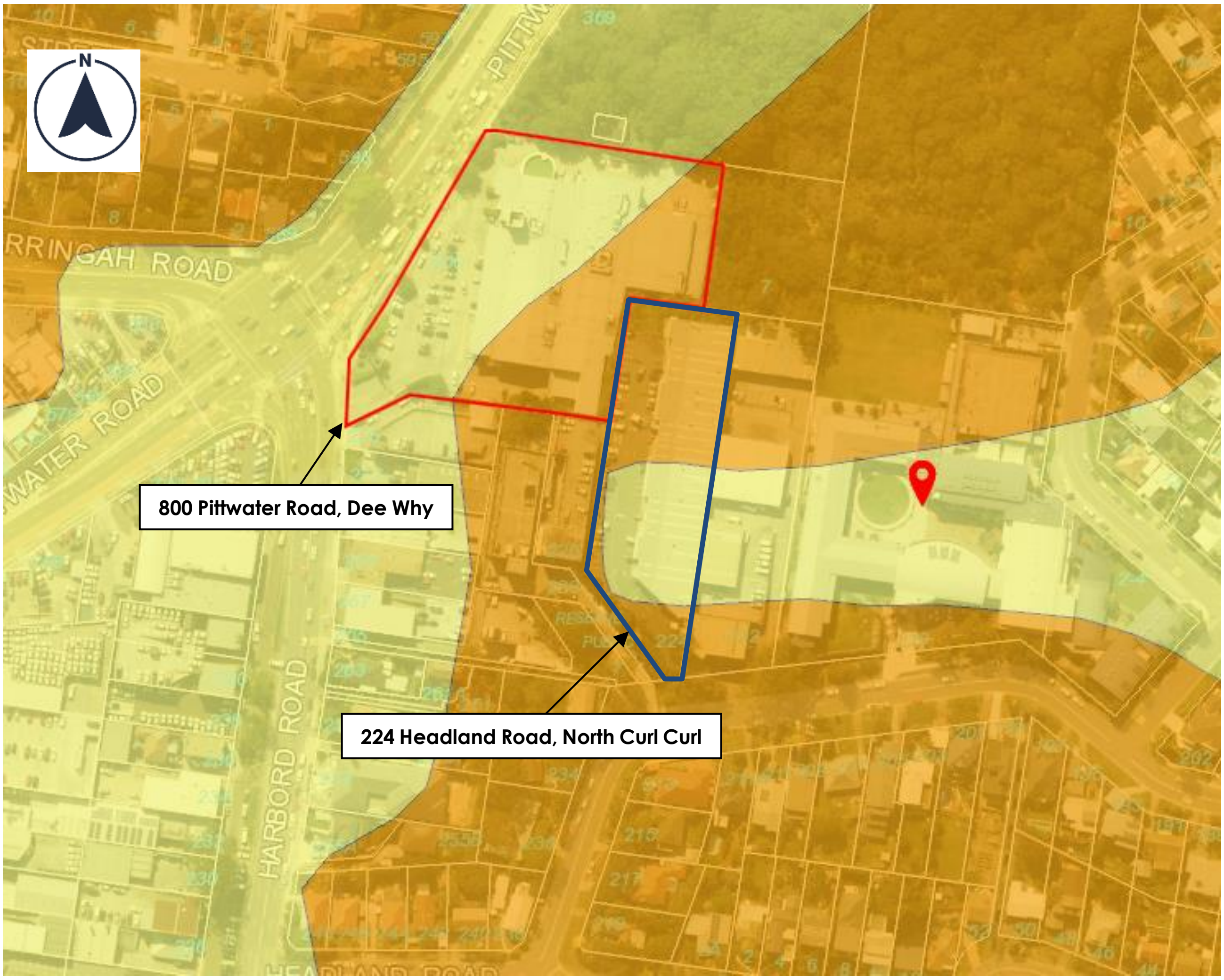
Stephen R. Carr Surveyor (2017) survey plan for 224 Headland Road, North Curl Curl, NSW, Plan No. 1819, dated 14 December 2017 (SRCS, 2017).

7 Attachment A – Site Location on Council Landslip Risk Map

LEGEND

Warringah Landslip Risk Map

- Area A - Slope less than 5 degrees
- Area B - Flanking Slopes from 5 to 25 degrees



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Martens & Associates Pty Ltd	ABN 85 070 240 890	Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	HN	SITE LOCATION RELATIVE TO COUNCIL LANDSLIP RISK MAP 800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW (Source: Council, 2019)	Drawing No: FIGURE 1
Approved:	RE		
Date:	02.07.2019		
Scale:	Not to Scale		File No: P1907215JR02V01

8 Attachment B – Geotechnical Testing Plan

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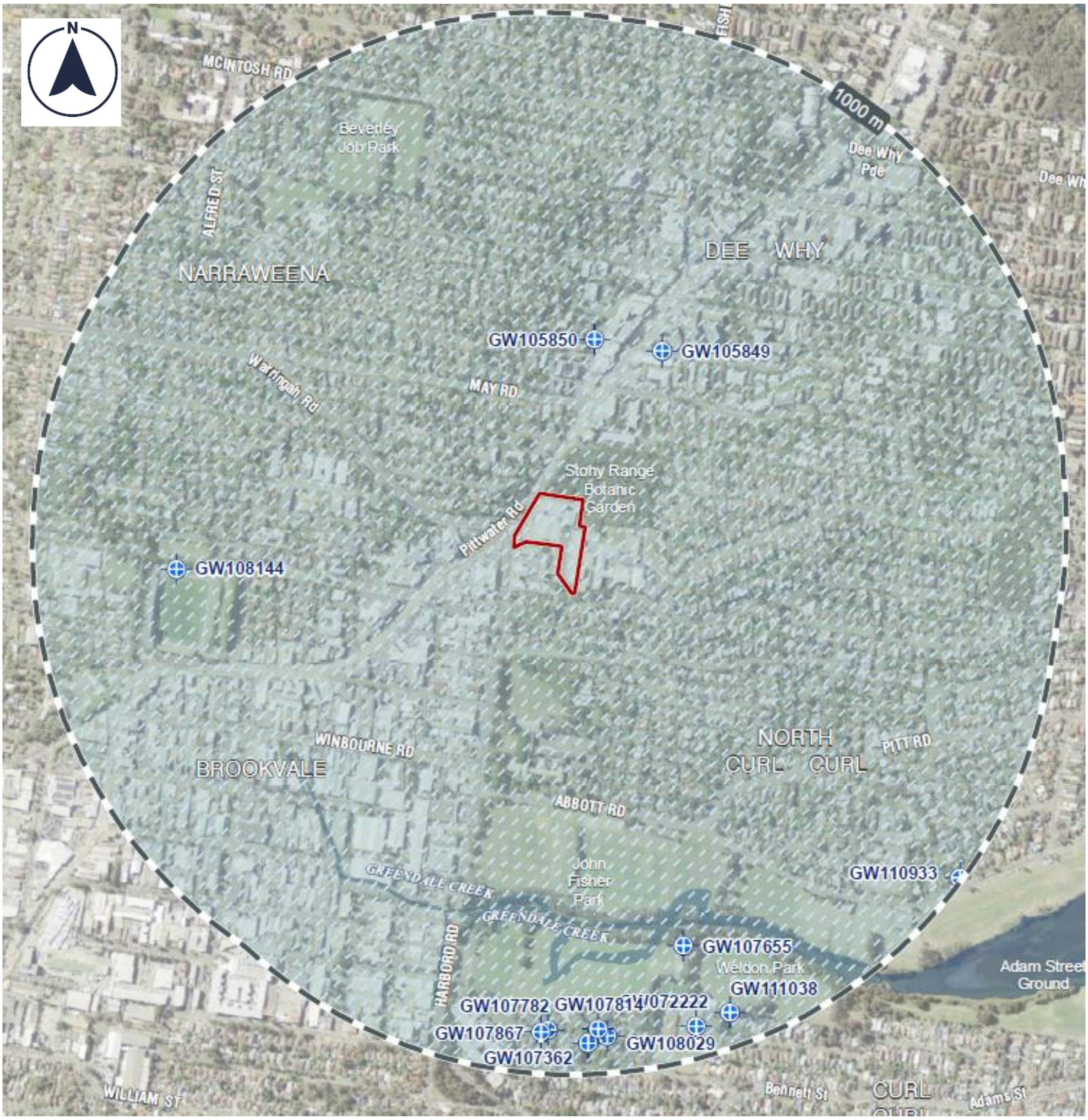
Key:

Approximate borehole and DCP test location

Indicative site boundary

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	HN	**GEOTECHNICAL TESTING PLAN** **800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW** (Source: SIX Maps, 2019)	Drawing No: **FIGURE 2**
Approved:	RE		
Date:	02.07.2019		
Scale:	NA		Project No: P1907215JR0V01

9 Attachment C – Available Groundwater Well Data



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Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	HN	**AVAILABLE GROUNDWATER WELLS AROUND THE SITE** **800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW** (Source: LIR, 2019)	Drawing No: **FIGURE 3**
Approved:	RE		
Date:	03.07.2019		
Scale:	Not to Scale		File No: P1907215JR02V01

10 Attachment D – Test Borehole Logs

CLIENT	The Anglican School's Corporation	COMMENCED	14/06/2019	COMPLETED	14/06/2019	**REF BH101**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HN	CHECKED	RE	Sheet 1 OF 1
SITE	800 Pittwater Rd, Dee Why, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	None	PROJECT NO. P1907215

EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	39 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 0.45 m depth	NORTHING		ASPECT	West	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
DT		Not Encountered		39.00 *0.20*					CONCRETE SLAB.			PAVEMENT
AD/T	L			38.80 *0.40*	P7215/101/0.2-0.3/S/1 D 0.20 m			SC	FILL: Clayey SAND: medium grained; grey and brown; trace sandstone gravels.	D / M	MD	FILL
	H			*0.45*					SANDSTONE: medium grained; red-brown and brown; inferred low to medium strength.			WEATHERED ROCK 0.45: TC-bit refusal on inferred medium to high strength sandstone.
			0.5						Hole Terminated at 0.45 m			
			1.0									
			1.5									
			2.0									
			2.5									
			3.0									
			3.5									

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

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Engineering Log - BOREHOLE

CLIENT	The Anglican School's Corporation	COMMENCED	14/06/2019	COMPLETED	14/06/2019	REF BH102
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HN	CHECKED	RE	Sheet 1 OF 1
SITE	800 Pittwater Rd, Dee Why, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	None	PROJECT NO. P1907215
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	37.2 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 3.30 m depth	NORTHING		ASPECT	Southwest	SLOPE: <5%

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
DT		Not Encountered		37.20					CONCRETE SLAB.			PAVEMENT
AD/V	L			0.22 / 36.98	P7215/102/0.3-0.4/S/1 D 0.30 m			SC	FILL: Clayey SAND: medium to coarse grained; brown and grey; with fine to medium sandstone and roadbase gravels.	M	MD	FILL
			0.5	0.50 / 36.70					Pale brown and orange.			
				0.75 / 36.45	P7215/102/0.8-1.0/S/1 D 0.80 m				Grey.			
			1.0	1.00 / 36.20				SC	Clayey SAND: medium to coarse grained; grey and green.		L	RESIDUAL SOIL
			1.5	1.60 / 35.60	P7215/102/1.5-1.6/S/1 D 1.50 m				Brown and red-brown; trace sandstone gravels.		MD	
			2.0	2.20 / 35.00					Pale grey with brown.		D and VD	2.20: Possibly extremely weathered rock.
			2.5		P7215/102/2.4-2.5/S/1 D 2.40 m							
AD/T	M-H		3.0	2.90 / 34.30					SANDSTONE: medium grained; grey; inferred low to medium strength; distinctly weathered.			WEATHERED ROCK 2.90: V-bit refusal.
				3.30					Hole Terminated at 3.30 m			3.30: TC-bit refusal on inferred medium to high strength sandstone.
			3.5									

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

CLIENT	The Anglican School's Corporation	COMMENCED	14/06/2019	COMPLETED	14/06/2019	REF BH103
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HN	CHECKED	RE	Sheet 1 OF 1
SITE	800 Pittwater Rd, Dee Why, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	None	PROJECT NO. P1907215
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	37.9 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 2.00 m depth	NORTHING		ASPECT	West	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
DT		Not Encountered		37.90					CONCRETE SLAB.			PAVEMENT
AD/V	L-M			0.20 37.70	P7215/103/0.4-0.5/S/1 D 0.40 m			SC	FILL: Clayey SAND: medium grained; brown and grey with red-brown; with sandstone and roadbase gravels; inferred medium dense.	D / M	MD	FILL
			1.0	1.00 36.90	P7215/103/1.2-1.4/S/1 D 1.20 m				With tree roots.			
AD/T	H		1.5	1.50 36.40					SANDSTONE: fine to medium grained; pale brown; inferred low to medium strength; distinctly weathered.			WEATHERED ROCK 1.50: V-bit refusal.
			2.0	2.00					Hole Terminated at 2.00 m			2.00: TC-bit refusal on inferred medium to high strength sandstone.

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

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P1907215 BH01V01.GPJ <<DrawingFile>> 03/07/2019 12:48 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Martens 2.00 2016-11-13 Prj: Martens 2.00 2016-11-13

CLIENT	The Anglican School's Corporation	COMMENCED	14/06/2019	COMPLETED	14/06/2019	REF BH104
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HN	CHECKED	RE	Sheet 1 OF 1
SITE	224 Headland Rd, North Curl Curl, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	None	PROJECT NO. P1907215
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	57.3 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 2.10 m depth	NORTHING		ASPECT	West	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
DT		Not Encountered		57.30					ASHPHALT and CONCRETE SLAB.			PAVEMENT
AD/V	L		0.5	*0.30* 57.00	P7215/104/0.3-0.4/S/1 D 0.30 m			SP	FILL: SAND: fine to medium grained; pale brown/yellow; with fine to medium sandstone gravels.	D	MD	FILL
AD/T	H		1.0, 1.5, 2.0	*0.90* 56.40					SANDSTONE: fine to medium grained; grey and pale brown; inferred low to medium strength.			WEATHERED ROCK 0.90: V-bit refusal.
			2.5, 3.0, 3.5	*2.10*					Hole Terminated at 2.10 m			2.10: TC-bit refusal on inferred medium to high strength sandstone.

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

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P1907215 BH01V01 - COPY.GPJ <<DrawingFile>> 03/07/2019 12:56 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Martens 2.00 2016-11-13 Prj: Martens 2.00 2016-11-13

11 Attachment E – DCP 'N' Counts

Dynamic Cone Penetrometer Test Log Summary

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Site	800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW	**DCP Group Reference**	P1907215JS01V01
Client	The Anglican Schools Corporation (TASC) C/- Midson Group	**Log Date**	14.06.2019
Logged by	HN		
Checked by	RE		
Comments	DCP101, DCP102 and DCP104 commenced below concrete slab. DCP103 commenced from 0.9 mBGL.		

TEST DATA

Depth Interval (m)	DCP101	DCP102	DCP103	DCP104
0.15	-	-	-	-
0.30	-	-	-	-
0.45	5 / 50 mm	6	-	10
0.60	Bounce @ 0.4 m	19	-	8
0.75		10	-	6
0.90		9	-	8 / 120 mm
1.05		3	15	Bounce @ 0.92 m
1.20		3	11	
1.35		3	7	
1.50		5	Bounce @ 1.4 m	
1.65		7		
1.80		6		
1.95		8		
2.10		11		
2.25		24		
2.40		22		
2.55		14		
2.70		22		
2.85		Bounce @ 2.75 m		
3.00				

12 Attachment F – Rock Exposure Photos

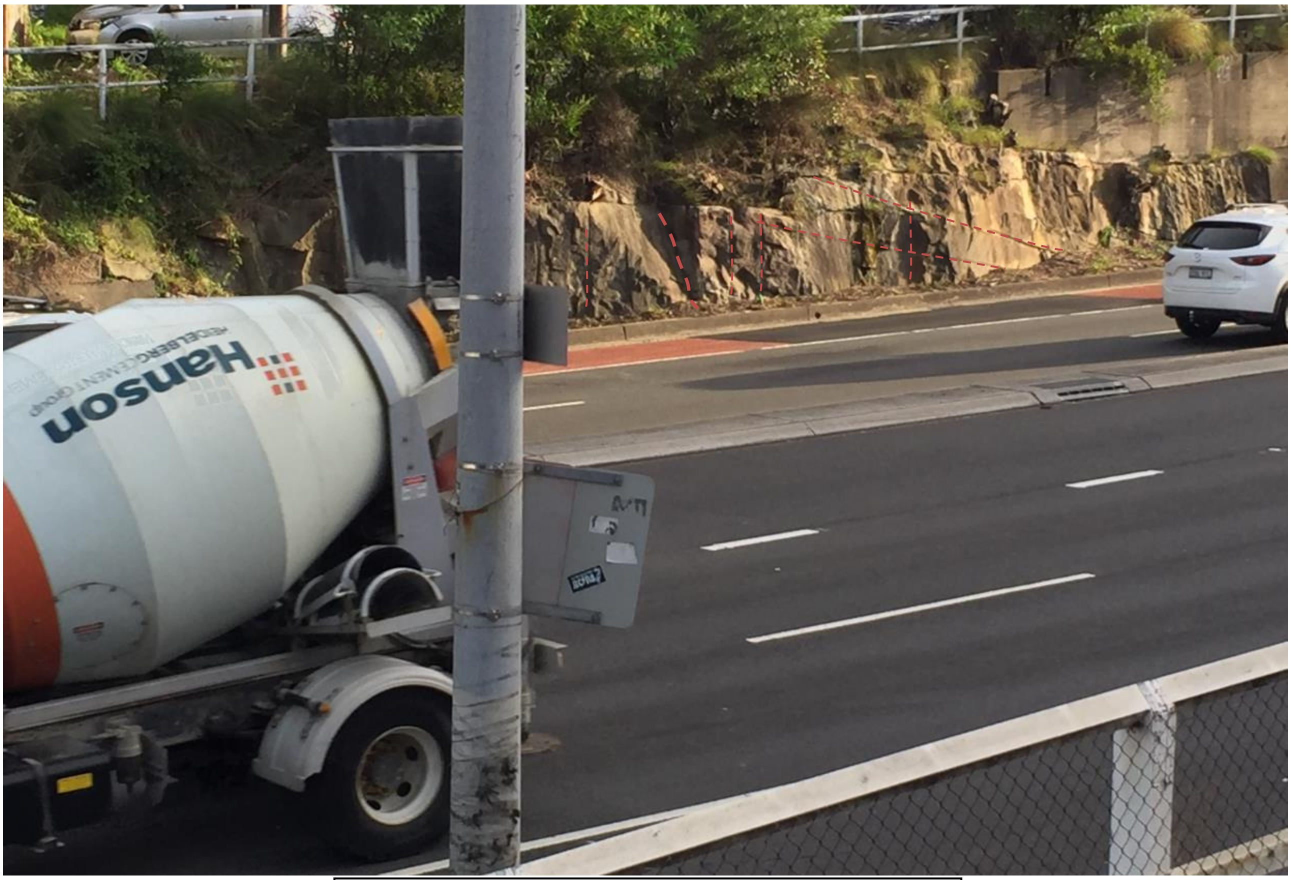

Plate 1 - Rock exposure along Pittwater Road cut – Looking towards the west

Plate 2 - Rock exposure, containing defects, along the northern site boundary – Looking towards the west

Key:
– – – Major rock defect sets

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Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	HN	**ROCK EXPOSURE PHOTOS** **800 Pittwater Road, Dee Why and 224 Headland Road, North Curl Curl, NSW**	Drawing No: **PLATES1 & 2**
Approved:	RE		
Date:	02.07.2019		
Scale:	Not to Scale		File No: P1907215JR01V01

Plate 3 - Rock exposure, containing defects, near entrance to 224 Headland Road – Looking towards the east

Key:

- - - Major rock defect sets

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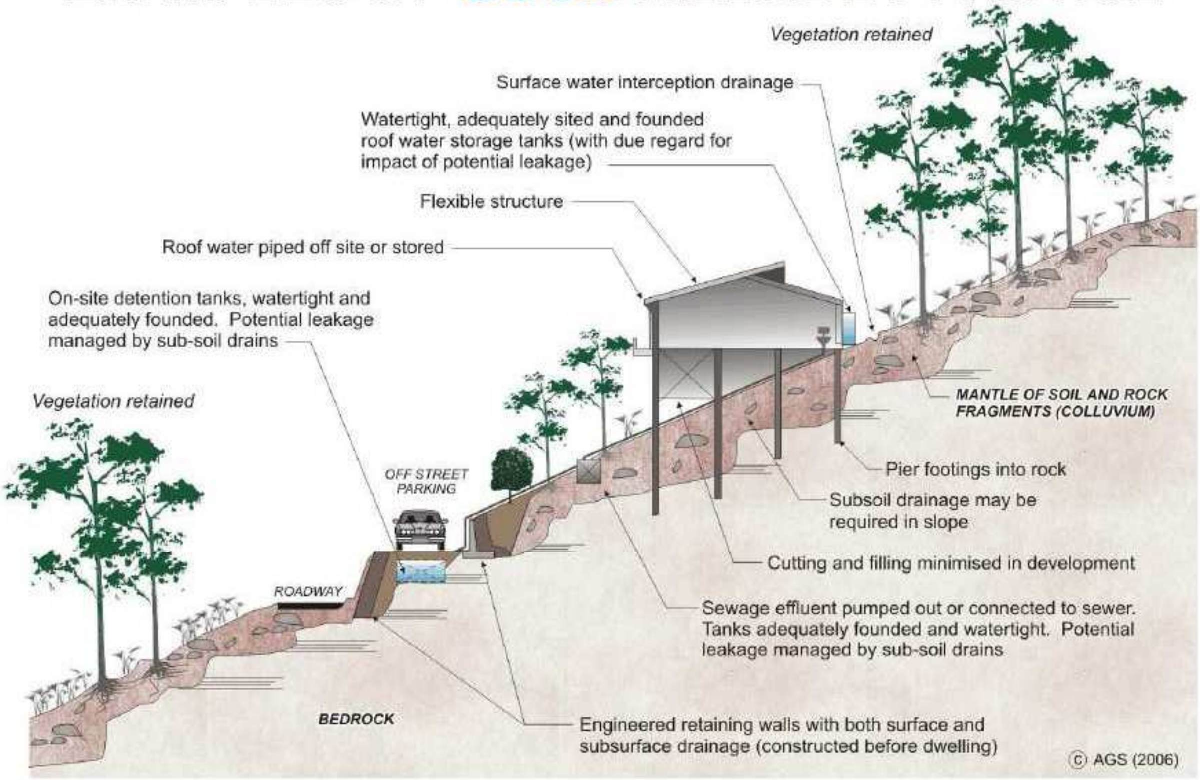
Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	HN	**ROCK EXPOSURE PHOTOS** **Near Entrance to 224 Headland Road,** **North Curl Curl, NSW**	Drawing No: **PLATE 3**
Approved:	RE		
Date:	02.07.2019		
Scale:	Not to Scale		File No: P1907215JR01V01

13 Attachment G – Hillside Construction Guidelines (AGS, 2007)

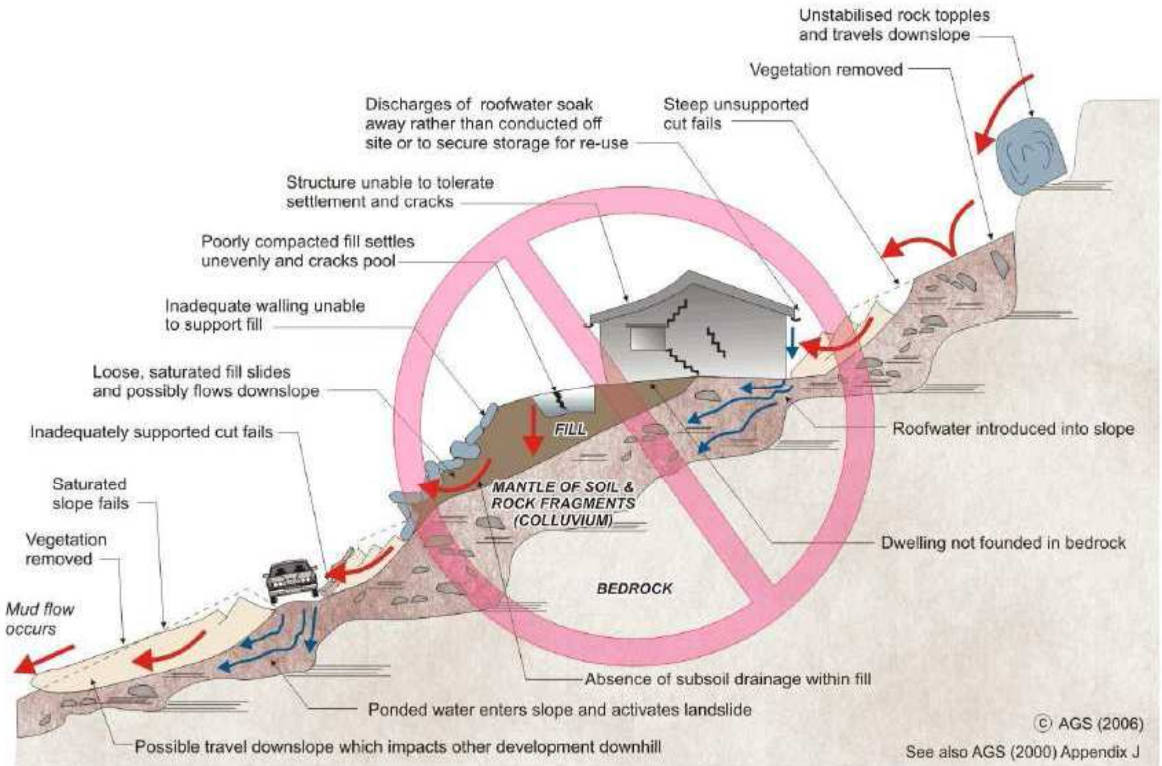
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE



14 Attachment H – 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.

15 Attachment I – Notes About This Report

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.

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, structure, soil or rock type and inclusions.

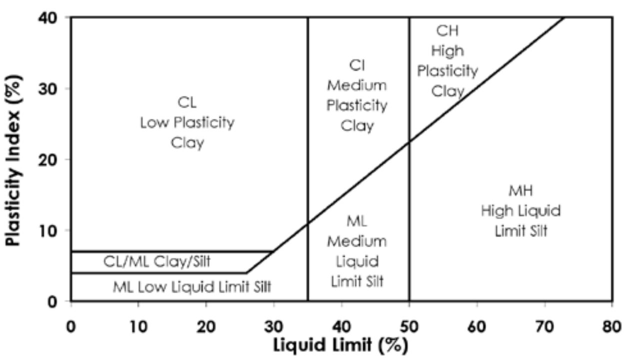
Particle Size

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	Size (mm)
BOULDERS		>200
COBBLES		63 to 200
GRAVEL	Coarse	20 to 63
	Medium	6 to 20
	Fine	2.36 to 6
SAND	Coarse	0.6 to 2.36
	Medium	0.2 to 0.6
	Fine	0.075 to 0.2
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.



Moisture Condition

Dry — Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist — Soil feels cool and damp and is darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet — As for moist but with free water forming on hands when handled.

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

Term	C_u (kPa)	Approx. SPT "N"	Field Guide
Very Soft	<12	2	A finger can be pushed well into the soil with little effort. Sample extrudes between fingers when squeezed in fist.
Soft	12 - 25	2 – 4	A finger can be pushed into the soil to about 25mm depth. Easily moulded in fingers.
Firm	25 - 50	4 – 8	The soil can be indented about 5mm with the thumb, but not penetrated. Can be moulded by strong pressure in the figures.
Stiff	50 - 100	8 – 15	The surface of the soil can be indented with the thumb, but not penetrated. Cannot be moulded by fingers.
Very Stiff	100 - 200	15 – 30	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	> 200	> 30	The surface of the soil can be marked only with the thumbnail. Brittle. Tends to break into fragments.
Friable	-	-	Crumbles or powders when scraped by thumbnail.

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 - 35	5 - 10	2 - 5
Medium dense	35 - 65	10 - 30	5 - 15
Dense	65 - 85	30 - 50	15 - 25
Very dense	> 85	> 50	> 25

* Values may be subject to corrections for overburden pressures and equipment type.

Minor Components

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

Term	Assessment	Proportion of Minor component In:
Trace of	Presence just detectable by feel or eye. Soil properties little or no different to general properties of primary component.	Coarse grained soils: < 5 % Fine grained soils: < 15 %
With some	Presence easily detectable by feel or eye. Soil properties little different to general properties of primary component.	Coarse grained soils: 5 – 12 % Fine grained soils: 15 – 30 %

Soil Data

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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 (OH)
- CLAY (CL, CI OR CH)
- SILTY CLAY
- SANDY CLAY
- PEAT
- TOPSOIL

OTHER

- FILL
- TALUS
- ASPHALT
- CONCRETE

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 50 % 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 2.0 mm.	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	Gravel
				Predominantly one size or a range of sizes with more intermediate sizes missing	GP	Gravel
			GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty Gravel
				Plastic fines (for identification procedures see CL below)	GC	Clayey Gravel
		SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of intermediate sizes missing.	SW	Sand
				Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Sand
			SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	SM	Silty Sand
				Plastic fines (for identification procedures see CL below)	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 50 % 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	None	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	Silt
		Medium to High	None	Medium	Inorganic clays of low to medium plasticity [1], gravely clays, sandy clays, silty clays, lean clays	CL [2]	Clay
		Low to Medium	Slow to Very Slow	Low	Organic slits and organic silty clays of low plasticity	OL	Organic Silt
		Low to Medium	Slow to Very Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	Silt
		High	None	High	Inorganic clays of high plasticity, fat clays	CH	Clay
		Medium to High	None	Low to Medium	Organic clays of medium to high plasticity	OH	Organic Silt
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	Peat

Notes:

1. Low Plasticity – Liquid Limit W_L < 35 % Medium Plasticity – Liquid limit W_L 35 to 60 % High Plasticity - Liquid limit W_L > 60 %.
2. 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

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 Substance In geotechnical engineering terms, rock substance is any naturally occurring aggregate of minerals and organic matter which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Rock substance is effectively homogeneous and may be isotropic or anisotropic.

Rock Defect Discontinuity or break in the continuity of a substance or substances.

Rock Mass Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

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	Soil derived from the weathering of rock. The mass structure and substance fabric are no longer evident. There is a large change in volume but the soil has not been significantly transported.
Extremely weathered[1]	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly weathered[2]	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decrease compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original rock substance is no longer recognisable.
Moderately weathered[2]	MW	Rock substance affected by weathering to the extent that staining extends throughout the whole of the rock substance and the original colour of the fresh rock is no longer recognisable.
Slightly weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh	FR	Rock substance unaffected by weathering

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	Is (50) MPa	Field Guide	Symbol
Very low	>0.03 ≤0.1	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VL
Low	>0.1 ≤0.3	A piece of 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	A piece of core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	M
High	>1 ≤3	A piece of core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife.	H
Very high	>3 ≤10	A piece of core 150mm long x 50mm diameter may be broken readily with hand held hammer. Cannot be scratched with pen knife.	VH
Extremely high	>10	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	EH

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

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

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)
- ● Unconfined 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.

Test, Drill and Excavation Methods

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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:

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

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

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

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.

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 the rod weight only
HW	Penetration occurred under the hammer and rod weight only
HB 30/80mm	Hammer double bouncing on anvil after 80 mm penetration
N=18	Where practical refusal occurs, report blows and penetration for that interval

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		