BELROSE RB1 Pty Ltd

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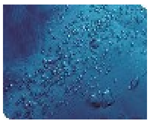
consulting engineers

Preliminary Geotechnical Assessment: 171 Forest Way, Belrose, NSW

ENVIRONMENTAL

WATER



WASTEWATER

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All enquiries regarding this project are to be directed to the Project Manager.

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Abbreviations

ABC – Allowable bearing capacity

BH – Borehole

CC – Construction Certificate

CFA – Continuous flight auger

DA – Development application

DBYD – Dial Before You Dig

DCP – Dynamic cone penetrometer

DP – Deposited plan

IA – Investigation area

kN – kilo Newtons

kN/m^3 – kilo Newtons per cubic metre

kPa – kilo Pascal

LGA – Local government area

MA – Martens & Associates Pty Ltd

mAHD – metres Australian height datum

mbgl – metres below ground level

MDD – Maximum Dry Density

MPa – Mega Pascal

1 Proposed Development and Investigation Scope

The proposed development details are summarised in Table 1.

Table 1: Summary of the proposed development.

Item	Details
Property Address	171 Forest Way, Belrose, NSW.
Lot/DP	Lot 9 DP 737255.
LGA	Northern Beaches Council.
Investigation area	The total investigation area is of 1.085 ha (Barry Rush, 202).
Proposed Development	The proposed development consists of: o Demolition of existing structures on site. o Construction of four new residential blocks, including basement car parking, requiring bulk excavation up to approximately 9.0 meters below ground level (mbgl). o Construction of a new private access roads. o Earthworks for preparation of development platforms, which may require limited filling (assumed less than 1.0 m). o Installation of services and other infrastructure. o Landscaping. The purpose of this geotechnical assessment is to support the Development Application (DA) for the proposed development.

2 Site Details and Subsurface Conditions

Table 2 summarises the general site details considered relevant to the investigation and the proposed development.

Table 2: Summary of site details and conditions.

Element	Description/Detail
Topography	The site is on slightly to moderately undulated land comprising multiple terraces and on the slope of an approximately north south aligned ridge.
Site Elevation	Ground level across the site ranges from approximately 127 mAHD (east) to 168 mAHD (west) (Barry Rush, 2021).
Typical Site Slope / Site Aspect	The site has easterly aspect generally slopes approximately less than 25%, with near-level terraces separated in parts by small steep grades. Existing rock outcrops and vertical escarpments at eastern corner of the site.
Investigation area description	At the time of the geotechnical investigation, the site consisted of brick and tile residence, timber shed, concrete septic tank, stockpile of old tyres, soil and metal pipes, storage sheds, a swimming pool and a bird aviary.
Surrounding land uses	The site is bounded by: o Residential lots to the south and north. o Forest Way to the west and bushland to the east.
Site Drainage	Via overland flow to the east, towards a tributary of Snake Creek approximately 30 m from the eastern site boundary (Figure 1, Attachment A).
Expected Geology Soil and Landscape	Hawkesbury Sandstone (Rh) comprising medium to coarse-grained quartz sandstone with minor shale and laminate lenses (Sydney 1:100,000 Geological Series Sheet 9130 (1983), 1st edition. Geological Survey of New South Wales). Somersby Soil Landscape consisting of moderately deep to deep red earths and yellow earths overlaying laterite gravels and clays on crests and upper slopes, yellow earths and earth sands on mid slopes, grey earths, leached sands and siliceous sand on lower slopes and drainage lines (Soil Landscapes of the Sydney 1:100,000 Sheet (1989) Soil Conservation Service of NSW)

3 Geotechnical Assessment

3.1 Field Investigation Scope of Works

A preliminary geotechnical assessment was previously undertaken by Martens and Associates Pty Ltd to support a development application (DA) on October 2016. The investigation primarily involved:

- Six boreholes (BH101 to BH106) to characterise subsurface materials and infer depth to top of rock. Borehole BH102 was hand augered to 0.8 metres below ground level (mBGL) due to drilling access restrictions. The remaining boreholes were drilled with a 4WD ute-mounted hydraulic rig using solid flight augers fitted with a V-shaped bit (V-bit) or tungsten-carbide bit (TC-bit), up to 2.3 mbgl.
- Six Dynamic Cone Penetrometer (DCP) test at the adjacent to the boreholes up to maximum depth of 1.8 mbgl to assess the near-surface soil consistency and assess the top of the rock.
- Collection of 2 bulk soil sample (CBR102 and CBR106) for laboratory California Bearing Ratio (CBR) and Atterberg Limits testing.

The findings and recommendations are presented in MA's report referenced P1605535JR01V02, dated October 2017. Results of the assessments have been reproduced in this report.

MA's engineer performed the site inspection on March 2021 to confirm existing site conditions. From the site inspection, it is observed that the site seems to be in similar condition as when previous geotechnical investigations was completed in 2017. Hence, the previous scope of geotechnical investigations is considered relevant for the site.

Investigation locations are shown in Figure 2, Attachment A.

3.2 Observed Subsurface Conditions

Table 3 summarises encountered subsurface materials and conditions, inferred from borehole and DCP test results, to investigation termination depth. Encountered conditions are described in more detail on borehole logs, Attachment B, and associated explanatory notes, Attachment G. For DCP test results refer to DCP 'N' counts in Attachment C.

Table 3: Generalised inferred subsurface profile to borehole termination depth.

Layer [1]	Depth (mBGL) [2]					
	BH101	BH102	BH103	BH104	BH105	BH106
FILL: SAND/Clayey SAND (trace of sandstone, gravel, bricks, very dense, moist) [8]	-	-	-	-	0.0 - 0.7	0.0 - 0.7
TOPSOIL: Silty SAND (dense, moist)	0.0 – 0.2	0.0 – 0.1	0.0 – 0.2	0.0 – 0.3	-	-
RESIDUAL SOIL: Clayey SAND (medium dense, moist)	-	-	-	0.3 – 0.9 [4]	-	-
RESIDUAL SOIL: SAND/Clayey SAND/ Gravelly SAND (dense to very dense, moist)	0.2 – 1.5 [4]	0.1 – 0.8 [3]	0.2 – 0.5 [4]	-	0.7 – 1.5 [4, 6]	0.7 – 1.9 [4, 7]
WEATHERED ROCK: SANDSTONE (inferred very low to low strength, distinctly weathered, moist/wet)	1.5 – 2.3 [7]	-	0.5 – 1.1 [5, 7]	0.9 – 1.4 [7]	1.5 – 1.9 [7]	-

Notes:

[1] Refer to borehole logs for more detailed material descriptions at test locations.

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

[3] Hand Auger refusal.

[4] V-bit refusal.

[5] Water inflow was observed at the depth of 0.8 mBGL.

[6] Wet soil encountered between 1.4 and 1.5 mBGL.

[7] TC-bit refusal in inferred medium to high strength sandstone. To be confirmed by further assessment.

[8] Assumed ‘uncontrolled’ fill.

Groundwater was not encountered in any of the boreholes. However the absence of groundwater within the boreholes does not preclude its presence even as ephemeral groundwater within the proposed depth of excavation.

3.3 Preliminary Material Properties

Preliminary material properties inferred from observations during borehole drilling, such as auger penetration resistance and DCP test results as well as engineering judgement are summarised in Table 4.

Table 4: Preliminary material properties.

Layer	$\gamma_{in\text{-}situ}$ [1] (kN/m^3)	Ø' [2] (deg)	E' [3] (MPa)
FILL: SAND/Clayey SAND (very dense, moist)	20	32	40
TOPSOIL: Silty SAND (dense, moist)	20	28	NA [5]
RESIDUAL SOIL: Clayey SAND (medium dense, moist)	18	30	20
RESIDUAL SOIL: SAND / Clayey SAND / Gravelly SAND (dense to very dense, moist)	19	35	50
WEATHERED ROCK: SANDSTONE (inferred very low to low strength, distinctly weathered, moist/wet) [4]	23	35	100

Notes:

1. Material in-situ unit weight, based on visual assessment (±10 %).
2. Effective internal friction angle (±2 °) assuming drained conditions; may be dependent on rock defect conditions.
3. Effective elastic modulus (±10 %).
4. Higher strength rock may be present below investigation termination depth.
5. Not Applicable.

3.4 Laboratory Testing

3.4.1 Atterberg Limits Testing

Laboratory testing was undertaken on two soil samples for the purpose of characterising encountered soil profiles. Testing was carried out by a National Association of Testing Authorities (NATA) accredited laboratory (Resource Laboratories). Table 5 presents a summary of test results. A laboratory test certificate is presented in Attachment D.

Table 5: Summary of Atterberg Limits laboratory test results.

BH	Depth (mBGL)	Soil Type	Atterberg Limits (%)			Plasticity Classification
			LL [1]	PL [1]	PI [1]	
BH102	0.12 - 0.7	Gravelly Sand	24	16	8	(CL) low plasticity Clay
BH106	0.7 - 1.2	Clayey Sand	18	15	3	(CL-ML) low plasticity inorganic clay/silt

Notes:

[1] LL = Liquid limit, PL= Plastic limit, PI=Plasticity index

Laboratory test results indicate that the tested soil samples are generally of low plasticity.

3.5 Risk of Slope Instability

Presence of detached boulders, colluviual slope and floaters observed during the site walkover survey indicate the risk of site slope instability. A detailed slope risk assessment in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (2007) was not undertaken.

The presence of rock outcrops and boulders at the site can cause instability during construction works and during the design life of the building. To reduce the level of risk of instability, the proposed development should be undertaken according to the recommendations provided in this report, including the following;

- In general, the design and construction of earthworks, foundations, retaining structures, excavation stabilisation and drainage measure for the proposed development should adhere to good engineering practice for hillside construction as set out in Appendix G of AGS 2007C guidelines, attached as Attachment F in this report.
- It is expected that some boulders, rock outcrops and rock overhangs may be broken up and removed from site as part of the site clearance works to allow excavation and subsequent construction of the development to safely take place. Breaking rocks into small pieces should be carried out in a manner that avoids rock pieces rolling down the slope.
- Prior to any construction work taking place, a scaling program should be undertaken to identify and remove any potentially unstable boulders or outcrops that were not removed as part of the initial site clearance mentioned above. This should include removal or battering back of any poorly consolidated soil/fill.

- Protective measures against rolling of rock boulders upslope of the site should be provided such as stabilisation by rock bolting, building rock-fall barriers or catch fences.
- Proposed excavations within the site should be accompanied by site observations by a geotechnical engineer and include monitoring for ground movement and vibration.
- Vibration levels should be monitored if methods of excavation adopted for excavation are likely to produce vibration intensities that may be detrimental to existing structures or triggering instability in the soils and rock within and adjacent to the site.
- Foundation systems for the building structures, retaining walls, etc. are to be founded and embedded into bedrock and where necessary designed for lateral earth pressures induced by translational soil movement along the interface between the soils and the underlying rock.
- Any cause of instability of the ground profile within the neighbouring properties should be addressed prior to commencement of excavation and proper stabilisation action needs to implement.
- Appropriate drainage measures should be incorporated to ensure all surface and subsurface water flows are diverted away from the slope into the stormwater drainage system or other appropriate discharge.
- Retaining walls and shoring should be constructed and supported in such a manner as not to induce instability that may be associated with construction procedures and sequencing or exposure of unsupported faces.
- Earth pressure coefficients for sloping ground should be adopted for design purposes as required.
- Construction activities should be carefully planned and observed by a geotechnical engineer for further assessment of the necessary mitigation and control measures.

It is recommended that a limited stability risk assessment be carried out for the site. The purpose of this assessment would be to assess the presence of adverse jointing, overhanging blocks, unstable wedges, etc. and the potential influence of the proposed development on its stability as well as any stabilisation measures that may be required to minimise the risk of rock falls as well as any mitigation measures that can be taken as part of the design and construction of the proposed works.

4 Hydrogeological Assessment

4.1 NSW Department of Primary Industries Water Bore Search

A review of the NSW Department of Primary Industries Water (DPIW) real time groundwater bore database revealed that there is one groundwater bore with relevant information located within 500 m of the site (denoted as GW014469 by DPIW, latitude: 33°43'35.3"S, longitude: 151°13'14.2"E - refer to Figure 1, Attachment A for approximate location). This groundwater bore is used for domestic/farming purposes, and groundwater is approximately 12.10 mBGL.

4.2 Groundwater Observations

Soils were encountered in a generally moist condition. Borehole BH105 became wet from 1.4 mBGL to the top of rock at 1.5 mBGL. Groundwater inflow was observed in borehole BH103 at 0.8 mBGL. Observed groundwater is inferred to be of ephemeral perched nature.

Ephemeral perched seepage water may be encountered in excavations, originating from surface water infiltration during prolonged or intense rainfall events. Excavations to 9.0 mBGL may encounter groundwater. Subject to results of further rock condition assessments, we consider a low groundwater inflow rate from defects within the rock profile.

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

5 Geotechnical Recommendations

5.1 General Recommendations

General geotechnical recommendations are provided in Attachment G. Specific recommendations are provided in the following sections for the proposed development.

5.2 Excavations

Shallow excavations will likely encounter topsoil / residual soils and fill. Weathered rock may also be encountered underlying the soil profile in some portions of the site. Deeper excavations for basements may encounter medium to high or higher strength rock. In light of this, excavations should be readily carried out as follows:

- For soils and extremely low to low strength rock conventional hydraulic tracked earthmoving equipment shall be used.

- For medium strength, or stronger rock, if encountered hydraulic earthmoving equipment with rock hammer attachment or ripping tyne shall be used.

All excavation work should be completed with reference to the Code of Practice 'Excavation Work' (most recent version), by Safe Work Australia and Warringah Council's Engineering Specifications (Warringah Council, 2004, AUS-SPEC-NSW-D2).

5.3 Vibrations

Where excavation is to be extended into medium or higher strength rock, care will be required when using a rock hammer to limit structural distress from excavation-induced vibrations. Consideration should be given to the use of rock sawing or fracturing techniques prior to using a hammer, particularly near boundaries.

To further limit vibrations, we recommend setting the rock hammer parallel to bedding planes and along defect planes, where possible and as advised by a geotechnical engineer.

We recommend limiting peak particle velocities (PPV) caused by construction equipment or resulting from excavation at the site to 5 mm/s (AS 2187.2, 1993, Appendix J). Higher values of 10 mm/s may be considered subject to further assessment by a geotechnical engineer. Exposed rock faces will need to be monitored to assess risk of block movement as a result of excavation vibrations.

We suggest that neighbours be made aware of timing of excavation works and expected vibration levels to allow securing of vibration sensitive items prior to the work.

5.4 Batter slopes

Soil overburden may be excavated without structural support but with a maximum temporary (less than 1 month) batter slope of 1 V (vertical): 2 H (horizontal) and permanent batter slope of 1 V: 3 H. For weathered rock, maximum grades of 1V:1.5H and 1V:2H can be adopted for temporary and permanent slopes, respectively. Vertical excavation may be carried out in medium and higher strength sandstone, should they be encountered, subject to 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 assessment.

Soil batters should be covered by plastic lining or protected by vegetation, and surface stormwater runoff should be diverted away from batter crests, to limit the risk of soil erosion and slope movement.

Excavated rock faces should be inspected during construction by a geotechnical engineer to determine whether any additional support, such as rock bolts or shotcrete or changes to batter angles are required. Unsupported long term vertical rock excavations should be covered by shotcrete to limit weathering. Allowances for installation of isolated rock bolts to support potential unstable rock blocks or weathered zones should be made.

Use of heavy machinery should be avoided, where possible, within 2 m of the crest of any open soil excavation to prevent excessive vibrations and undue settlement within exposed soils.

5.5 Temporary shoring / retaining structures

Where forming batters is considered not feasible, e.g. due to proximity of excavation to site boundaries or excessive earth work due to topography of the site, excavations into soil and weathered rock exceeding 0.75 m in height should be supported by suitably designed and installed temporary shoring and / or retaining structures to limit lateral deflection of excavation faces and associated ground surface settlements.

Temporary support of soil excavation faces may include steel I-beams, keyed into bedrock below bulk excavation level, with timber lagging.

Temporary shoring may be designed for inclusion as permanent retaining structures, e.g. adopting cast in situ reinforced concrete soldier piles. If necessary, reinforced mesh and shotcrete may need to be utilised to support remaining exposed areas e.g. impacted by rock defects / weathering.

Shoring / retaining wall design should consider additional surcharge loading from sloping ground behind walls, construction equipment, backfill compaction and static water pressures unless subsoil drainage is provided behind shoring structures / retaining walls.

5.6 Rock support

Steeply dipping joints and other rock defects may have an adverse effect on 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 anchors or bolts.

The presence of weakly cemented (extremely weathered) seams within the rock may require shotcreting and rock bolting.

Rock 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. Rock support should not extend beyond property boundaries unless approval has been granted by relevant property owners or stakeholders. 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. Martens and Associates can complete the necessary mapping and provide advice for possible remediation measures, where required.

5.7 Earthworks

Areas of the site, where existing ground levels are to be raised to reach the proposed development subgrade levels, may require site filling.

Filling should be carried out following removal of topsoil or fill and other unsuitable materials in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect the condition of the exposed material to assess suitability of the prepared surface as foundation for fill placement.

Fill material is to comprise approved imported granular fill material and should be placed in horizontal layers of not more than 300 mm loose thickness. However, the layer thickness should be appropriate for the compaction plant adopted. Site-won excavation spoil may be adopted for general fill placement.

Earthworks compliance testing should be carried out in accordance with Table 8.1 of AS3798 (2007), with testing to be provided by a NATA accredited testing authority.

For areas likely to be subjected to a loading of up to 20 kPa, fill material should be moisture conditioned and compacted to a minimum density index (DI) of 75% or density ratio (DR) of 98% SMDD, within 2% of Optimum Moisture Content (OMC). For areas loaded to greater than 20 kPa and under new pavements, the material should be moisture conditioned and compacted to a DI of 80% or DR of 100% SMDD, within 2% of OMC. For general fill areas, fill should be compacted to a minimum DI of 70% or DR of 95% SMDD and moisture conditioned to be within 2% of OMC.

5.8 Footings

It is expected that shallow footings, i.e. pad or strip footings founded on sandstone will be adopted as support for the proposed building. Where rock is not encountered at foundation level, footings should be extended, e.g. using bored cast in situ piles, to found on rock. All footings should found on material with similar end bearing capacity to limit differential movement across the building footprint. Individual pad footings should not span the interface between different foundation materials.

Geotechnical design parameters presented in Section 3.3 and 5.13, can be adopted for preliminary design of footings. These values should be confirmed by a geotechnical engineer during construction, as detailed in Section 7.2.

All footings should be constructed with minimal delay following excavation. Geotechnical Engineer is to confirm encountered conditions satisfy design assumptions and that the base of all excavations is free from loose or softened material and water prior to footing construction. Water that has ponded in the base of excavations and any resultant softened material is to be removed prior to footing construction. If a delay in construction is anticipated, a concrete blinding layer of at least 50 mm thickness should be placed to protect the foundation material of shallow footings.

5.9 Retaining wall and backfill and drainage

Backfill to conventional retaining walls should comprise engineered granular fill, free of organic material, contaminants and deleterious substances and a maximum particle size of 40mm. Backfill should be placed in maximum 100mm thick layers compacted using a hand held compactor. Care should be taken to ensure excessive compaction stresses are not transferred to retaining walls.

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 retaining walls to a suitable discharge path downslope of the site.

5.10 Surface and Groundwater

It is considered that the proposed excavation will likely encounter limited seepage inflow. Sump and pump methods are considered to be appropriate for dewatering during construction and in the long term. We recommend monitoring of flow during the early phases of excavation to assess potential long-term pumping requirements. Groundwater ingress should be monitored during excavation by a geotechnical engineer.

All surface runoff should be diverted away from excavation areas during construction works and from any retaining structures, footings or the crest and base of embankments to prevent water accumulation, foundation / embankment material strength reduction and pore water pressure increases.

All site discharges should be passed through a filter material prior to release into the Council stormwater system or approved alternative.

5.11 Soil erosion

Soil overburden should be removed in a manner that reduces the risk of sedimentation of natural drainage channels. Spoil on site should be properly controlled by erosion control measures to prevent transportation of sediments off-site. The following erosion control measures should be considered, in conjunction with recommendation by Landcom (2004), to limit surface run-off and associated risk of surface scour, soil erosion and sedimentation:

- Maintain vegetation where possible.
- Disturb minimal area during excavation.
- Landscape disturbed areas following completion of constructions.
- Use gabion mattress, or other suitable energy reduction solutions, where required.
- Direct water away from structures.

5.12 Off-site removal of excavation spoil and groundwater

Soil to be disposed off-site should be classified in accordance with the NSW EPA/DECCW guidelines. Groundwater should also be tested prior to discharge to ensure contaminant levels (if applicable) are appropriate for discharge locations. MA can complete the necessary classification and testing if required. Time allowance should be made for such testing in the construction program.

5.13 Preliminary Design Parameters

Preliminary design parameters for footing and retaining wall design are presented in Table 6. These have been estimated from field test results in conjunction with borehole derived soil profile data, where available.

Table 6: Preliminary footing design parameters.

Layer	Shallow Footings	Piles [2]		K_a[5]	K_p[5]
	AEB [1, 4]	AEB [1, 4]	ASF [3, 4]		
FILL: SAND/Clayey SAND - TOPSOIL: Silty SAND (medium to dense)	NA [7]	NA [7]	NA [7]	0.27	3.60
RESIDUAL: Clayey SAND (medium dense)	80	NA [7]	NA [7]	0.31	3.26
RESIDUAL SOIL: SAND/ Clayey SAND/ Gravelly SAND (dense to very dense)	300 [6]	NA [7]	NA [7]	0.22	4.5
WEATHERED ROCK: SANDSTONE (inferred very low to low strength)	450	700	60	NA[7]	NA[7]
WEATHERED ROCK: SANDSTONE (inferred medium to high strength)	1000	1500	250	NA [7]	NA [7]

Notes:

[1] Allowable end bearing pressure (kPa) for shallow footings embedded at least 0.5 m and piles embedded at least 1m or 1 pile diameter, whichever is greater, into the design material type.

[2] Assuming bored, cast in-situ concrete pile.

[3] Allowable skin friction (kPa) 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] AEB 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 and to 25 mm for a large single pad footing.

[5] K_a = Coefficient of active earth pressure; K_p = Coefficient of passive earth pressure.

[6] Assuming high level structures supported by square footing with $D_f/B < 0.5$ and $D_f > 0.5$ m.

[7] Not applicable.

5.14 Site Classification

Subject to shallow footings founding in residual soil, we recommend adopting a preliminary site classification of 'S' for design of high level and lightly loaded shallow footings in accordance with AS 2870 (2011).

A revised classification of 'A' may be adopted should footings extend through topsoil / fill and residual soils and be founded on rock.

5.15 Site Drainage

Surface water run-off should be diverted away from the proposed building platform. Ponding and infiltration of surface water should be prevented to limit the impact of associated soil softening and degradation of exposed rock.

All site discharges should be passed through a filter material prior to release. Diverted flows should be directed (where possible) to a suitable stormwater system downslope of the site so as to prevent water accumulating in areas surrounding retaining structures and footings.

6 Preliminary Pavement Thickness Design

6.1 Overview

The preliminary pavement thickness design was undertaken for potential new access roads, in accordance with Northern Beaches Council's DCP and Austroads (2012) *Guide to Pavement Technology Part 2 Pavement Structural Design*.

6.2 Design Parameters

A traffic loading of 5×10^4 Equivalent Standard Axles (ESA) was adopted for the proposed private access roads.

Two bulk soil samples were collected from BH102 and BH106, and submitted to Resource Laboratories for CBR testing. A four day soaked CBR test was conducted in accordance with AS 1289.1.1, 2.1.1, 5.1.1 and 6.1.1. Test results are summarised in Table 7. A laboratory test certificate is provided in Attachment D.

Table 7: Laboratory CBR test results.

Sample Number	Material	Sample Depth (mBGL)	CBR [1] Value
BH102/0.12-0.7/S/1	Gravelly Sand	0.12 – 0.70	20
BH106/0.7-1.2/S/1	Clayey Sand	0.70 – 1.20	25

Notes:

[1] Four day soak, compacted to 98 % SMDD (± 2 % of OMC), applying a 4.5 kg surcharge.

Based on correlations between CBR and DCP test results (Austroads, 2012), a CBR value of between 3.5 and 42 applies to sand / silty sand / clayey sand. However, we note that the soils were encountered in a generally moist condition and that higher values reflect conditions near top of rock. Correlated CBR values should be reduced when considering long-term ground conditions.

For the purpose of this assessment, a CBR value of 10 % was adopted, considered typical for medium to dense sand encountered in the site area, and considering council design requirement.

Additional CBR testing is recommended to provide a better indication of subgrade conditions across pavement areas considering final design alignments and levels, to confirm suitability of adopted CBR value and / or provide statistical means to support a higher CBR design value. The additional testing may be undertaken at Construction Certification stage.

6.3 Pavement Thickness

Table 8 presents preliminary recommended pavement material thicknesses for proposed private access roads.

Table 8: Preliminary pavement material thickness design for CBR of 10 %.

Road Type	Layer	Thickness (mm)
Private Access Roads	Asphaltic concrete wearing course (1 x 25 mm layer of AC10 over 7 mm of primer seal)	32 [1, 2]
	Base (DGB 20, or similar)	100 [3]
	Sub-base (DGS 40, or similar)	125
	Total pavement thickness	257

Notes:

[1] Based on Warringah Council (2004), AUS-SPEC-NSW-D2, *Pavement Design*.

[2] Impact of turning or stopping vehicles at end of road or intersections not included in assessment.

[3] Minimum based on AUS-SPEC-NSW-D2, *Pavement Design*.

7 Proposed Additional Assessments

7.1 Proposed Additional Assessment

We recommend the following additional assessments are carried out during development of final design and prior to issuing of a construction certificate to better manage geotechnical risks, where applicable:

- Undertake rock coring at 3 – 4 locations for the assessment of foundation condition up to at least 2 m below final bulk excavation and foundation levels, as applicable.

- Limited stability risk assessment should be carried out to assess adverse jointing, overhanging blocks, unstable wedges, etc.

- Laboratory testing of soil and rock, as necessary, for more accurate assessment of subsurface conditions at future dwelling and infrastructure locations and of associated design parameters to confirm or alter preliminary site classifications and design assumptions. This should include conducting point load testing on rock samples.

- Review of final design and construction staging plans by a geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of recommendations provided in this report.

- Preparation of geotechnical monitoring program.

- Geotechnical inspection should be carried out during excavation in rock at maximum 1.5 meter depth intervals to assess adverse jointing etc. and stabilisation measures as required.

- Assessment of site specific foundation material capacity to support adopted footing types.

- In-situ testing during fill placement to ensure acceptable level of compaction is achieved, if required.

7.2 Proposed Monitoring and Inspection Program

To maintain site stability during site works and limit adverse geotechnical impacts on the site and surrounding areas as a result of the proposed development, we recommend the following is inspected and monitored (Table 9) during site works. This program may be updated following further detailed investigations.

Table 9: Recommended inspections/monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall, anchor, rock bolt) installations and monitor associated performance, if applicable.	Daily / As required [2]	Builder / MA [1]
Inspect exposed un-retained excavation sides to monitor performance and assess additional support requirements, if necessary.	Per each 1.5 m excavation lift / As required.	MA [1]
Monitor groundwater seepage from excavation faces, where encountered, to assess adequacy of drainage provision.	When encountered	Builder / MA [1]
Monitor excavation-induced vibrations, if required.	Daily at on-set of excavation and as agreed thereafter [2]	MA [1]
Monitor settlement and lateral deflection of retained materials along site boundaries, if required.	Daily at on-set of excavation and as agreed thereafter	Builder / MA [1]
Monitor sedimentation downslope of excavated areas.	During and after rainfall events	Builder
Monitor sediment and erosion control structures to assess adequacy and for removal of built up spoil.	After rainfall events	Builder
Inspect exposed material to verify suitability as foundation/ lateral support/ subgrade.	Prior to reinforcement set-up and concrete placement for footing construction and fill or pavement material placement.	MA [1]
Proof roll pavement subgrade and pavement materials	As required [2]	MA [1]
Inspect fill material to verify suitability for fill placement at the site and for provision of advice associated with fill placement.	Prior to fill / pavement placement.	MA [1]

Notes:

[1] MA = Martens and Associates Geotechnical Engineer.

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

7.3 Contingency Plan

In the event that the proposed development works cause an adverse impact on overall site stability or on neighbouring properties, works shall cease immediately. The nature of the impact shall be documented and the reason(s) for the adverse impact investigated. This might require site inspection by a qualified geotechnical or structural engineer.

8 Limitations

The recommendations presented in this report are based on limited preliminary investigations and include specific issues to be addressed during the design and construction phases of the project. In the event that any of the recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Martens & Associates accept no responsibility whatsoever for the performance of the works undertaken where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, subsurface 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 are based on interpolation between spot levels from the survey plan prepared by Barry Rush & Associates Pty Ltd, drawing no. A02, dated 21 February 2018. These values may not be accurate and should be confirmed by onsite survey.

9 References

Australia Standard 1289.6.3.2 (AS, 1997) *Determination of the penetration resistance of a soil - 9kg dynamic cone penetrometer test.*

Australia Standard 1726 (AS, 2017) *Geotechnical site investigations*.

Australia Standard 2870 (AS, 2011) Australia Standard, *Residential slabs and footings*.

Australia Standard 3600 (AS, 2009) *Concrete structures*.

Australia Standard 3798 (AS, 2007) *Guidelines on earthworks for commercial and residential developments.*

Austroads (2012) *Guide to Pavement Technology, Part 2 Pavement Structural Design.*

Barry Rush and Associates Pty Ltd architectural drawings, Nos. A01 to A26, Job No. 2101, dated 8 December 2021.

CSIRO BTF 18 (2003) *Foundation Maintenance and Footing Performance: A homeowner's Guide*.

Herbert C. (1983) Sydney 1:100 000 Geological Series Sheet 9130, 1st edition, Geological Survey of New South Wales Department of Minerals and Energy.

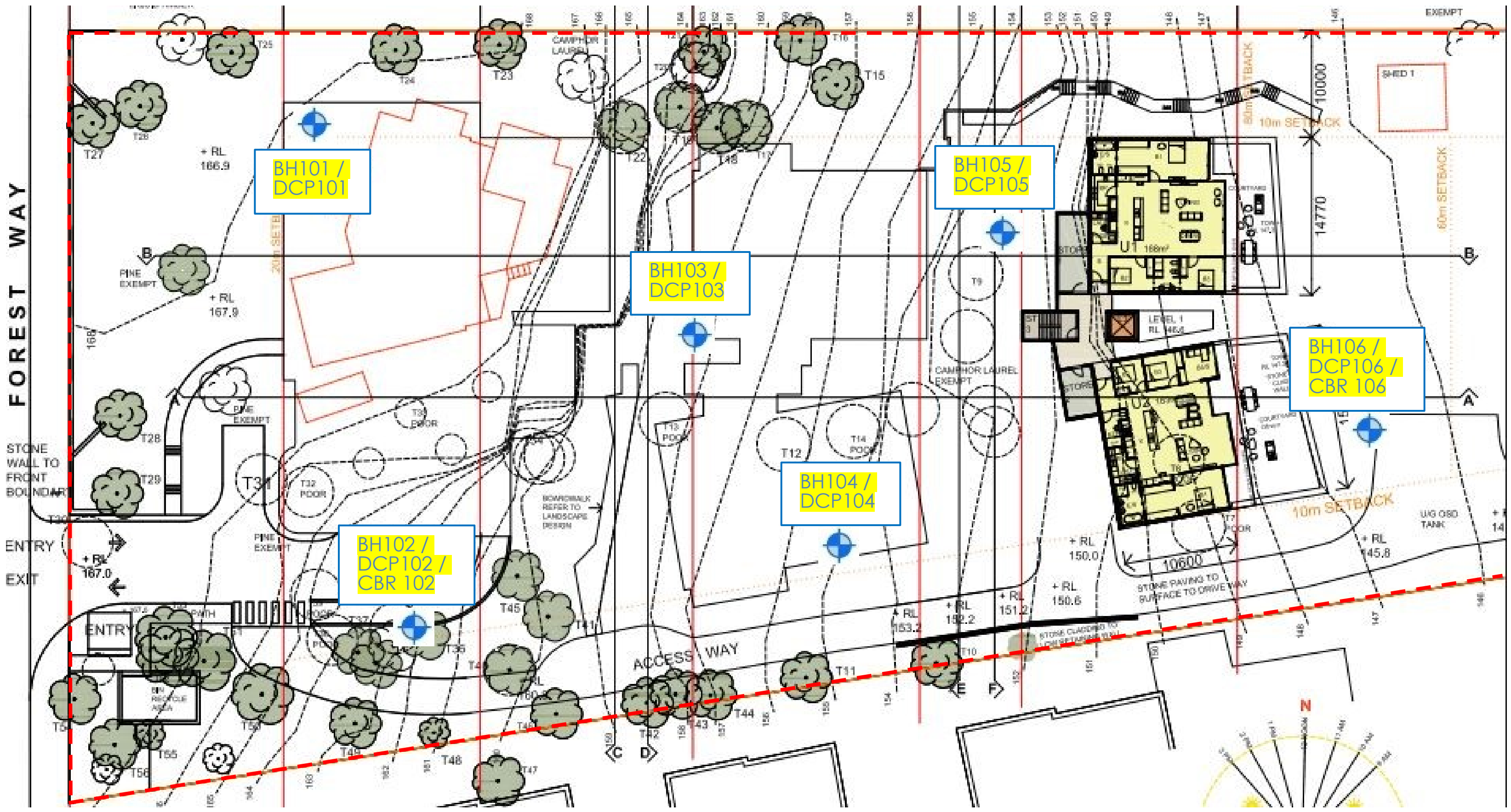
Landcom (2004) *Managing Urban Stormwater: Soils and Construction*.

Warringah Council (2004), AUS-SPEC-NSW-D2, *Pavement Design.*

Warringah Council (2011), *Development Control Plan*, Part E10.

Attachment A – Figures

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	AG	**SITE AND CREEK LOCATION** **171 Forest Way, Belrose, NSW** **(Source: Nearmap)**	**FIGURE 1**
Approved:	KB		
Date:	19.03.2021		
Scale:	Not to Scale		Job No: P2108124JR02V01



Key:

🟥 Approximate site boundary

⊕ Indicative borehole, DCP test and CBR sampling location

Martens & Associates Pty Ltd ABN 85 070 240 890

Environment | Water | Wastewater | Geotechnical | Civil | Management

Drawn:	AG
Approved:	KB
Date:	11.07.2022
Scale:	Not to Scale

GEOTECHNICAL INVESTIGATION PLAN
171 Forest Way, Belrose, NSW
(Source: Barry Rush, 2021)

Drawing:
FIGURE 2

File No: P2108124JR02V01

Attachment B – Borehole Logs

CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH101**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet	1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124	
EQUIPMENT	4WD ute- mounted hydraulic drill rig	EASTING		RL SURFACE	168 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 2.30 m depth	NORTHING		ASPECT	East	SLOPE	

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L	Not Encountered		168.00 0.20				SM	Silty SAND, fine to medium grained, dark brown.	M	D	TOPSOIL
	M H		0.5 1.0	167.80 1.50				SP	SAND, medium grained, pale brown, with some clay.		D VD	RESIDUAL SOIL
AD/T	L H		1.5 2.0	166.50 2.30					SANDSTONE, orange/brown, inferred very low to low strength, distinctly weathered, with inferred extremely low strength band between 1.9 - 2.1 m.			WEATHERED ROCK 1.50: V-bit refusal.
			2.5 3.0 3.5 4.0 4.5						Hole Terminated at 2.30 m			2.30: TC-bit refusal.

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

martens

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P1605535 BH01V01 161025.GPJ <<DrawingFile>> 23/11/2016 09:04 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Martens 2.00 2016-11-13 Prj: Martens 1.01.5 2015-12-17

CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH102**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet	1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124	
EQUIPMENT	Hand Auger	EASTING		RL SURFACE	162 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø85 mm x 0.80 m depth	NORTHING		ASPECT	East	SLOPE	

Drilling					**Sampling**		**Field Material Description**					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	*DEPTH* RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
HA	L	Not Encountered		162.00 *0.10*				SM	Silty SAND, fine to medium grained, dark brown.	M	D	TOPSOIL
	M		0.5	161.90	5535/BH102/0.1/S/1 D 0.10 m			SP	Gravelly SAND, fine to coarse grained, pale brown, fine to coarse sandstone gravels, with clay, trace silt.			RESIDUAL SOIL
	H			*0.80*							VD	
			1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5						Hole Terminated at 0.80 m			0.80: Hand auger refusal on inferred very low to low strength sandstone.

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CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH103**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124
EQUIPMENT	4WD ute- mounted hydraulic drill rig	EASTING		RL SURFACE	157 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.10 m depth	NORTHING		ASPECT	East	SLOPE

Drilling					**Sampling**		**Field Material Description**					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L			157.00 0.20			▨	SM	Silty SAND, fine to medium grained, dark brown.			TOPSOIL
	M			156.80 0.50				SC	Clayey SAND, medium grained, grey.	M	D	RESIDUAL SOIL
AD/T	M H	Water inflow ▽	0.5 1.0	156.50 1.10					SANDSTONE, grey mottled pink, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 0.50: V-bit refusal.
			1.5 2.0 2.5 3.0 3.5 4.0 4.5						Hole Terminated at 1.10 m			1.10: TC-bit refusal on inferred medium to high strength sandstone.

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CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH104**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet	1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124	
EQUIPMENT	4WD ute- mounted hydraulic drill rig	EASTING		RL SURFACE	153 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.40 m depth	NORTHING		ASPECT	East	SLOPE	

Drilling					**Sampling**		**Field Material Description**					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L	Not Encountered		153.00 0.30				SM	Silty SAND, fine to medium grained, dark brown.	M	D	TOPSOIL
	M		0.5	152.70 0.90				SC	Clayey SAND, medium grained, grey.		MD	RESIDUAL SOIL
AD/T	H		1.0	152.10 1.40					SANDSTONE, grey mottled pink, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 0.90: V-bit refusal.
			1.5 2.0 2.5 3.0 3.5 4.0 4.5						Hole Terminated at 1.40 m			1.40: TC-bit refusal on inferred medium to high strength sandstone.

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CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH105**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124
EQUIPMENT	4WD ute- mounted hydraulic drill rig	EASTING		RL SURFACE	150 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.90 m depth	NORTHING		ASPECT	East	SLOPE

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L	Not Encountered	0.5	150.00 0.70				SC	FILL: Clayey SAND, fine to medium grained, brown, trace sandstone gravel, bricks and plastic.	M	VD	FILL
	M		1.0	149.30 1.50				SC	Clayey SAND, medium grained, grey.	W	D	RESIDUAL SOIL
AD/T	H		1.5	148.50 1.90					SANDSTONE, grey mottled pink, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 1.50: V-bit refusal.
			2.0 2.5 3.0 3.5 4.0 4.5						Hole Terminated at 1.90 m			1.90: 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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CLIENT	BELROSE RB1 Pty Ltd	COMMENCED	14/10/2016	COMPLETED	14/10/2016	**REF BH106**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	171 Forest Way, Belrose, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2108124
EQUIPMENT	4WD ute- mounted hydraulic drill rig	EASTING		RL SURFACE	146 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.90 m depth	NORTHING		ASPECT	East	SLOPE

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L	Not Encountered	0.5	146.00				SP	FILL: SAND, fine to medium grained, pale brown, trace sandstone gravels.			FILL
	M		1.0 1.5	0.70 145.30 1.90	5535/BH106/0.7-1.2/S/1 D 0.70 m			SC	Clayey SAND, medium grained, grey, trace sandstone gravel. Clay band (approximately 100mm) between 1.5-1.6m. mottled red and orange and pink below 1.8m.	M	D - VD VD	RESIDUAL SOIL
			2.0 2.5 3.0 3.5 4.0 4.5						Hole Terminated at 1.90 m			1.90: V-bit and TC-bit refusal on medium to high strength sandstone.

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

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Attachment C – DCP ‘N’ Counts

Dynamic Cone Penetrometer Test Log Summary

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Site	171 Forest Way, Belrose, NSW	**DCP Group Reference**	P2108124JS01V01
Client	BELROSE RB1 Pty Ltd	**Log Date**	14.10.2016
Logged by	AT		
Checked by	RE		
Comments			

TEST DATA

Depth Interval (m)	**DCP101**	**DCP102**	**DCP103**	**DCP104**	**DCP105**	**DCP106**		**Depth Interval (m)**
0.15	10	10	8	8	17	8		**0.15**
0.30	8	10	12	6	14	9		**0.30**
0.45	6	12	12	5	14	7		**0.45**
0.60	7	12	Terminated at 0.45 mBGL due to bounce	3	10	8		**0.60**
0.75	18	26		6	12	13		**0.75**
0.90	40	Terminated at 0.75 mBGL due to bounce		15/10cm	9	6		**0.90**
1.05	Terminated at 0.90 mBGL due to high 'N' count.			Terminated at 0.85 mBGL due to bounce	10	11		**1.05**
1.20					7	6		**1.20**
1.35					13	16		**1.35**
1.50					10/5cm	8		**1.50**
1.65					Terminated at 1.40 mBGL due to high 'N' count.	14		**1.65**
1.80						14		**1.80**
						Terminated at 1.80 mBGL due to bounce		

Attachment D – Laboratory Test Certificates

resource laboratories
AGGREGATE, ROCK, AND SOIL TESTING
ABN: 25 131 532 020
Sydney: 12/1 Boden Road Seven Hills NSW 2147 | PO Box 45 Pendle Hill NSW 2145
Ph: (02) 9674 7711 | **Fax:** (02) 9674 7755 | **Email:** info@resourcelab.com.au

Test Report

Customer: Martens & Associates Pty Ltd
Project: P1605535
Location: 171 Forest Way, Belrose, NSW

Job number: 16-0099
Report number: 1
Page: 1 of 1

Soil Index Properties

Sampling method: Samples tested as received

Test method(s): AS 1289.1.1, 2.1.1, 3.1.1, 3.2.1, 3.3.1

	Results				
Laboratory sample no.	9796	9797			
Customer sample no.	8124/106/ 0.7-1.2/S/1	8124/102/ 0.12-0.7/S/1			
Date sampled	14/10/2016	14/10/2016			
Material description	SILTY SAND, trace of gravel, pale grey/brown	SILTY SAND, with gravel, brown			
Liquid limit (%)	18	24			
Plastic limit (%)	15	16			
Plasticity index (%)	3	8			
Linear shrinkage (%)	-	-			
Cracking / Curling / Crumbling	-	-			
Sample history	Air dried	Air dried			
Preparation	Dry sieved	Dry sieved			

Approved Signatory: E. Maldonado

Date: 26/10/2016

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025.

NATA Accredited Laboratory Number: **17062**

resource laboratories
AGGREGATE, ROCK, AND SOIL TESTING
ABN: 25 131 532 020
Sydney: 12/1 Boden Road Seven Hills NSW 2147 | PO Box 45 Pendle Hill NSW 2145
Ph: (02) 9674 7711 | **Fax:** (02) 9674 7755 | **Email:** info@resourcelab.com.au

Test Report

Customer: Martens & Associates Pty Ltd
Job number: 16-0099
Project: P1605535
Report number: 2
Location: 171 Forest Way, Belrose, NSW
Page: 1 of 2

California Bearing Ratio

Sampling method: Samples tested as received
Test method(s): AS 1289.1.1, 2.1.1, 5.1.1, 6.1.1

	Results			
Laboratory sample no.	9796	9797		
Customer sample no.	8124/106/ 0.7-1.2/S/1	8124/102/ 0.12-0.7/S/1		
Date sampled	14/10/2016	14/10/2016		
Material description	SILTY SAND, trace of gravel, pale grey/brown	SILTY SAND, with gravel, brown		
Maximum dry density (t/m^3)	1.91	1.89		
Optimum moisture content (%)	13.2	12.5		
Field moisture content (%)	n/a	n/a		
Oversize retained on 19.0mm sieve (%)	0	3		
Oversize included (Y/N)	N	N		
Dry density before soak (t/m^3)	1.88	1.85		
Dry density after soak (t/m^3)	1.88	1.85		
Moisture content before soak (%)	12.8	12.5		
Moisture content after soak (%)	13.7	13.9		
Moisture content after test - top 30mm (%)	13.4	13.6		
Moisture content after test - remaining depth (%)	13.3	13.4		
Density ratio before soaking (%)	98.5	98.0		
Moisture ratio before soaking (%)	96.5	100.0		
Period of soaking (days)	4	4		
Compactive effort	Standard	Standard		
Mass of surcharge applied (kg)	4.5	4.5		
Swell after soaking (%)	0.0	0.0		
Penetration (mm)	5.0	5.0		
CBR Value (%)	**25**	**20**		

Notes: Specified LDR: 98 ±1%

Approved Signatory: E. Maldonado

Date: 31/10/2016

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NATA Accredited Laboratory Number: **17062**

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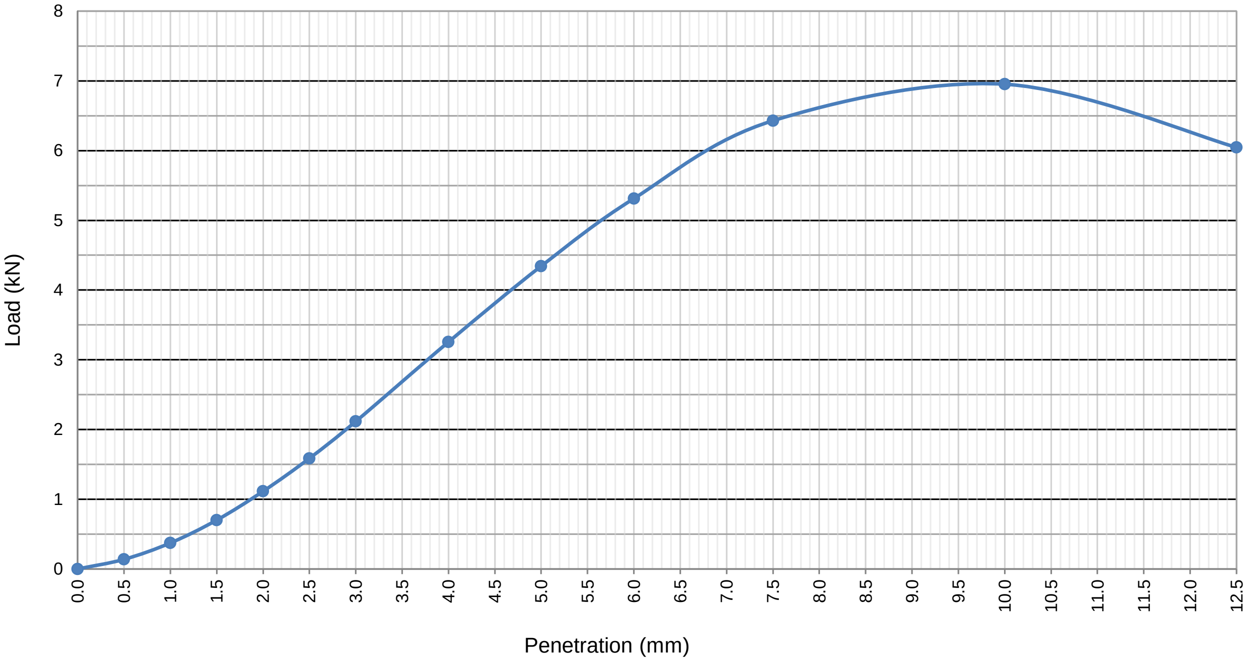
Ph: (02) 9674 7711 | **Fax:** (02) 9674 7755 | **Email:** info@resourcelab.com.au

Test Report

Customer:	Martens & Associates Pty Ltd	**Job number:**	16-0099
Project:	P2108124	**Report number:**	2
Location:	171 Forest Way, Belrose, NSW	**Page:**	2 of 2

CBR Load Penetration Curve

Sampling method:	Samples tested as received	**Test method(s):**	AS 1289.1.1, 2.1.1, 5.1.1, 6.1.1
Date sampled:	14/10/2016	**Laboratory sample no.:**	9796
Material description:	SILTY SAND, trace of gravel, pale grey/brown	**Customer sample no.:**	8124/106/0.7-1.2/S/1



Approved Signatory: E. Maldonado **Date:** 31/10/2016

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Attachment E – CSIRO Sheet BTF 18

Foundation Maintenance and Footing Performance: A Homeowner’s Guide

PUBLISHING

BTF 18-2011 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870-2011, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil’s lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume, particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.

In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

Notes

1. Where controlled fill has been used, the site may be classified A to E according to the type of fill used.
2. Filled sites. Class P is used for sites which include soft fills, such as clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soil subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise.
3. Where deep-seated moisture changes exist on sites at depths of 3 m or greater, further classification is needed for Classes M to E (M-D, H1-D, H2-D and E-D).

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

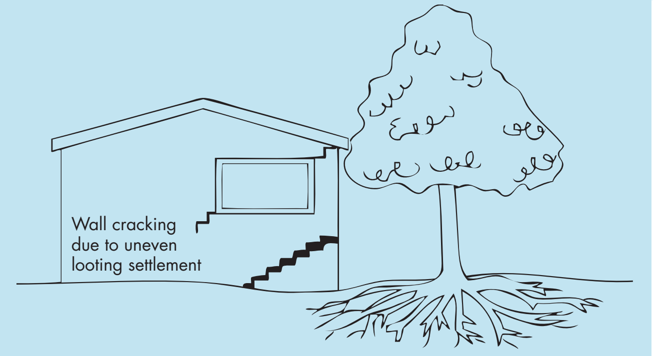
Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Trees can cause shrinkage and damage



Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures

Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation causes a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures

Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem. Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870-2011.

AS 2870-2011 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing

Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage

In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

Protection of the building perimeter

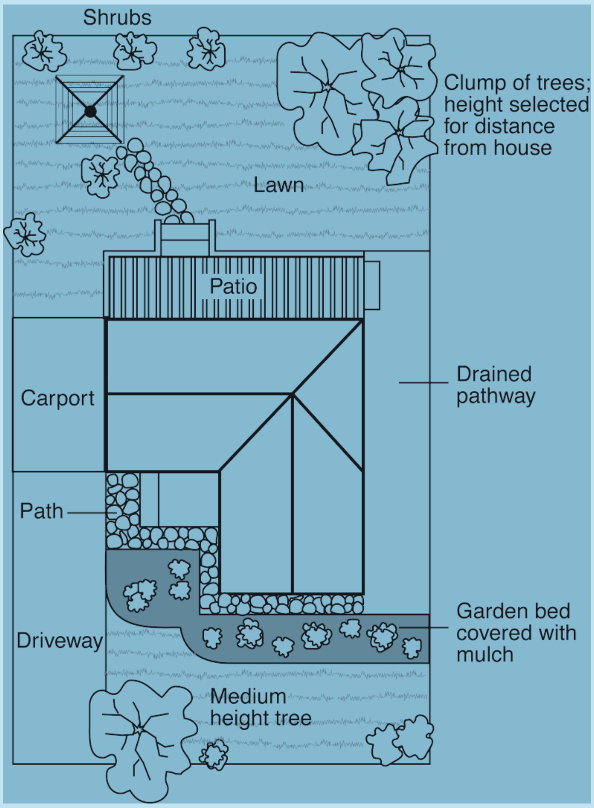
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving should

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly.	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired.	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted.	15–25 mm but also depends on number of cracks	4

Gardens for a reactive site



extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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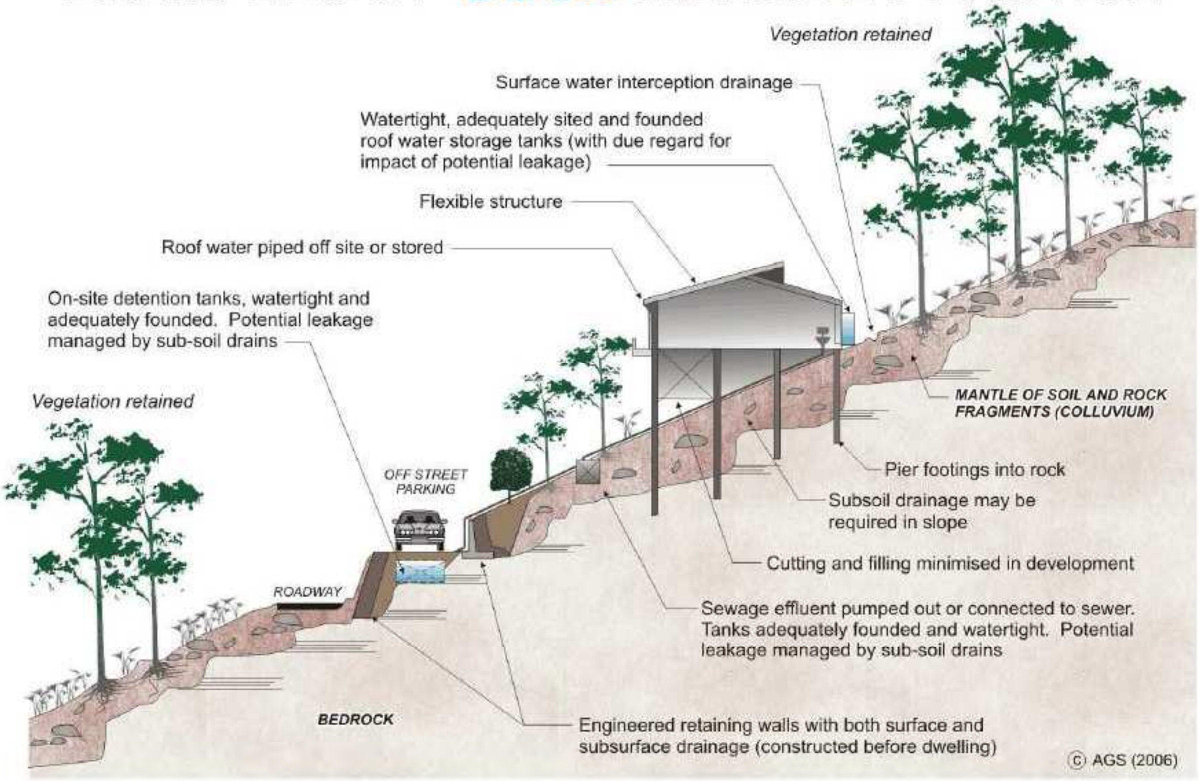
Attachment F – Hillslide Construction Guideliens (AGS, 2007)

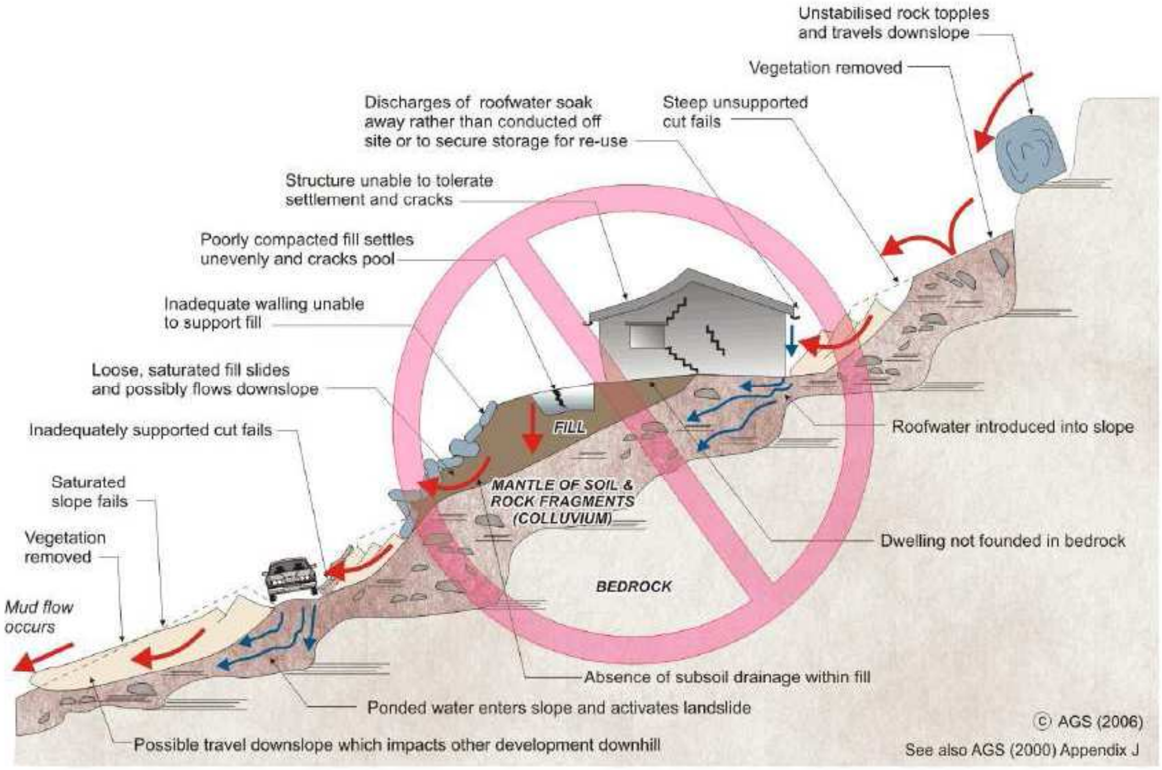
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	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



Attachment G – General Geotechnical Recommendations

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.

Attachment H – Notes Relating To This Report

Important Information About Your Report (1 of 2)

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

Engineering Reports - Limitations

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

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

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

Engineering Reports – Project Specific Criteria

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

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

Engineering Reports – Recommendations

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

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

Engineering Reports – Use for Tendering Purposes

Where information obtained from investigations is provided for tendering purposes, Martens recommend that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document.

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

Engineering Reports – Data

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

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

Engineering Reports – Other Projects

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

Subsurface Conditions - General

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

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

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

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

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

Subsurface Conditions - Changes

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

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

Subsurface Conditions - Site Anomalies

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

Report Use by Other Design Professionals

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

Subsurface Conditions – Geo-environmental Issues

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

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

Responsibility

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

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

Site Inspections

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

Soil Data

Explanation of Terms (1 of 3)

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.

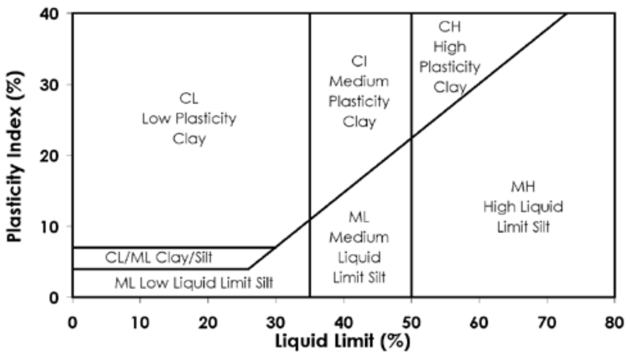
Particle Size

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

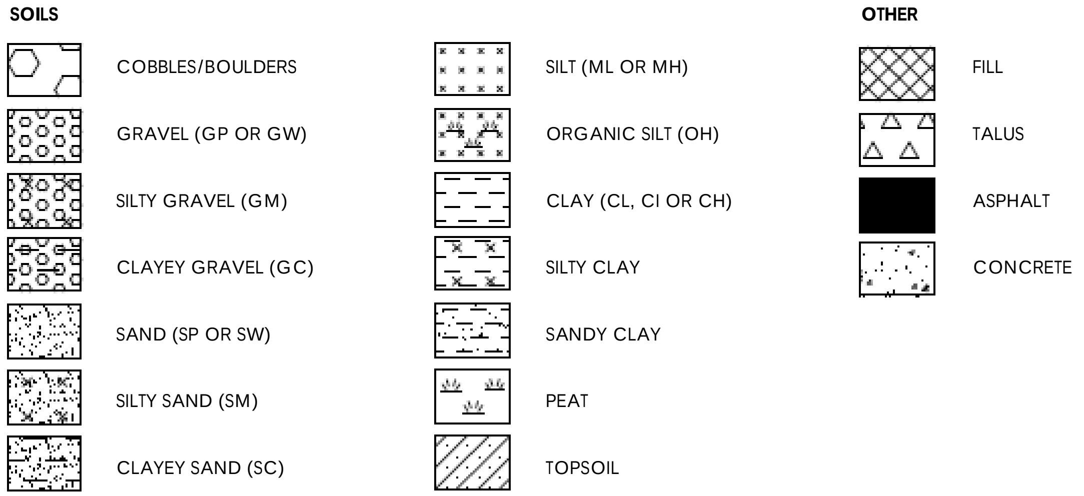
Minor components in soils may be present and readily detectable, but have little bearing on general geotechnical classification. Terms include:

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

Explanation of Terms (2 of 3)

Symbols for Soils and Other



Unified Soil Classification Scheme (USCS)

<table>
<tr><td colspan="6">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 63 mm and basing fractions on estimated mass)</td><td>USCS</td><td>Primary Name</td></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than 50 % of material less than 63 mm is larger than 0.075 mm</td><td rowspan="16">(A 0.075 mm particle is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than 2.0 mm.</td><td rowspan="2">CLEAN GRAVELS (Little or no fines)</td><td colspan="2">Wide range in grain size and substantial amounts of all intermediate particle sizes.</td><td>GW</td><td>Gravel</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with more intermediate sizes missing</td><td>GP</td><td>Gravel</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES (Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty Gravel</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey Gravel</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than 2.0 mm</td><td rowspan="2">CLEAN SANDS (Little or no fines)</td><td colspan="2">Wide range in grain sizes and substantial amounts of intermediate sizes missing.</td><td>SW</td><td>Sand</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td><td>Sand</td></tr>
<tr><td rowspan="2">SANDS WITH FINES (Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>SM</td><td>Silty Sand</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>SC</td><td>Clayey Sand</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
More than 50 % of material less than 63 mm is smaller than 0.075 mm</td><td colspan="6">IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM</td></tr>
<tr><td>DRY STRENGTH (Crushing Characteristics)</td><td>DILATANCY</td><td>TOUGHNESS</td><td>DESCRIPTION</td><td>USCS</td><td>Primary Name</td></tr>
<tr><td>None to Low</td><td>Quick to Slow</td><td>None</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td><td>ML</td><td>Silt</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Medium</td><td>Inorganic clays of low to medium plasticity [1], gravely clays, sandy clays, silty clays, lean clays</td><td>CL [2]</td><td>Clay</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low</td><td>Organic slits and organic silty clays of low plasticity</td><td>OL</td><td>Organic Silt</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low to Medium</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td><td>MH</td><td>Silt</td></tr>
<tr><td>High</td><td>None</td><td>High</td><td>Inorganic clays of high plasticity, fat clays</td><td>CH</td><td>Clay</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Low to Medium</td><td>Organic clays of medium to high plasticity</td><td>OH</td><td>Organic Silt</td></tr>
<tr><td colspan="2">HIGHLY ORGANIC SOILS</td><td colspan="4">Readily identified by colour, odour, spongy feel and frequently by fibrous texture</td><td>Pt</td><td>Peat</td></tr>
</table>

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.

Soil Agricultural Classification Scheme

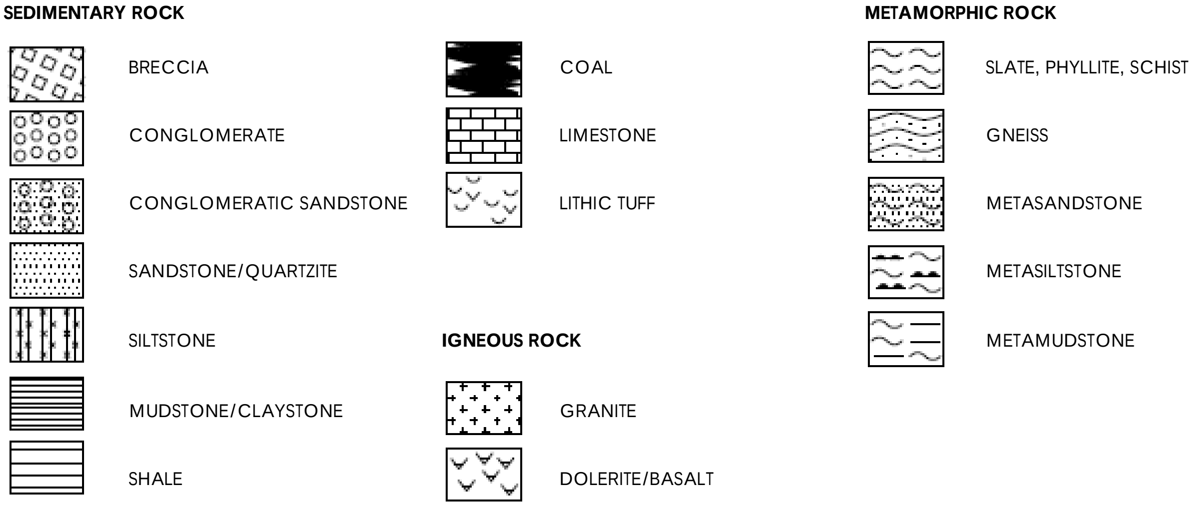
In some situations, such as where soils are to be used for effluent disposal purposes, soils are often more appropriately classified in terms of traditional agricultural classification schemes. Where a Martens report provides agricultural classifications, these are undertaken in accordance with descriptions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

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

Rock Data

Explanation of Terms (1 of 2)

Symbols for Rock



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

Rock Data

Explanation of Terms (2 of 2)

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

Explanation of Terms (1 of 3)

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.

Test, Drill and Excavation Methods

Explanation of Terms (3 of 3)

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	◁	Partial water loss
▷	Water inflow	◀	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		