Universal Property Group Pty Ltd

Preliminary Salinity and Geotechnical Assessment: 326-330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P1605186JR02V01
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Head Office
Suite 201, 20 George Street
Hornsby, NSW 2077, Australia
ACN 070 240 890 ABN 85 070 240 890
Phone: +61-2-9476-9999
Fax: +61-2-9476-8767
Email: mail@martens.com.au
Web: www.martens.com.au

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Author(s)		**Reviewer(s)**		**Project Manager**		**Signature**	
Grant Taylor		Ralph Erni		Gray Taylor		[redacted]	
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All enquiries regarding this project are to be directed to the Project Manager.

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1 Introduction

1.1 Overview

This report documents the findings of a preliminary salinity and geotechnical assessment (including preliminary pavement thickness design), completed to support a development application (DA) to Penrith City Council (PCC) for the proposed residential subdivision at 326-330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW ('the site'). The site location is shown in Figure 1, Attachment A.

1.2 Proposed Development

The proposed development will consist of site subdivision for 65 low density residential lots, which will include:

- Earthworks for preparation of development platforms, including redirecting and filling of existing water courses / dam.
- Construction of above-ground buildings requiring limited bulk excavation, assumed < 1m below ground level (BGL).
- Installation of stormwater and other infrastructure.
- Construction of new local access roads.
- Landscaping.

1.3 Background

Review of historic aerial photography indicates that the site was previously used as rural residential / market gardens. Figure 1, Attachment A, shows the presence of one man-made dam located in the western corner of 27 Kent Road, Claremont Meadows.

1.4 Assessment Objectives

1.4.1 Salinity Assessment

The objective of the salinity assessment is to assess the risk of soil salinity so that consideration can be given to local prevailing salinity conditions and the impacts of, and on, the proposed development. This assessment has been carried out in general accordance with the following guidelines:

- Department of Land and Water Conservation (DLWC, 2002), *Site Investigations for Urban Salinity*.

- Australian Standard (AS) 3600 (2009), *Concrete structures.*

1.4.2 Geotechnical Assessment and Pavement Thickness Design

The objectives of the geotechnical assessment and pavement thickness design include:

- Assessing geotechnical conditions for management of geotechnical risks that may affect the proposed development, the site and surrounding land and infrastructure.

- Provision of preliminary recommendations and advice for initial design and construction of the proposed development.

- Provision of preliminary pavement thickness design.

The assessment was undertaken in general accordance with the principles of the following guidelines / standards:

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

- AS 1726 (1993), *Geotechnical site investigations*.

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

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

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

- Penrith City Council (2013), *Engineering Guide for Development*.

1.5 Field Investigation

Site investigation undertaken on 30 May 2016 included:

- A site walkover survey to confirm expected topography and geology, to assess existing site conditions such as geomorphological features, soil / rock exposures, surface drainage and vegetation and to identify evidence of possible saline soil or groundwater conditions.

- Reviewing DBYD survey plans.

- Nine boreholes, BH101 to BH109, to characterise subsurface materials and infer depth to top of rock. Boreholes were drilled with a 4WD truck-mounted hydraulic rig using solid flight augers fitted with a V-shaped bit (V-bit) or tungsten-carbide bit (TC-bit), up to 5.5 metres below ground level (mBGL).

- Nine Dynamic Cone Penetrometer (DCP) tests, DCP101 to DCP109, up to 3.3 mBGL, to assist soil characterisation and estimation of soil strength in accordance with AS 1289.6.3.2 (1997) and assess depth to top of rock.

- Collection of soil samples for chemical testing (electrical conductivity (EC), pH and soluble sulphate).

- Collection of two bulk soil samples for laboratory Californian Bearing Ratio (CBR) testing.

Approximate investigation locations are shown in Figure 2, Attachment A.

2 Site Conditions

2.1 General Site Details

Table 1 presents a summary of general site details. Existing site features are shown in Figure 1, Attachment A.

Table 1: Summary of general site details.

Item	Description/Detail
Site address (Lot/DP)	326 - 330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW (Lot 14 DP27107 and Lot 1001 DP1131997)
Local Government Area (LGA)	Penrith City Council
Site area	Approximately 11.67 ha (from SIX Maps)
Topography	Within slightly undulating land
Typical slopes, aspect	North west aspect with grades generally < 5 %
Elevation (from Google Earth)	Ranging from approximately 50 mAHD at the southern site boundary to approximately 47 mAHD at the northern site boundary
Expected geology	Wianamatta Group Bringelly Shale comprising shale, carbonaceous claystone, claystone, laminite, fine to medium grained lithic sandstone and rare coal and tuff (DME (1983) *Penrith 1:100,000 Geological Series Sheet 9030*).
Expected soil landscape	The NSW Environment and Heritage eSPADE website identifies the site as having soils of the Blacktown soil landscape consisting of shallow to moderately deep (>100 cm) hardsetting mottled texture contrast soils, red and brown podzolic soils on crests grading to yellow podzolic soils on lower slopes and in drainage lines.
Current land use	Rural residential
Proposed land use	Residential (Low density)
Existing site development	326 – 330 Caddens Road: A dwelling and several sheds are located in the north east corner. 27 Kent Road: A dwelling and several sheds are located in the south east corner, and a farm dam is located in the west of site. The majority of site comprises managed grass paddocks.
Existing vegetation	Managed grass
Drainage	Site drainage is via overland flow towards the west.

2.2 Subsurface Conditions

2.2.1 Encountered Subsurface Materials

Table 2 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 2: Generalised inferred subsurface profile to borehole termination depth.

Layer [1]	Depth (mBGL) [2]								
	BH101	BH102	BH103	BH104	BH105	BH106	BH107	BH108	BH109
TOPSOIL: Sandy SILT / Silty SAND (very stiff, dry)	0.0 – 0.25	0.0 – 0.3	0.0 – 0.3	0.0 – 0.35	0.0 – 0.3	0.0 – 0.3	0.0 – 0.15	0.0 – 0.1	0.0 – 0.1
RESIDUAL SOIL: CLAY / Sandy CLAY (very stiff to hard, dry / moist)	0.25 – 1.8 [3]	0.3 – 1.8 [3]	0.3 – 3.8 [3]	0.35 – 1.7 [3]	0.3 – 0.8 [3]	0.3 – 1.0 [3]	0.15 – 3.0	0.1 – 2.5	0.1 – 1.0 [3]
WEATHERED ROCK: SHALE (inferred very low to low strength)	1.8 – 2.6 [4]	>1.8 [5]	>3.8 [5]	1.7 – 2.5 [4]	0.8 – 5.5 [4]	>1.0 [5]	>3.0 [5]	>2.5 [5]	1.0 - 4.5 [4]

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 V-bit refusal.

4 Terminated due to TC-bit refusal on inferred medium strength shale. Further assessments should be carried out to confirm / revise assumed material conditions below investigation termination depths, if necessary.

5 Investigation termination upon reaching target depth.

2.2.2 Groundwater

Groundwater inflow was not observed in the boreholes up to a depth of 5.5 mBGL. Should further information on permanent site groundwater levels be required, additional investigation would need to be carried out (i.e. rock coring and installation of groundwater monitoring bores).

3 Salinity Assessment

3.1 Documented Salinity Risk Potential

The 1:100,000 *Salinity Potential in Western Sydney Map* (DIPNR, 2002) indicates the site to be located in an area of moderate salinity potential (Figure 4, Attachment A). Surrounding creek basins have been mapped as having high salinity potential and known salinity.

3.2 Broad Scale Salinity Processes

In producing the Salinity Potential Map, the Western Sydney Regional Organisation of Councils (WSROC) developed a number of alternative models of processes by which salinity may occur in Western Sydney (WSROC, 2003, pgs. 16 to 20).

A list of key broad scale salinity processes likely to impact the site, including summarised descriptions of each process, is presented in Table 3.

3.3 Signs of Potential Saline Soils at the site

No obvious signs of saline conditions were observed at the site:

- Vegetation growth appeared healthy and uninhibited.
- No water marks or salt crystals were observed on the ground surface.
- Site surface drainage appeared generally good.
- No evidence of concentrated surface erosion was observed.

3.4 Assessed Salinity Risk Potential

In Table 3, the broad scale salinity processes have been assessed in terms of likelihood of occurring at the site, considering the proposed development, site observations and investigation findings.

Table 3: Potential for broad scale salinity processes at the site.

Key salinity process	Description	Potential at subject site
Localised concentration of salinity	Localised concentration of salts due to relatively high evaporation rates. Usually associated with waterlogged soil and poor drainage. Exacerbated by increased water use and / or blocking of surface and subsurface water flow associated with urban development.	Moderate to High – No evidence of localised salt concentration or poor drainage observed. Possible waterlogged soils in area of dam and as a result of irrigation of former market gardens may have influenced site salinity.
Shale soil landscapes	In poorly drained duplex (texture contrast) soils, shallow subsurface water flows laterally across a clayey upper B-Horizon with salt usually accumulating in the clayey subsoil. Salt concentrations may increase where subsurface water accumulates and evaporates, e.g. on lower slopes or natural and constructed flats in mid-slope. Exacerbated by subsoils exposure through deep cutting, by installing buildings into the B-horizon and by impeding subsurface water flows. Highly dispersive, erodible and poorly draining sodic soils due to salinity.	Moderate to high – The site is underlain by low permeable clays, overlying shale. No evidence of impeded surface vegetation growth and surface soil erosion observed. Water accumulation and evaporation of perched water in dam. Evaporation of irrigation water associated with former market gardens.
Deep groundwater salinity	Brackish or saline groundwater rises to a level where, through capillary action in the soil, the water with dissolved salts reaches the ground surface and evaporates, resulting in localised salt concentration. Groundwater rises are typically caused by increased water infiltration, e.g. above average rainfall, vegetation loss, irrigation, increased water use in urban areas, construction of surface pits. Exacerbated by buildings or infrastructure intercepting the zone of groundwater level fluctuation.	Moderate – Groundwater was not encountered in boreholes to 5.5 mBGL. However, long-term surface water inflow into dams could lead to a rise in perched groundwater level. Proposed structures are to be constructed with appropriate drainage measures installed.
Deeply weathered soil landscape	High salt loads with high sulphate levels related to un-mapped deeply weathered soil landscapes beneath fluvial gravel, sand and clay. Usually in mid-slope or on hilltops affected by perched saline groundwater.	Low to Moderate – No evidence of deeply weathered soils observed. Deep weathering may be present within previous natural drainage channels.

3.5 Laboratory Testing

3.5.1 Overview

24 soil samples from nine boreholes were submitted to Envirolab Services, a National Association of Testing Authorities (NATA) accredited laboratory, for salinity and aggressivity testing (Electrical Conductivity (EC), pH and soluble SO_4). The testing was carried out for salinity classification and to assess an exposure classification for design

of buried concrete structures. Sampling was targeted to achieve a representative coverage of site conditions in line with assessed subsurface profiles, proposed earthworks and the limited investigation scope.

Groundwater was not observed down to investigation depth limits, being 5.5 mBGL. However, perched groundwater from surface water infiltration may occur as a result of rainfall events and should be tested, if encountered, during construction.

3.5.2 Results – Salinity Classification

Laboratory test results for salinity classification are summarised in Table 4. Laboratory test certificates are provided in Attachment D.

Table 4: Salinity test results.

Sample ID [1]	Material	$EC_{(1:5)}$ (dS/m)	EC_e (dS/m) [2]	Salinity Classification [3]
5186/101/0.2	Clay	0.220	1.87	Non – saline
5186/101/0.5	Clay	0.150	1.05	Non – saline
5186/101/1.0	Clay	0.780	5.46	Moderately saline
5186/101/1.5	Clay	0.490	3.43	Slightly saline
5186/102/1.0	Clay	0.650	4.55	Moderately saline
5186/102/1.5	Clay	0.660	4.62	Moderately saline
5186/103/0.2	Clay	0.150	1.28	Non – saline
5186/103/0.5	Clay	0.650	4.55	Moderately saline
5186/103/1.5	Clay	0.620	4.34	Moderately saline
5186/103/2.0	Clay	0.480	3.36	Slightly saline
5186/104/0.2	Clay	0.130	1.11	Non – saline
5186/104/1.0	Clay	0.720	5.04	Moderately saline
5186/105/0.2	Clay	0.170	1.45	Non – saline
5186/105/0.5	Clay	0.150	1.05	Non – saline
5186/106/0.2	Clay	0.110	0.94	Non – saline
5186/106/1.0	Clay	0.150	1.05	Non – saline
5186/107/0.5	Clay	0.410	2.87	Slightly saline
5186/107/1.5	Clay	0.430	3.01	Slightly saline

Sample ID [1]	Material	$EC_{(1:5)}$ (dS/m)	EC_e (dS/m) [2]	Salinity Classification [3]
5186/107/2.0	Clay	0.370	2.59	Slightly saline
5186/108/0.2	Clay	0.120	1.02	Non – saline
5186/108/1.0	Clay	0.380	2.66	Slightly saline
5186/108/2.5	Clay	0.330	2.31	Slightly saline
5186/109/0.2	Clay	0.140	0.98	Non – saline
5186/109/0.5	Clay	0.240	1.68	Non – saline

Notes:

1 Project#/Borehole#/Depth (mBGL)

2 Based on EC to EC_e multiplication factors from Table 6.1 in DLWC (2002).

3 Based on Table 6.2 of DLWC (2002) where EC_e <2 dS/m = non-saline, EC_e of 2-4 dS/m = slightly saline, EC_e of 4-8 dS/m = moderately saline, EC_e of 8-16 dS/m = very saline and EC_e of >16 dS/m = highly saline.

Results indicate:

- Subsurface materials at 326 – 330 Caddens Road are generally classified as non-saline to moderately saline.
- Subsurface materials at 27 Kent Road are generally classified as non-saline to slightly saline.

3.5.3 Results – Exposure Classification

Sulphate and pH test results for exposure classification are summarised in Table 5. Laboratory test certificates are presented in Attachment D.

Table 5: Exposure classification test results.

Sample ID[1]	EC_e (dS/m) [2]	pH	Sulphate (SO_4) (mg/kg)	Exposure Classification [3]
5186/101/0.2	1.87	6.8	28	A1
5186/101/0.5	1.05	5.5	150	A1
5186/101/1.0	5.46	4.7	540	A2
5186/101/1.5	3.43	5.2	360	A2
5186/102/1.0	4.55	5.0	260	A2
5186/102/1.5	4.62	5.1	310	A2
5186/103/0.2	1.28	7.3	20	A1
5186/103/0.5	4.55	6.7	420	A2
5186/103/1.5	4.34	4.9	580	A2
5186/103/2.0	3.36	5.2	220	A2

Sample ID[1]	EC_e (dS/m) [2]	pH	Sulphate (SO_4) (mg/kg)	Exposure Classification [3]
5186/104/0.2	1.11	6.6	34	A1
5186/104/1.0	5.04	5.8	300	A2
5186/105/0.2	1.45	7.2	31	A1
5186/105/0.5	1.05	5.7	170	A1
5186/106/0.2	0.94	7.2	10	A1
5186/106/1.0	1.05	5.0	190	A2
5186/107/0.5	2.87	5.0	490	A2
5186/107/1.5	3.01	5.1	390	A2
5186/107/2.0	2.59	5.2	230	A2
5186/108/0.2	1.02	5.7	160	A1
5186/108/1.0	2.66	5.2	330	A2
5186/108/2.5	2.31	5.2	200	A2
5186/109/0.2	0.98	6.4	83	A1
5186/109/0.5	1.68	5.7	290	A1

Notes:

1 Project#/Borehole#/Depth (mBGL)

2 From Table 4.

3 Exposure classification for buried reinforced concrete based on Tables 4.8.1 and 4.8.2 of AS 3600 (2009).

In accordance with AS3600 (2009), an exposure classification for concrete of 'A2' should be adopted for preliminary design of buried concrete structures.

3.6 Recommendations

We recommend that saline soil management strategies are included in the design and construction of the proposed development.

Management strategies for earthworks and landscaping should include, but not be limited to:

- Maintaining natural water balance.
- Limiting irrigation.
- Limiting soil disturbance, such as cut and fill, so saline or sodic subsoils are not exposed or groundwater is not intercepted.
- Planting of suitable salt-tolerant plant species.
- Retention of existing deep-rooted vegetation.

- Offset landscaping and gardens from building and retaining walls.
- Treating soils with gypsum before landscaping to suit selective species.
- Where consistent with future land use and landscaping plan, planting of deep-rooted, preferably native, trees to increase water absorption.
- Sealing, e.g. by lining, of stormwater detention ponds and water features to reduce infiltration.
- Preparing sediment and erosion control plans that take into account saline soils.
- Replacing excavated soils in their original order.
- Any long term irrigation or watering on-site is to be at a level that does not cause groundwater to become perched.

Management strategies for new buildings and services should include, but not be limited to:

- Limiting soil disturbance, such as compaction of soils, cutting and filling.
- Designing and building structures to limit interference with natural water flow on site.
- Using appropriate construction materials and techniques to salt proof buildings and infrastructure.
- Utilising damp proof courses and water proofing of slabs.
- Using exposure grade bricks / masonry below damp course or in retaining walls.
- Providing concrete strength and cover to steel reinforcing in accordance with AS 3600 (2009) and the exposure classifications outlined in Table 5.
- Limiting excess surface water infiltration into the soil by designing, installing and maintaining appropriate stormwater drainage (gutters, downpipes, pits and pipes).
- Further assessment including laboratory testing, to improve characterisation of site salinity conditions, particularly in proposed development areas, and assess potential ensuing implications on the proposed development and mitigation requirements.

4 Geotechnical Assessment

4.1 Preliminary Soil and Rock Strength Properties

Soil and rock strengths were estimated from DCP test results in conjunction with borehole derived soil profile data. Preliminary soil and rock strength properties are summarised in Table 6.

Table 6: Preliminary estimated soil and inferred rock strength properties.

Layer [1]	$\gamma_{in\text{-}situ}$ [2] (kN/m³)	Cu [3] (kPa)	Φ' [4] (°)	E' [5] (MPa)
TOPSOIL: Sandy SILT (very stiff)	17	25	NA [6]	5
RESIDUAL SOIL: CLAY/ Sandy CLAY (very stiff to hard)	18	100	NA [6]	20
WEATHERED ROCK: SHALE (low to medium strength)	22	NA [6]	28	100

Notes:

1 Refer to borehole logs in Attachment B for material description details.

2 Inferred average In-situ unit weight for layer, based on visual assessment only (±2 kN/m³)

3 Undrained shear strength (± 5 kPa) assuming normally consolidated clay.

4 Effective internal friction angle (± 2 °) assuming drained conditions; may be dependent on rock defect conditions.

5 Effective Elastic Modulus (±10 %).

6 Not applicable.

4.2 Risks of Slope Instability

No evidence of former land instability was observed during the site walkover survey.

We consider the risk of potential slope instability, such as landslide or soil creep, to be very low subject to the recommendations in this report being followed, adoption of relevant engineering standards and CSIRO BTF 18 (Attachment F). A detailed slope risk assessment in accordance with Australian Geomechanics Society's Landslide Risk Management Guidelines (2007) was not undertaken.

4.3 Initial Design and Construction Advice and Recommendations

From a geotechnical viewpoint, we consider the site to be suitable for the proposed development, subject to the recommendations presented in this report. No information regarding proposed structures, including footing requirements, was made available at the time of reporting. For the purpose of this report, we have considered typical structures associated with low density residential development. Site specific testing will be required at detailed design stage to provide further recommendations.

Table 7 presents a summary of preliminary geotechnical recommendations for initial design and developing construction methodologies for the proposed development.

Table 7: Preliminary geotechnical recommendations for initial design and construction.

Item	Recommendation
Excavations	Shallow excavations will likely encounter residual clay soils over weathered shale. In light of this, shallow excavations should be readily carried out as follows: o Soils and extremely to low strength rock — Conventional hydraulic earthmoving equipment. o Medium strength, or stronger rock, if encountered — Hydraulic earthmoving equipment with rock hammer attachment or ripping tyne for medium and higher rock. Earthwork and site preparation is to be reviewed by a Geotechnical Engineer and undertaken in accordance with AS 3798 (2007), Code of Practice 'Excavation Work' (Safe Work Australia, 2015) and Penrith City Council's engineering guidelines (2013).
Footings	Shallow footings, such as pad and strip footings, slab-on-ground or deepened pier or piled footings may be adopted as support for new structures. Footings should be founded on natural material. Shallow footings may be designed adopting preliminary geotechnical design parameters of 100 kPa and 300 kPa, for foundations on very stiff and hard residual clay, respectively, subject to an embedment depth of at least 0.3 m into the design material unit. Alternatively, should higher bearing capacities be required, the use of deepened footings such as piles founding in rock may be considered. Subject to embedment of at least 0.5 m or 1 pile diameter, whichever is greater, an allowable end bearing capacity of 700 kPa may be adopted for preliminary design. Bearing capacity values should be confirmed by further assessment as detailed in Section 6.2 and during construction by a geotechnical engineer on site. 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. 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.
Retaining structures/ batters	Any excavations exceeding 0.75 m in height should be supported by suitably designed and installed retaining or shoring structures. Alternatively, 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. Retaining structures to be constructed as part of site works are to be engineer designed and backfilled with suitable granular material and free-draining drainage materials. Preliminary design may adopt preliminary active and passive earth pressure coefficients

Item	Recommendation
	of 0.4 and 2.5 respectively. Retaining wall design should consider additional surcharge loading from proposed structures, construction equipment, backfill compaction and static water pressures unless subsoil drainage is provided behind 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.
Earthworks	The existing dam may require earthworks for site preparation. All earthworks should be carried out following removal of topsoil, silt deposits and other unsuitable materials in accordance with AS3798 (2007) *Guidelines on Earthworks for Commercial and Residential Development*. A qualified geotechnical engineer should inspect the condition of the exposed material to assess suitably of the exposed / prepared surface as foundation for fill placement. Fill material comprising approved imported granular fill material 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. Further assessment is to be carried out should excavated site material is to be used for filling at the site. Earthworks compliance testing should be carried out in accordance with Table 8.1 of AS3798 (2007), with testing to be provided by a National Association of Testing Authorities (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, 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 DI of 70% or DR of 95% SMDD and moisture conditioned to be within 2% of OMC.
Overland flows	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.
Soil erosion	Soil overburden should be removed in a manner that reduces the risk of sedimentation of natural drainage channels and existing stormwater drainage systems in the vicinity of the site. All 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: o Maintain vegetation where possible. o Disturb minimal area during excavation. o Landscape disturbed areas following completion of constructions. o Use gabion mattress, or other suitable energy reduction solutions, where required. o Direct water away from structures.

Item	Recommendation
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.

4.4 Site Classification

Clay of generally medium to high plasticity was encountered up to 3.8 mBGL. The clay is likely to have a high potential to shrink and swell as a result of changes in soil moisture content.

A preliminary site classification of 'H1' should be adopted for design of lightly loaded shallow footings, in accordance with AS 2870 (2011), subject to the recommendations presented in this report and CSIRO guidelines (CSIRO BTF 18, 2003; Attachment F). A reclassification to 'M' may be possible subject to results of further laboratory testings and earthworks adopted for site development.

A preliminary site classification of 'P' should be adopted where footings are likely to be impacted by the presence of uncontrolled fill or soft or loose foundation material or by environments that could lead to exceptional moisture condition variations within foundation material, such as areas impacted by dams and filling.

5 Preliminary Pavement Thickness Design

5.1 Overview

Preliminary pavement thickness design was undertaken for potential new cul-de-sac and local access roads. The design adopted a traffic loading of Equivalent Standard Axles (ESA) in accordance with Penrith City Council's Engineering Guide for Development (2013) and Austroads (2012) Guide to Pavement Technology Part 2 Pavement Structural Design. A CBR value obtained from limited lab testing was adopted for the preliminary design.

5.2 Design Parameters

Table 8 presents Equivalent Standard Axles (ESA) values adopted for design of cul-de-sac and local access roads.

Table 8: ESA values for proposed cul-de sac and local access roads.

Road Type	N (ESA)
Cul-de-sac	$5x10^4$
Local street	$5x10^5$

Two bulk soil samples were collected from adjacent to BH103 and BH108 (Figure 2, Attachment A) and submitted to Resource Laboratories, a NATA accredited laboratory for CBR testing. A four day soaked CBR testing 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 9. A laboratory test certificate is provided in Attachment E.

Table 9: Laboratory CBR.

Sample Number	Material	Sample Depth (mBGL)	CBR [1] Value (%)
5186/103/0.3-0.8/S/1	Silty CLAY	0.3 – 0.8	3.0
5186/108/0.1-0.6/S/1	Silty CLAY	0.1 – 0.6	4.0

Notes:

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

Based on correlation between Austroads (2012) and the DCP test results, a CBR value of between 6 and 25 applies to the residual clay subgrade. We note, however that the soils were encountered in a dry condition. These CBR values should therefore be reduced to consider long-term ground conditions.

For the purpose of this assessment, a CBR value of 3 % was adopted.

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

5.3 Pavement Thickness

Based on laboratory test result and DCP-CBR correlations, a CBR value of 3.0 has been adopted for preliminary design purposes. If medium to high plasticity clay is encountered at subgrade level during excavation, lower CBR values may be applicable and the pavement material thickness may need to be revised.

Table 10 presents preliminary recommended pavement material thicknesses for proposed cul-de-sac and local access roads.

Table 10: Preliminary pavement material thickness design for CBR of 3.

Road Type [1]	Layer	Thickness (mm)
Cul-de-sac	1 layer of AC10 and final layer of 25 mm "Residential" mix (Pavement Surfacing)	50 [2]
	Base (DGB)	150 [3]
	Sub-base (DGS)	175 [3]
	Total pavement thickness	375
Local Street	1 layer of AC10 and final layer of 25 mm "Residential" mix (Pavement Surfacing)	50 [2]
	Base (DGB)	150
	Sub-base (DGS)	270
	Total pavement thickness	470

Notes:

1 Based on Penrith Council's Engineering Guide for Development (2013).

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

3 Minimum layer thickness (PCC, 2016)

5.4 Earthworks

5.4.1 Subgrade Preparation

The subgrade is to be trimmed and compacted with density testing of the upper 300 mm layer at a rate of 1 test per 50 m of road length. Minimum density shall be 100 % Maximum Dry Density (MDD) at a standard compactive effort within 0 % and -3 % of optimum moisture content (OMC). Prior to placement of pavement material, the subgrade shall be proof rolled and approved by a Geotechnical Engineer.

Soft spots can be treated by one of the following methods subject to final design by Martens:

1. Removal and replacement with approved fill under Geotechnical Engineer's direction.

2. *In-situ* stabilisation with cement, lime or similar binding agent to a depth of at least 300 mm below finished level. Use of this method and extent will depend on the condition of material to be stabilised.

5.4.2 Subsoil Drainage

Surface and subsoil drainage should be provided in accordance with Council requirements. Typically subsurface drains are installed on the upslope side of all internal roads and generally extend 500 mm below pavement level.

5.4.3 Placement and Testing of Pavement Material

Pavement materials shall be placed in layers (when compacted) not thicker than 250 mm or less than 75 mm. Pavement materials shall be compacted to the following condition:

- Sub-base - Minimum 98 % MDD at modified compactive effort (±2 % OMC).

- Base - Minimum 98 % MDD at modified compactive effort (±2 % OMC).

Compaction testing shall be undertaken by a NATA accredited laboratory in accordance with procedures as outlined in AS1289. Tests should be carried out at a rate of 1 per 250 m^2 per layer or 3 per layer placed, whichever is the greater. Each pavement layer shall be proof rolled under geotechnical engineer's supervision. Subsequent layers of pavement shall not be placed prior to approval of underlying layer.

5.4.4 Earthworks

Should filling be required to raise subgrade levels, the use of site-won excavated materials may be considered, subject to approval by a geotechnical engineer and implementing stringent moisture conditioning controls, or mixing with lime, to assist material placement and testing. Alternatively, suitable granular fill, approved for use by a Geotechnical Engineer, may be adopted.

All earthwork and fill material testing and preparation is to be approved by a Geotechnical Engineer and undertaken in accordance with AS 3798 (2007) and Penrith City Council's engineering guidelines (2013).

6 Proposed Additional Assessments

6.1 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:

- Assessment of foundation condition up to at least 2 m below final bulk excavation and foundation levels, as applicable.
- 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 shrink / swell and Atterberg Limits laboratory testings.
- Assessment of subsurface conditions, including CBR testing, along proposed road alignments.
- Assessment of site specific foundation material capacity to support adopted footing types.
- Assessment of subsurface conditions in the vicinity of the existing dam for earthworks requirements.
- Review of construction staging plans by a Geotechnical Engineer.
- Further salinity testing to delineate salinity conditions across soil profiles and development areas, considering final development details.

6.2 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 11) during site works. This program may be updated following further detailed investigations.

Table 11: Recommended inspections/monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall) installations and batters and monitor associated performance.	Daily / As required	Builder / MA
Monitor groundwater seepage from excavation faces to assess adequacy of drainage provision.	When encountered	Builder / MA
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
Inspect fill material to verify suitability for placement at the site and for provision of advice associated with fill placement.	Prior to fill placement.	MA

Notes:

MA = Martens and Associates Geotechnical Engineer.

6.3 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.

7 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 spot levels from Google Earth.

8 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, 1993) *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.*

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

Department of Infrastructure Planning and Natural Resources (DIPNR, 2002) *Salinity Potential in Western Sydney Map.*

Department of Land and Water Conservation (DLWC, 2002) *Site investigations for urban salinity.*

Geological Survey of NSW Department of Minerals and Energy (1991), *Penrith 1:100,000 Geological Series Sheet 9030*.

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

Penrith City Council (2010), *Development Control Plan*.

Penrith City Council (2016), *Engineering Construction Specification for Civil Works.*

Western Sydney Regional Organisation of Councils (WSROC, 2003) *Western Sydney Salinity Code of Practice.*

9 Attachment A – Figures

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Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	GMT	**SITE AND EXISTING DAM LOCATION** **326 - 330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW** (Source: Nearmap)	Drawing No: **FIGURE 1**
Approved:	RE		
Date:	25.08.2016		
Scale:	Not to Scale		File No: P1605186JR02V01

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Key:

Approximate site boundary

Indicative borehole, DCP test location

Indicative CBR sampling location

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	GMT	**GEOTECHNICAL TESTING LOCATION PLAN** **326 - 330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW** (Source: Nearmap)	Drawing No: **FIGURE 2**
Approved:	RE		
Date:	25.08.2016		
Scale:	Not to Scale		File No: P1605186JR02V01

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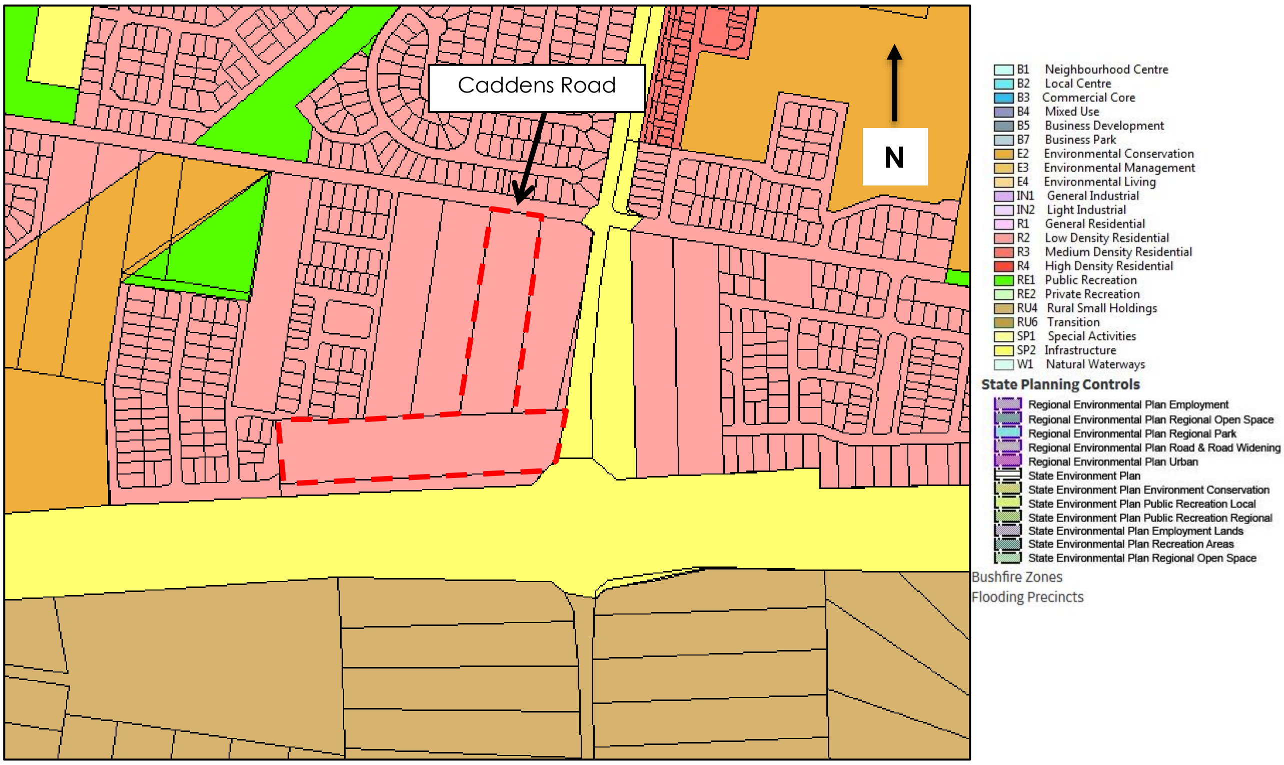
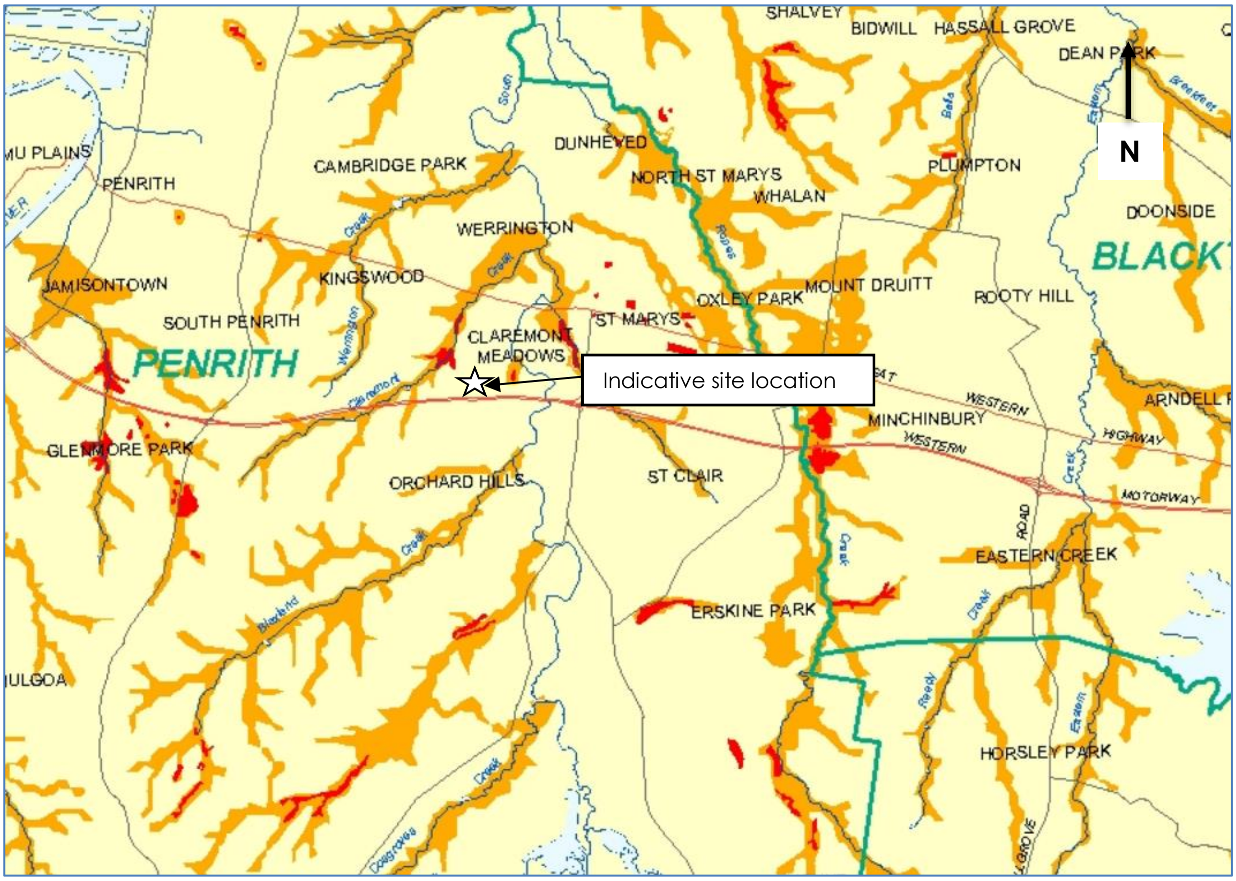


Key:

Approximate site boundary

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	GMT	**MAP OF LANDUSE ZONES** (Source: PCC MapsOnline)	Drawing No: **FIGURE 3**
Approved:	RE		
Date:	2508.2016		
Scale:	Not to Scale		File No: P1605186JR02V01



MAPPING CATEGORY	ASSOCIATED SOIL LANDSCAPES	LANDFORM - GEOLOGY
KNOWN SALINITY Areas where there is a known occurrence of saline soil, or where air photo interpretation and field observations have confirmed more than one of these: a - scalding b - salt efflorescence c - vegetation dieback d - salt tolerant plant species e - waterlogging A high relative wetness index occurs in these areas.	* Salinity outbreaks occur in Blacktown (bt), Luddenham (lu) and Richmond (ri) Soil Landscapes - common at breaks of slope, lower slopes and drainage lines. * Berkshire Park (bp) and Upper Castlereagh (up) Soil Landscapes have localised salinity due to the impermeable nature of the clay parent material. * South Creek (sc), Monkey Creek (mk), Freemans Reach (fr) and Theresa Park (tp) Soil Landscapes have common saline outbreaks due to high run-on and low local relief. * Soils in the above landscapes have high clay content in subsoils and are imperfectly to poorly drained.	* Break of slope, lower slope and drainage lines of Wianamatta Shales (Rwb,Rwa and Rwn). * Localised salinity also occurs at the geological boundary between Tertiary Gravels (Tl, Tr) and underlaying Wianamatta Shales (Rwb,Rwa/ Quaternary Alluvials (Qpd, Qpa, Qpl, Qal). * Localised salinity occurs in Quaternary Alluvium (Qal, Qpn. Qpd) which underlies many of the drainage systems and wetland margins.
HIGH SALINITY POTENTIAL Areas where soil, geology, topography and groundwater conditions predispose a site to salinity. These conditions are similar to areas of known salinity (see above). These areas are most common in lower slopes and drainage systems where water accumulation is high (ie. high relative wetness index).	* Soil Landscapes include Birrong (bi), Blacktown (bt) Berkshire Park (bp), Freemans Reach (fr), South Creek(sc0, Theresa Park (tp), Richmond (ri) and Luddenham (lu). Drainage systems and convergent slopes are areas of highest risk. * Soils in these landscapes have high clay content in the subsoils, low permeability and high run-on. * Soil profiles may display signs of high salt concentrations at depth (i.e. >0.5m).	* Salinity is most likely to occur in lower slopes, foot-slopes, floodplains and creek lines on Quaternary Sediments (Qal, Qpn, Qpd, Qpc, Qpp, Qha)/Wianamatta Shales (Rwb, Rwn, Rwa) where run-on is high, resulting in seasonally high water tables and soil saturation.
MODERATE SALINITY POTENTIAL Areas on Wianamatta Group Shales and Tertiary Alluvial Terraces. Scattered areas of scalding and indicator vegetation have been noted but no concentrations have been mapped. Saline areas may occur in this zone, which have not yet been identified or may occur if risk factors change adversely.	* Areas of Agnes Banks (ab), Berkshire Park (bp), Blacktown (bt), Luddenham (lu) and Lucas Heights (lh). * Steeper areas with moderate to high local relief and well drained subsoils such as Picton (pn), West Pennant Hills (wp) and Glenorie (gn) are at a lower risk of developing salinty. * Soils are moderate to well-drained due to their elevated position in the landscape.	* Hill-slopes and hill-crests on Wianamatta Shales (Rwb, Rwn, Rwa). * Raised abandoned alluvial terraces and drainage lines on Quaternary Alluvium (Qal, Qpn, Qpd, Qpc, Qpp) from Richmond to Camden and east to Rookwood. Localised areas of elevated, well-drained Tertiary Gravels (Ta, Tl, Tr).
VERY LOW SALINITY POTENTIAL Areas where salinity processes do not operate or are of minor significance. Soils are rapidly drained and underlaying strata (Hawkesbury/Narrabeen Sandstone) are highly permeable, resulting in continual flushing and removal of salts in the landscape. No salinity has been observed in these areas and is not expected to occur.	* Rapidly drained soil landscapes with shallow soils include Warragamba (wb) and Hawkesbury (ha). * Gymea (gy) and Faulconbridge (fb) Soil Landscapes consist of highly permeable sands with well-drained subsoils. * Soils are well to rapidly drained. * Soils have high sand content.	* Occurring on Hawkesbury and Narrabeen Sandstone (Rh, Rno). * Groundwater is relatively fresh in these areas due to the sandstone's elevated position in the landscape and highly permeable nature, resulting in continuous flushing of the system (removal of any accumulated salts).

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	GMT	**1:100,000 MAP OF SALINITY POTENTIAL IN WESTERN SYDNEY** (Source: DIPNR, 2002)	Drawing No: **FIGURE 4**
Approved:	RE		
Date:	25.08.2016		
Scale:	Not to Scale		File No: P1605186JR02V01

10 Attachment B - Borehole Logs

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	**REF BH101**
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	44 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø90 mm x 2.60 m depth	NORTHING		ASPECT	NW	SLOPE: <5%

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		44.00				ML	Sandy SILT, low liquid limit, brown, fine to medium sand	D		TOPSOIL
				0.25 43.75	5186/BH101/0.2/S/1 D 0.20 m; 5186/BH101/0.5/S/1 D 0.50 m; 5186/BH101/1.0/S/1 D 1.00 m; 5186/BH101/1.5/S/1 D 1.50 m			CI-CH	CLAY, medium to high plasticity, brown grading to grey, trace fine to medium sand.	D - M	VSt	RESIDUAL SOIL
	M		1								H	
AD/T	M-H		2	1.80 42.20					SHALE, brown/grey, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 1.80: V-bit refusal.
				2.60					Hole Terminated at 2.60 m (Target depth reached)			2.60: TC-bit refusal on inferred low to medium strength shale.
			3									
			4									
			5									
			6									

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

martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH102
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	48 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø90 mm x 1.80 m depth	NORTHING		ASPECT	NW	SLOPE: <5%

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		48.00	5186/BH102/0.2/S/1 D 0.20 m			ML	Sandy SILT, low liquid limit, brown, fine to medium sand			TOPSOIL
				0.30 47.70				CI-CH	CLAY, medium to high plasticity, brown grading to grey, trace fine to medium sand.	D	VSt	RESIDUAL SOIL
			1		5186/BH102/1.0/S/1 D 1.00 m							
	H			1.80	5186/BH102/1.5/S/1 D 1.50 m							
			2						Hole Terminated at 1.80 m (Target depth reached)			1.80: V-bit refusal on inferred very low to low strength shale.
			3									
			4									
			5									
			6									

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martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

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CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH103
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	47 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø90 mm x 3.80 m depth	NORTHING		ASPECT	NW	SLOPE <5%

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		47.00	5186/BH103/0.2/S/1 D 0.20 m			ML	Sandy SILT, low liquid limit, brown, fine to medium sand	D	VSt	TOPSOIL
	M			0.30 46.70	5186/BH103/0.3-0.8/S/1 D 0.30 m; 5186/BH103/0.5/S/1 D 0.50 m			CI-CH	CLAY, medium to high plasticity, brown, trace fine to medium angular sand.			RESIDUAL SOIL
	M			1.50 45.50	5186/BH103/1.5/S/1 D 1.50 m; 5186/BH103/2.0/S/1 D 2.00 m			CH	CLAY, high plasticity, grey mottled red/brown, trace medium ironstone gravel.	M		
	H				5186/BH103/3.0/S/1 D 3.00 m						H	
				3.80					Hole Terminated at 3.80 m (Target depth reached)			3.80: V-bit refusal on inferred very low to low strength shale.

Depth scale (metres): 1, 2, 3, 4, 5, 6

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

martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF **BH104**
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	52 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø90 mm x 2.50 m depth	NORTHING		ASPECT	NW	SLOPE <5%

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		52.00	5186/BH104/0.2/S/1 D 0.20 m			ML	Sandy SILT, low liquid limit, brown, fine to medium sand	D	VSt	TOPSOIL
	M			0.35 51.65	5186/BH104/0.5/S/1 D 0.50 m			CI-CH	CLAY, medium to high plasticity, brown mottled grey, with some fine to medium sand.		H	RESIDUAL SOIL
			1		5186/BH104/1.0/S/1 D 1.00 m						VSt	
	H										H	
AD/T	M		2	1.70 50.30					SHALE, grey inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 1.70: V-bit refusal.
				2.50					Hole Terminated at 2.50 m (Target depth reached)			2.50: TC-bit refusal on inferred low to medium strength shale.
			3									
			4									
			5									
			6									

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

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH105
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186

EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	54.5 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø90 mm x 5.50 m depth	NORTHING		ASPECT	NW	SLOPE	<5%

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		54.50				SM	Silty SAND, fine to medium grained, brown, trace fine angular gravels.	D	VSt	TOPSOIL
				0.30 54.20	5186/BH105/0.2/S/1 D 0.20 m							
	M-H				5186/BH105/0.5/S/1 D 0.50 m			CI	Sandy CLAY, medium plasticity, brown, fine to coarse sand, trace fine angular gravel.	M	H	RESIDUAL SOIL
AD/T	M		1	0.80 53.70					SHALE. grey, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 0.80: V-bit refusal.
	H		2									
	M		3									
			4									
			5	5.50								
			6						Hole Terminated at 5.50 m (Target depth reached)			5.50: TC-bit refusal on inferred low to medium strength shale.

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

martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH106
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	55 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø90 mm x 1.00 m depth	NORTHING		ASPECT	NW	SLOPE <5%

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		55.00	5186/BH106/0.2/S/1 D 0.20 m			SM	Silty SAND, fine to medium grained, brown, trace rootlets.	D	VSt	TOPSOIL
	M			0.30 54.70	5186/BH106/0.5/S/1 D 0.50 m			CI-CH	Sandy CLAY, medium to high plasticity, brown mottled grey, fine to medium sand.			RESIDUAL SOIL
	H		1	1.00	5186/BH106/0.95/S/1 D 0.95 m						H	
									Hole Terminated at 1.00 m (Target depth reached)			1.00: V-bit refusal on inferred very low to low strength shale.
			2									
			3									
			4									
			5									
			6									

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

martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	**REF BH107**			
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1			
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186			
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	50 m	DATUM	AHD		
EXCAVATION DIMENSIONS	Ø90 mm x 3.00 m depth	NORTHING		ASPECT	NW	SLOPE	<5%		

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		50.00				ML	Sandy SILT, low liquid limit, brown, fine to medium sand	D		TOPSOIL
	M		0.15	49.85	5186/BH107/0.2/S/1 D 0.20 m 5186/BH107/0.5/S/1 D 0.50 m 5186/BH107/1.0/S/1 D 1.00 m 5186/BH107/1.5/S/1 D 1.50 m			CI-CH	CLAY, medium to high plasticity, brown becoming grey mottled red below 0.9 m, trace fine to medium sand.		VSt VSt - H	RESIDUAL SOIL
	H				5186/BH107/2.0/S/1 D 2.00 m						H	
			3.00						Hole Terminated at 3.00 m (Target depth reached)			

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

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Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH108
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	52 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø90 mm x 2.50 m depth	NORTHING		ASPECT	NW	SLOPE <5%

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		0.10				ML	Sandy SILT, low liquid limit, brown, fine to medium sand			TOPSOIL
				51.90	5186/BH108/0.1-0.6/S/1 B 0.10 m 5186/BH108/0.2/S/1 D 0.20 m 5186/BH108/0.5/S/1 D 0.50 m			CI-CH	CLAY, medium to high plasticity, brown, grading to grey mottled red below 0.9 m, trace sand.	D	VSt	RESIDUAL SOIL
	M		1		5186/BH108/1.0/S/1 D 1.00 m 5186/BH108/1.5/S/1 D 1.50 m					M	VSt - H	
			2	2.50	5186/BH108/2.4/S/1 D 2.40 m						H	
			3 4 5 6						Hole Terminated at 2.50 m (Target depth reached)			

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

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MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Universal Property Group Pty Ltd	COMMENCED	30/05/2016	COMPLETED	30/05/2016	REF BH109
PROJECT	Preliminary Salinity & Geotechnical Assessment	LOGGED	AT	CHECKED	RE	Sheet 1 OF 1
SITE	326-330 Caddens Rd, Claremont Meadows, NSW.	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1605186
EQUIPMENT	Hydraulic Rig	EASTING		RL SURFACE	46 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø90 mm x 4.50 m depth	NORTHING		ASPECT	NW	SLOPE <5%

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		0.10				ML	Sandy SILT, low liquid limit, brown, fine to medium sand.			TOPSOIL
AD/V	M			45.90				CI-CH	CLAY, medium to high plasticity, brown, with some fine to medium sand.	D	VSt	RESIDUAL SOIL
AD/V	H										VSt - H	
AD/T	M		1	1.00 45.00					SHALE, grey/brown, inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 1.00: V-bit refusal.
			2									
			3									
			4	4.50								
			5						Hole Terminated at 4.50 m (Target depth reached)			
			6									

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

martens

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

MARTENS 1.01.7 LIB.GLB Log MARTENS BOREHOLE P1605186BH101-BH109.GPJ <<DrawingFile>> 27/07/2016 15:14 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Martens 1.01.7 2016-03-04 Prj: Martens 1.01.5 2015-12-17

11 Attachment C - DCP 'N' Counts

Dynamic Cone Penetrometer Test Log Summary

martens
consulting engineers since 1989

Suite 201, 20 George Street, Hornsby, NSW 2077 Ph: (02) 9476 9999 Fax: (02) 9476 8767, mail@martens.com.au, www.martens.com.au

Site	326 - 330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW	**DCP Group Reference**	P1605186JS01V01
Client	Universal Property Group Pty Ltd	**Log Date**	30.05.2016
Logged by	GMT		
Checked by	RE		
Comments			

TEST DATA

Depth Interval (m)	DCP101	DCP102	DCP103	DCP104	DCP105	DCP106	DCP107	DCP108	DCP109	
0.15	10	10	8	8	13	8	10	7	8	
0.30	12	15	10	12	15	8	6	10	12	
0.45	14	11	11	15	9	11	7	13	14	
0.60	14	11	11	22	15	8	7	13	15	
0.75	15	9	8	25	17	30	16	14	18	
0.90	10	8	7	12	21	Terminated @ 0.75 m due to high 'N' counts	18	12	18	
1.05	9	9	7	12	Bounce @ 0.9 m		19	14	Bounce @ 0.9 m	
1.20	7	8	7	10			18	17		
1.35	9	9	7	13			17	19		
1.50	15	10	7	11			18	22		
1.65	20	9	8	16			18	24		
1.80	26	13/100mm	7	25/100mm			23	28		
1.95	Bounce @ 1.8 m	Bounce @ 1.75 m	8	Bounce @ 1.75 m			32	35		
2.10			15				Terminated @ 1.95 m due to high 'N' counts	Terminated @ 1.95 m due to high 'N' counts		
2.25			9							
2.40			13							
2.55			10							
2.70			15							
2.85			15							
3.00			21							
3.15			26							
3.30			35							
3.45			Terminated @ 3.3 m due to high 'N' counts							
3.60										
3.75										

12 Attachment D - Envirolab Laboratory Test Results

12 Ashley Street, Chatswood, NSW 2067
tel: +61 2 9910 6200

email: sydney@envirolab.com.au
envirolab.com.au

Envirolab Services Pty Ltd - Sydney | ABN 37 112 535 645

CERTIFICATE OF ANALYSIS 147738

Client:
Martens & Associates Pty Ltd
Suite 201, 20 George St
Hornsby
NSW 2077

Attention: Amir Tavasol

Sample log in details:

Your Reference:	**P1605186, Salinity Assess. Claremont Meadows**
No. of samples:	32 Soils
Date samples received / completed instructions received	01/06/16 / 01/06/16

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	8/06/16 / 7/06/16
Date of Preliminary Report:	Not issued

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

[redacted]

Misc Inorg - Soil						
Our Reference:	UNITS	147738-1	147738-2	147738-3	147738-4	147738-6
Your Reference	-------------	5186/101/0.2/S/1	5186/101/0.5	5186/101/1.0	5186/101/1.5	5186/102/1.0
Date Sampled	------------	30/05/2016	30/05/2016	30/05/2016	30/05/2016	30/05/2016
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
Date analysed	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
pH 1:5 soil:water	pH Units	6.8	5.5	4.7	5.2	5.0
Electrical Conductivity 1:5 soil:water	µS/cm	220	150	780	490	650
Sulphate, SO4 1:5 soil:water	mg/kg	28	150	540	360	260

Misc Inorg - Soil						
Our Reference:	UNITS	147738-7	147738-8	147738-9	147738-10	147738-11
Your Reference	-------------	5186/102/1.5	5186/103/0.2	5186/103/0.5	5186/103/1.5	5186/103/2.0
Date Sampled	------------	30/05/2016	30/05/2016	30/05/2016	30/05/2016	30/05/2016
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
Date analysed	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
pH 1:5 soil:water	pH Units	5.1	7.3	6.7	4.9	5.2
Electrical Conductivity 1:5 soil:water	µS/cm	660	150	350	620	480
Sulphate, SO4 1:5 soil:water	mg/kg	310	20	420	580	220

Misc Inorg - Soil						
Our Reference:	UNITS	147738-13	147738-15	147738-16	147738-17	147738-18
Your Reference	-------------	5186/104/0.2	5186/104/1.0	5186/105/0.2	5186/105/0.5	5186/106/0.2
Date Sampled	------------	30/05/2016	30/05/2016	30/05/2016	30/05/2016	30/05/2016
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
Date analysed	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
pH 1:5 soil:water	pH Units	6.6	5.8	7.2	5.7	7.2
Electrical Conductivity 1:5 soil:water	µS/cm	130	720	170	150	110
Sulphate, SO4 1:5 soil:water	mg/kg	34	300	31	170	10

Misc Inorg - Soil						
Our Reference:	UNITS	147738-20	147738-22	147738-24	147738-25	147738-26
Your Reference	-------------	5186/106/1.0	5186/107/0.5	5186/107/1.5	5186/107/2.0	5186/108/0.2
Date Sampled	------------	30/05/2016	30/05/2016	30/05/2016	30/05/2016	30/05/2016
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
Date analysed	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016	06/06/2016
pH 1:5 soil:water	pH Units	5.0	5.0	5.1	5.2	5.7
Electrical Conductivity 1:5 soil:water	µS/cm	150	410	430	370	120
Sulphate, SO4 1:5 soil:water	mg/kg	190	490	390	230	160

Document Set ID: 9942239
Version: 1, Version Date: 11/03/2022

Misc Inorg - Soil					
Our Reference:	UNITS	147738-28	147738-30	147738-31	147738-32
Your Reference	-------------	5186/108/1.0	5186/108/2.5	5186/109/0.2	5186/109/0.5
Date Sampled	------------	30/05/2016	30/05/2016	30/05/2016	30/05/2016
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016
Date analysed	-	06/06/2016	06/06/2016	06/06/2016	06/06/2016
pH 1:5 soil:water	pH Units	5.2	5.2	6.4	5.7
Electrical Conductivity 1:5 soil:water	µS/cm	380	330	140	240
Sulphate, SO4 1:5 soil:water	mg/kg	330	200	83	290

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			03/06/2016	147738-1	06/06/2016 || 06/06/2016	LCS-1	03/06/2016
Date analysed	-			06/06/2016	147738-1	06/06/2016 || 06/06/2016	LCS-1	06/06/2016
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	147738-1	6.8 || 6.7 || RPD: 1	LCS-1	103%
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	147738-1	220 || 210 || RPD: 5	LCS-1	103%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	147738-1	28 || 21 || RPD: 29	LCS-1	107%

QUALITY CONTROL Misc Inorg - Soil	UNITS	Dup. Sm#	Duplicate Base + Duplicate + %RPD	Spike Sm#	Spike % Recovery
Date prepared	-	147738-13	06/06/2016 || 06/06/2016	LCS-2	03/06/2016
Date analysed	-	147738-13	06/06/2016 || 06/06/2016	LCS-2	06/06/2016
pH 1:5 soil:water	pH Units	147738-13	6.6 || 6.7 || RPD: 2	LCS-2	101%
Electrical Conductivity 1:5 soil:water	µS/cm	147738-13	130 || 100 || RPD: 26	LCS-2	98%
Sulphate, SO4 1:5 soil:water	mg/kg	147738-13	34 || 29 || RPD: 16	LCS-2	108%

QUALITY CONTROL Misc Inorg - Soil	UNITS	Dup. Sm#	Duplicate Base + Duplicate + %RPD	Spike Sm#	Spike % Recovery
Date prepared	-	[NT]	[NT]	147738-2	03/06/2016
Date analysed	-	[NT]	[NT]	147738-2	06/06/2016
pH 1:5 soil:water	pH Units	[NT]	[NT]	[NR]	[NR]
Electrical Conductivity 1:5 soil:water	µS/cm	[NT]	[NT]	[NR]	[NR]
Sulphate, SO4 1:5 soil:water	mg/kg	[NT]	[NT]	147738-2	130%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NR: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

13 Attachment E - CBR Test Results

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:** 16-0033
Project: P1605186 **Report number:** 1
Location: 326-330 Caddens Road and 27 Kent Road, Claremont Meadows, NSW **Page:** 1 of 1

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.	8452	8453		
Customer sample no.	5186/103/ 0.3-0.8/S/1	5186/108/ 0.1-0.6/S/1		
Date sampled	30/05/2016	30/05/2016		
Material description	CLAY, with silt, trace of gravel, brown mottled grey/red-brown	CLAY, with silt, trace of gravel, pale-brown mottled red-brown		
Maximum dry density (t/m^3)	1.64	1.64		
Optimum moisture content (%)	21.1	21.9		
Field moisture content (%)	n/a	n/a		
Oversize retained on 19.0mm sieve (%)	0	0		
Oversize included (Y/N)	N	N		
Dry density before soak (t/m^3)	1.60	1.61		
Dry density after soak (t/m^3)	1.57	1.58		
Moisture content before soak (%)	21.2	21.7		
Moisture content after soak (%)	24.4	24.1		
Moisture content after test - top 30mm (%)	29.0	27.3		
Moisture content after test - remaining depth (%)	22.6	22.7		
Density ratio before soaking (%)	98.0	98.0		
Moisture ratio before soaking (%)	100.5	99.5		
Period of soaking (days)	4	4		
Compactive effort	Standard	Standard		
Mass of surcharge applied (kg)	4.5	4.5		
Swell after soaking (%)	2.0	1.5		
Penetration (mm)	2.5	2.5		
CBR Value (%)	**3.0**	**4.0**		

Notes: Specified LDR: 98 ±1%

Approved Signatory: [redacted] E. Maldonado **Date:** 15/06/2016

NATA
ACCREDITED FOR TECHNICAL COMPETENCE
Accredited for compliance with ISO/IEC 17025.

NATA Accredited Laboratory Number: **17062**

14 Attachment F - 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.

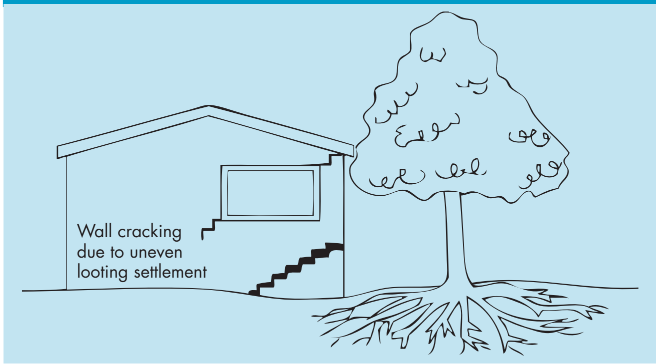
Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends 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.

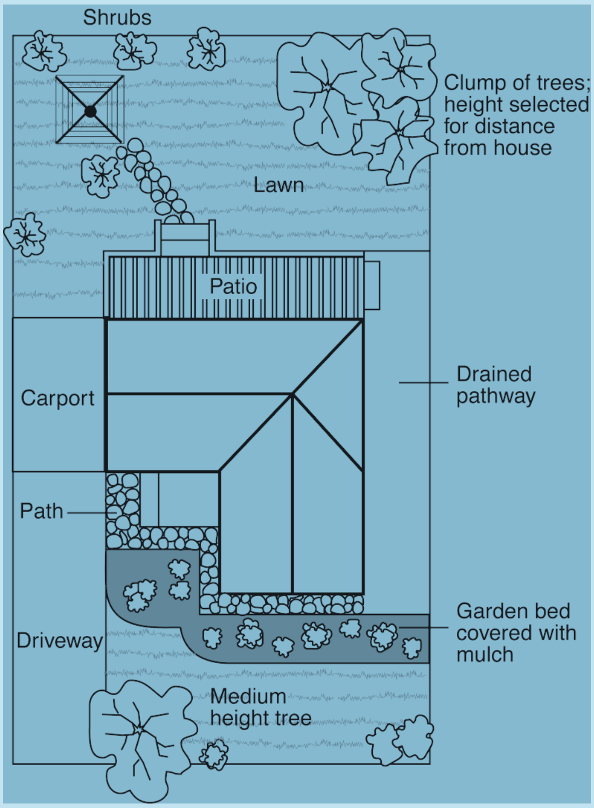
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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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Tel (03) 9662 7666 Fax (03) 9662 7555 www.publish.csiro.au

Email: publishing.sales@csiro.au

15 Attachment G - Notes About This Report

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

Engineering Reports - Limitations

Engineering reports are based on information that may be gained from limited subsurface site testing and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretative rather than factual documents, limited to some extent by the scope of information on which they rely.

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.
- o Changes in guidelines, standards and policy or interpretation of guidelines, standards and policy by statutory authorities.

- The actions of contractors responding to commercial pressures.

- Actual conditions differing somewhat from those inferred to exist, because no professional, no matter how qualified, can reveal precisely what is hidden by earth, rock and time.

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

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

Subsurface Conditions - Changes

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

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

Subsurface Conditions - Site Anomalies

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

Report Use by Other Design Professionals

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

Subsurface Conditions – Geo-environmental Issues

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

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

Responsibility

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

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

Site Inspections

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

Definitions

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

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

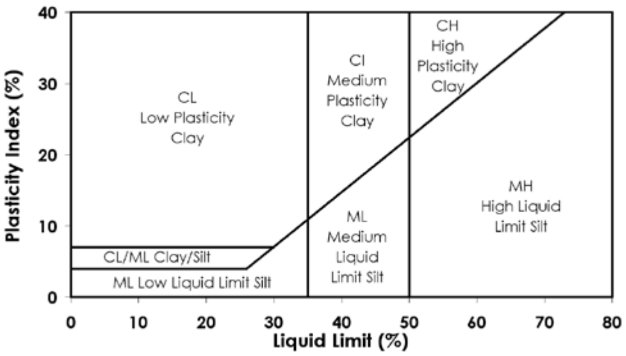
Particle Size

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

Division	Subdivision	Size (mm)
BOULDERS		>200
COBBLES		63 to 200
GRAVEL	Coarse	20 to 63
	Medium	6 to 20
	Fine	2.36 to 6
SAND	Coarse	0.6 to 2.36
	Medium	0.2 to 0.6
	Fine	0.075 to 0.2
SILT		0.002 to 0.075
CLAY		< 0.002

Plasticity Properties

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



Moisture Condition

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

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

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

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

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

Density of Granular Soils

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

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

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

Minor Components

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

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

Soil Data

martens consulting engineers

Symbols for Soils and Other

SOILS

- COBBLES/BOULDERS
- GRAVEL (GP OR GW)
- SILTY GRAVEL (GM)
- CLAYEY GRAVEL (GC)
- SAND (SP OR SW)
- SILTY SAND (SM)
- CLAYEY SAND (SC)
- SILT (ML OR MH)
- ORGANIC SILT (OH)
- CLAY (CL, CI OR CH)
- SILTY CLAY
- SANDY CLAY
- PEAT
- TOPSOIL

OTHER

- FILL
- TALUS
- ASPHALT
- CONCRETE

Unified Soil Classification Scheme (USCS)

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 63 mm and basing fractions on estimated mass)					USCS	Primary Name
COARSE GRAINED SOILS More than 50 % of material less than 63 mm is larger than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.0 mm.	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	Gravel
				Predominantly one size or a range of sizes with more intermediate sizes missing	GP	Gravel
			GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty Gravel
				Plastic fines (for identification procedures see CL below)	GC	Clayey Gravel
		SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of intermediate sizes missing.	SW	Sand
				Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Sand
			SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	SM	Silty Sand
				Plastic fines (for identification procedures see CL below)	SC	Clayey Sand

		IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM					
		DRY STRENGTH (Crushing Characteristics)	**DILATANCY**	**TOUGHNESS**	**DESCRIPTION**	**USCS**	**Primary Name**
FINE GRAINED SOILS More than 50 % of material less than 63 mm is smaller than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	None to Low	Quick to Slow	None	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	Silt
		Medium to High	None	Medium	Inorganic clays of low to medium plasticity [1], gravely clays, sandy clays, silty clays, lean clays	CL [2]	Clay
		Low to Medium	Slow to Very Slow	Low	Organic slits and organic silty clays of low plasticity	OL	Organic Silt
		Low to Medium	Slow to Very Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	Silt
		High	None	High	Inorganic clays of high plasticity, fat clays	CH	Clay
		Medium to High	None	Low to Medium	Organic clays of medium to high plasticity	OH	Organic Silt
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	Peat

Notes:

1. Low Plasticity – Liquid Limit W_L < 35 % Medium Plasticity – Liquid limit W_L 35 to 60 % High Plasticity - Liquid limit W_L > 60 %.
2. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.

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Soil Agricultural Classification Scheme

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

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

Rock Data

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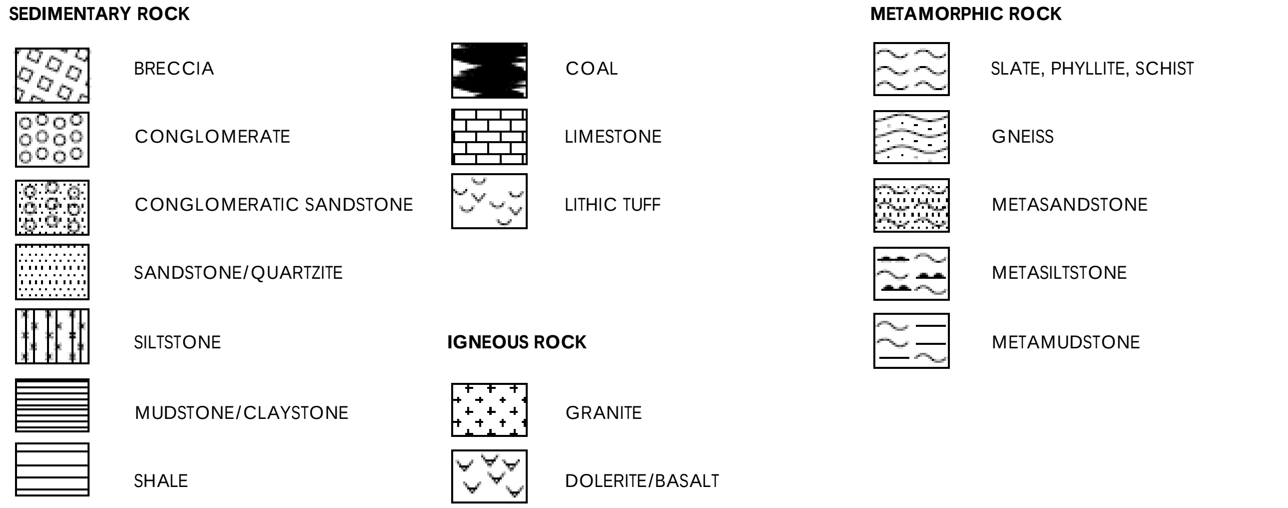
Symbols for Rock



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

Rock Strength

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

Term	Is (50) MPa	Field Guide	Symbol
Very low	>0.03 ≤0.1	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VL
Low	>0.1 ≤0.3	A piece of core 150mm long x 50mm diameter may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	L
Medium	>0.3 ≤1.0	A piece of core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	M
High	>1 ≤3	A piece of core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife.	H
Very high	>3 ≤10	A piece of core 150mm long x 50mm diameter may be broken readily with hand held hammer. Cannot be scratched with pen knife.	VH
Extremely high	>10	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	EH

Degree of Fracturing

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

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

Rock Core Recovery

TCR = Total Core Recovery

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

SCR = Solid Core Recovery

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

RQD = Rock Quality Designation

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

Rock Strength Tests

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

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

● Unconfined compressive strength (UCS) (MPa)

Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)

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

Planarity

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

Roughness

Pol	Polished
Sl	Slickensided
Sm	Smooth
Ro	Rough
VR	Very rough

Thickness

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

Coating or Filling

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

Inclination

Inclination of defect is measured from perpendicular to and down the core axis.

Direction of defect is measured clockwise (looking down core) from magnetic north.

Sampling

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

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

Undisturbed samples may be taken by pushing a thin-walled sampling tube, e.g. U_{50} (50 mm internal diameter thin walled tube), into soils and withdrawing a soil sample in a relatively undisturbed state. Such samples yield information on structure and strength and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

Drilling / Excavation Methods

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

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

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

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

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

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

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

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

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

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

In-situ Testing and Interpretation

Cone Penetrometer Testing (CPT)

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

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

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

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

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

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

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

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

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

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

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

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

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

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DRILLING / EXCAVATION METHOD

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

SUPPORT

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

WATER

▽ Water level at date shown

▷ Water inflow

◁ Partial water loss

◀ Complete water loss

GROUNDWATER NOT OBSERVED (NO) — The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

GROUNDWATER NOT ENCOUNTERED (NX) — The borehole/test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/test pit been left open for a longer period.

PENETRATION / EXCAVATION RESISTANCE

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

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

SAMPLING

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

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

TESTING

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

Notes:

RW	Penetration occurred under 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		