



Geotechnical Investigation

Proposed Commercial Development Heritage Drive, Chisholm-Whitewater Park

Report Ref: G0244-GI-001-Rev0

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We confirm that the following report has been produced for Maitland City Council, based on the described methods and conditions within.

For and on behalf of Hunter Civilab,



Daniel Barnes

Principal Geotechnical Engineer

Executive Summary

The following report details the geotechnical investigation undertaken by Hunter Civilab (HCL) under the request of Maitland City Council. The investigation was undertaken at Heritage Drive, Chisholm-Whitewater Park on the 27th of July 2023 and consisted of a desktop study, a visual site assessment, intrusive excavations, and testing.

The desktop study indicated that the site lies within an area of no known occurrences of acid sulfate soils.

The desktop study also indicated that the site does not lie within a mine subsidence district.

The site was bordered by Heritage Drive to the east, Tigerhawk Drive to the north, Whitewater Park to the south, Whitewater Street and an existing pond to the west.

The subsurface profile generally consisted of up to 0.4 m of silty sand / sandy silt / clayey sandy silt topsoil, overlying up to 0.7 m of gravelly sand / clayey gravelly sand fill (BH1), overlying Silty Clay/Sandy Clay/Silty Sandy Clay residual soils and Clayey Sand, Extremely Weathered Sandstone material.

A site classification was undertaken based on the laboratory testing results and the subsurface profile encountered at the time of investigation. The results indicated a Class P site with a reactivity of Class M, having a characteristic free surface movement of 20-40 mm. Therefore, a site classification of Class P-M is recommended for the site.

The site would be suitable for the use of both shallow and deep footings. Refer to **Section 7.2** for footing details and recommended allowable bearing capacity.

A detailed pavement investigation and design was undertaken in accordance with Maitland City Council engineering guidelines, Austroads Design Guide 2017 and APRG21 "A Guide to the Design of New Pavements for Light Traffic", 2006. Based on the results of the laboratory testing a soaked subgrade CBR of 5% was adopted for the purpose of the pavement design. For flexible pavement design a traffic loading of 5×10^5 DESA's was adopted for a 20-year design life based on COUNCIL engineering guidelines. For rigid pavement design a traffic loading of 5×10^6 DESA's was adopted for a 20-year design life based on Maitland City Council engineering guidelines. Flexible and rigid pavement design options were determined for the proposed paved area and a summary of the recommended thickness design/s can be seen below in **Table 1**.

Table 1 - Summary of flexible and rigid pavement designs

Flexible pavement option	Rigid pavement option
30 mm of Asphalt wearing course	125 mm of Concrete basecourse
150 mm of granular basecourse	100 mm of granular sub-base
180 mm of granular sub-base	

Refer to **Section 8** for the detailed pavement design including material and compaction requirements.

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Annex A Borehole Location Plan

Annex B Borehole & DCP Log Report

Annex C Hunter Civilab Laboratory Test Reports

Annex D BTF 18-2011 – CSIRO – Foundation Maintenance & Footing Performance – A Homeowner’s Guide

1 Introduction

At the request of Maitland City Council, Hunter Civilab (HCL) have carried out a geotechnical investigation at Heritage Drive, Chisholm-Whitewater Park. It is understood that the proposed development is to consist of the construction of a new community centre and associated car parking. The investigation works were undertaken in accordance with HCL services agreement Q1922, dated the 13th of July 2023.

The purpose of the investigation was to provide recommendations on the following:

- surface and sub-surface conditions
- geotechnical laboratory testing results
- site classification to AS 2870-2011
- alternative footing types and foundation design parameters
- temporary and permanent safe batter slopes
- retaining wall options
- retaining wall design parameters
- flexible pavement thickness design in accordance with Austroads, Council, TfNSW design guidelines
- rigid (concrete) pavement thickness design in accordance with Austroads, Council, TfNSW design guidelines
- earthworks advice including subgrade preparation requirements, geotechnical suitability of excavated material for reuse as fill and excavatability assessment

This report provides details of the investigation, laboratory testing and provides recommendations for the proposed development.

2 Site Description

The site was located at Heritage Drive, Chisholm-Whitewater Park. The site was bordered by Heritage Drive to the east, Tigerhawk Drive to the north, Whitewater Park to the south, Whitewater Street and existing pond to the west.

At the time of investigation, existing development at the site included an asphalt sealed carpark and the Whitewater Park children's playground.

Existing vegetation at the site consisted of short kept grass with growing trees toward the eastern boundary, and very dense long grass toward the western boundary.

Topographically the site consists of a minor slope towards the western and north-western boundaries.

A photograph showing the surface conditions at the site is shown below in Figure 1.



Figure 1 - Photograph of site

3 Desktop Review

3.1 Geological & Soil Landscape Setting

Reference to the 1:250,000 Newcastle Geological Map indicates that the site sits on the border of the Tomago Coal Measures and Maitland Group and Mulbring Siltstone subgroup with rock types typically including shale, mudstone, sandstone, tuff, coal and conglomerate.

Reference to the 1:100,000 Newcastle Soil Landscape Sheet indicates that the site is underlain by the Beresfield Landscape. The Landscape is characterized by undulating rises to rolling low hills on Permian sediments to the southeast of the Hunter Region. Slope gradients are found to be between 3 to 15% on local reliefs of less than 50m with elevations between 10 to 90m. The soil is known to consist of shallow to moderately deep, imperfectly drained Brown and Yellow Kurosols (Yellow Podzolic Soils and Soloths); and moderately deep imperfectly drained Red, Brown, and Yellow Kurosols (Red and Yellow Podzolic Soils and Soloths). The vegetation on the landscape is comprised of partially cleared tall-open forest.

3.2 Acid Sulfate Soils Risk Maps

Reference to the NSW Office of Environment and Heritage's online database 'ESPADE' indicates that the site lies in an area of no known occurrences of acid sulfate soils.

3.3 Mine Subsidence

Reference to Subsidence Advisory NSW Mine District Maps indicates that the site does not lie within a Mine Subsidence District.

4 Fieldwork Methodology

Fieldwork was undertaken on the 27th of July 2023 and consisted of:

- underground utility service clearances using a Telstra accredited locator.
- a visual assessment of the existing surface of the site and surrounding area
- locating borehole locations by approximate measurements from existing site features
- the drilling of 5 x boreholes (BH1 – BH5) to depths of up to 2.5 m using mechanical rotary methods
- the driving of 5 x Dynamic Cone Penetrometer (DCP) probes at BH locations (BH1 – BH5) to depths of up to 2.3 m

Laboratory testing consisted of:

- 1 x Shrink Swell Index test.
- 1 x Atterberg Limits and Linear Shrinkage test
- 2 x California Bearing Ratio tests.

The fieldwork was supervised by an engineering geologist / geotechnical engineer from HCL who logged the subsurface conditions at bore hole locations in accordance with AS1726-2017 and collected bulk disturbed, disturbed, and undisturbed soil samples for laboratory testing and soil identification purposes.

5 Subsurface Conditions

The subsurface soil conditions encountered at test locations are presented in detail in the borehole logs and have been summarised into the following units:

UNIT 1A – TOPSOIL:

- Silty SAND, dark grey / brown, with organics
- Sandy SILT / Clayey Sandy SILT, dark grey / brown, with gravel, with organics

UNIT 1B – FILL (GENERAL):

- Gravelly SAND / Clayey Gravelly SAND, pale grey / brown (floater)

UNIT 2A – RESIDUAL SOIL:

- Silty CLAY, dark brown / dark grey, mottled orange, stiff
- Sandy CLAY, pale grey / white, mottled orange / red, with extremely weathered sandstone inclusions, stiff to very stiff
- Silty Sandy CLAY, dark grey / brown, mottled orange, stiff to firm

UNIT 2B – RESIDUAL (Extremely Weathered Material):

- Clayey SAND, pale grey / brown / red / white, trace fine gravel, very dense.

A summary of the soil unit depths encountered in each borehole are presented below in Table 5.1.

Table 5.1 - Summary of the soil unit depths encountered.

Borehole	Borehole Depth from Surface (m)*	Unit Depth (m)			
		UNIT 1A	UNIT 1B	UNIT 2A	UNIT B
BH1	2.2	0.0–0.1	0.1–0.7	0.7–1.8	1.8–2.2
BH2	2.5	0.0–0.3	NE	0.3–1.7	1.7–2.5
BH3	1.5	0.0–0.4	NE	0.4–0.9	0.9–1.5
BH4	1.2	0.0–0.1	NE	0.1–1.2	NE
BH5	1.1	0.0–0.1	NE	0.1–1.1	NE

Note: NE – Not Encountered

* All borehole depths encountered refusal on inferred weathered sandstone. Borehole Surface RL was not measured.

Neither groundwater nor surface water were encountered at the site. It is noted groundwater conditions are dependent on a range of factors including soil permeability and climatic conditions and will therefore vary with time.

Refer to **Annex A** for the borehole location plan and **Annex B** for detailed borehole logs.

6 Laboratory Test Results

1 x undisturbed, 1 x disturbed, and 2 x bulk disturbed samples were recovered from the boreholes for geotechnical laboratory testing. The samples were transported to HCL's NATA accredited soil testing laboratory for analysis.

The laboratory test results are summarised below in **Table 6.1**, **Table 6.2** and **Table 6.3**.

Table 6.1 - Shrink Swell Index test results.

Borehole	Depth (m)	Soil Description	I _{ss} (%)
BH2	0.4–0.65	Silty CLAY	3.0

Table 6.2 - Atterberg Limit test results

Borehole	Depth (m)	Soil Description	Plasticity Index (%)	Linear Shrinkage (%)
BH3	0.9–1.2	Clayey SAND	15	7.5

Table 6.3 - California Bearing Ratio test results.

Borehole	Depth (m)	Soil Description	FMC (%)	OMC (%)	MDD (t/m ³)	Swell (%)	CBR (%)
BH4	0.2 – 1.0	Silty Sandy CLAY	18.3	18.0	1.76	0.5	5.0
BH5	0.2 – 0.9	Silty Sandy CLAY	21.0	19.5	1.68	1.5	4.0

Laboratory test results from the soil samples can be found in **Annex C**.

7 Comments and Recommendations

7.1 Site Classification

AS2870-2011 applies to residential construction, however, may have some relevance to the proposed building on this site. The principles of design, construction, and maintenance on which AS2870-2011 is based, should be taken into consideration for the development of the site.

7.1.1 Background Information

Site classification is based off the characteristic surface movements encountered at the site due to the moisture variations within the soil profile. Characteristic surface movements are estimated in accordance with AS2870-2011 “Residential Slabs & Footings”. Surface movement calculation take into consideration the depth of the soil profile layers, the soil reactivity and the soil suction depth.

The site classification based on characteristic surface movements are summarised below in **Table 7.1**.

Table 7.1 - Summary of AS2870-2011 characteristic surface movement & site classification

Characteristic Surface Movement (γ_s) (mm)	Site Classification AS2870-2011	Underlying Soil / Geology
0	Class A	SAND or ROCK site (non-reactive)
0 – 20	Class S	CLAY (slightly reactive)
20 – 40	Class M	CLAY (moderately reactive)
40 – 60	Class H1	CLAY (highly reactive)
60 – 75	Class H2	CLAY (highly reactive)
> 75	Class E	CLAY (extremely reactive)

Sites subjected to deep-seated moisture change are modified with the addition of “-D”. As defined by AS2870-2011 and other sites should be classified as a Class P (Problem) site. Sites classified as Class P have one or more of the following geotechnical related issues:

- inadequate bearing capacity

- expected excessive foundation settlement due to loading on the foundation.
- significant moisture variations
- mine subsidence risk
- slope stability risk
- erosion issues
- greater than 0.8m of fill for sand sites and greater than 0.4m for other sites (in general)

7.1.2 Site Classification Recommendations

The proposed development should be designed in accordance with AS2870-2011 “Residential Slabs and Footings”. Based on the visual inspection, dynamic cone penetrometer tests and soil profile shown above in **Section** Error! Reference source not found., the site classification is summarised below in **Table 7.2**.

Table 7.2 - Site classification & characteristic surface movement (y_s)

Site Classification	Site Reactivity	Characteristic Surface Movement (y_s) (mm)
Class P	Class M	20 – 40

The site was classified as a Class P due to the presence of fill >0.4m (clay site), presence of trees that may create abnormal moisture conditions.

Based on the subsurface profile and the results of the laboratory testing a site reactivity of Class M has been assigned to the Class P site.

The estimated characteristic surface movement assumes fill has been present on site for at least five years. It should be noted that in the event of placement of additional fill or cut into the existing profile, a more severe characteristic surface movement would apply.

Classification of the site **has not** taken into account the effects of abnormal moisture conditions listed in **Section** Error! Reference source not found. and **Section 7.1.3** below. If the site undergoes any earthworks operations, the site shall be reclassified in accordance with AS2870-2011.

7.1.3 Abnormal Moisture Effects

Abnormal moisture conditions in the foundation can be caused by the following:

- existing development
- leaking water services
- prolonged periods of draught or heavy rainfall
- trenches or other man-made water courses
- poor roof plumbing or obstruction to the roof plumbing system
- poor rainfall runoff control
- corroded gutters or downpipes

Abnormal moisture conditions specified above can cause adverse effects to the development's foundation such as:

- erosion significantly effecting the lateral and founding support of the structure's footing system.
- saturation of the founding material which can cause a significant decrease in the strength of the founding material.
- shrinkage creating subsidence of the founding material and causing additional stresses within the building structure.
- swelling which creates an upward force in the footings which causes additional stresses within the building structure.

7.1.4 Effects from Trees

The existence of trees within or adjacent to the building footprint can cause significant soil movement due to the following:

- roots growing within the foundation and causing an upward force on footings.
- roots drawing in and absorbing the moisture below a footing system causing subsidence due to shrinkage of the soil volume.

The site should take into account the tree score effect in accordance with and designed to AS2870-2011. The site was found to have a "Moderate" tree score effect and has not been taken into consideration in the characteristic surface movement calculation.

7.2 Footing Recommendations

The site is suitable for the use of both shallow strip, pad, and pier foundation systems dependant on the development and structural bearing pressure required. Alternatively, piled foundation solutions could be considered. Recommended bearing pressures are provided in **Section 7.2.1** and **Section 7.2.2** below.

7.2.1 Shallow Footings

A maximum allowable bearing capacity of 150kPa is recommended at the site for shallow level footings founded within very stiff clay soils and/or medium dense to dense sand soils, below topsoil, or other deleterious material (e.g., root affected soils, soft / loose soils, silt soils, uncontrolled fill etc).

If weathered rock is exposed at the base of the excavation of footings it is recommended that the rest of the footing system be pried / taken to bedrock to reduce the risk of differential settlement.

Bearing pressures of all exposed foundation areas should be confirmed at the time of earthworks and prior to concrete pour by a qualified geotechnical engineer.

The footing systems must be designed by a structural engineer in accordance with engineering principles and AS 2870 - 2011 "Residential Slabs and Footings" for no less than the minimum requirements for the site classification and soil reactivity given as per **Section 7.1.2** above.

The bearing pressures presented above have been correlated from Dynamic Cone Penetration (DCP) tests and should be considered as estimates only.

7.2.2 Deep Footings

Bored piers socketed at least 0.5 m into Unit 2B very stiff to hard residual soil (extremely weathered sandstone) could be proportioned for an allowable bearing pressure of 300kPa. The estimated bearing pressure has been inferred from falling weight penetrometer testing.

Care should be taken during construction to ensure that the base of the bored pile holes is clean and free from loose debris or water prior to placement of concrete. Pile hole inspections should be undertaken during construction by a suitably qualified geotechnical engineer to confirm that the appropriate foundation stratum is achieved.

All pile types should be suitably protected against decay or corrosion, taking account of the subsurface conditions, water table fluctuations and site-specific conditions (existing chemical concentrations at the site).

7.2.3 Footing Construction

Pile installation should be accompanied by appropriate verification inspections and testing, for example, load test results, pile driving records, grout pressure records, pile integrity testing, in keeping with the requirements of AS 2159-2009.

All footings should be excavated, cleaned, and inspected by a qualified Geotechnical Engineer. Concrete should be poured with minimal delay. If delays in pouring mass concrete footings is anticipated, a concrete blinding layer should be provided to protect the foundation material.

Should softening of exposed foundation occur, the effected material should be over excavated and backfilled to design footing level by engineered fill or mass concrete.

7.2.4 Ongoing Footing Maintenance

Foundations including effective site drainage are required to be maintained over the life of the development to ensure footing performance. Refer to **Annex D** for the following:

- BTF 18-2011- CSIRO - Foundation Maintenance and Footing Performance – A Homeowner's Guide

7.3 Retaining Walls

Recommended site soil parameters for retaining wall design at the site are provided in **Table 7.3** below.

Table 7.3 - Recommended retaining wall design soil parameters

Parameter	Supported material	
	Unit 2A Silty Clay / Sandy Clay Stiff to Very Stiff	Unit 2B Clayey SAND (Extremely Weathered Sandstone Material) Very Stiff to Hard
γ (kN/m ³)	19	20
Φ' (°)	25	28
C' (kPa)	5	5
C_u (kPa)	75	100
K_a	0.30	0.25
K_p	2.00 (Drained) 150 kPa (Undrained)	2.50 (Drained) 200 kPa (Undrained)
K_o	0.57	0.53

Legend:

γ – unit weight

Φ' – angle of friction

C' – drained cohesion

C_u – undrained cohesion

K_a – coefficient of active earth pressure

K_p – coefficient of passive earth pressure

K_o – coefficient of at rest earth pressure

Parameters shown assume horizontal and free draining granular backfill behind the retaining wall.

The parameters above are unfactored and appropriate factors of safety should be used in design.

The design of retaining walls should account for the separate hydrostatic water pressures behind the walls unless adequate subsurface and surface drainage is provided behind the wall to prevent build-up of water pressure including the provision for maintenance (i.e. flushing points).

The pressure distribution given above assumes that no surcharging of the walls occurs from nearby footings. If the footings behind retaining walls from existing retaining walls, or proposed structures are not taken below the retaining wall zone of influence (which is approximated by a line drawn at 45° above

the horizontal from the base of the wall) or to low strength rock or stronger, than additional allowance should be made for the load from the footings.

Cantilever walls should not be used to support nearby building foundations or underground services.

8 Pavement Thickness Design

8.1 Standards and Specifications Adopted for Design

Pavement design was completed in accordance with:

- Maitland City Council Engineering Guidelines.
- AP-T36-06 Pavement Design for Light Traffic: A supplement to the Austroads Pavement Design Guide, 2006.
- Austroads Design Guide 2017.

CIRCLY 7 was utilised for forward calculations to determine critical strains and pavement damage.

Adopted design CBR:

Based on laboratory test results as described in **Section** Error! Reference source not found., sub-grade soaked CBR values adopted were as follows:

- Soaked CBR of 5% for proposed new pavement options.

Adopted Traffic Loadings:

As per Maitland City Council Engineering Guidelines / Austroads Design Guide 2017 the following traffic loadings were adopted for design:

Flexible Pavements:

- Local Road – Primary or Secondary 5×10^5 Design ESA's.

Rigid Pavements:

- Local Road – Primary or Secondary 5×10^6 Design ESA's.

Adopted Design Life:

As per Maitland City Council Engineering Guidelines / Austroads Design Guide 2017 the following design life is was adopted for design:

Flexible Pavements:

- 20 year design life

Rigid Pavements:

- 30 year design life

Recommended Pavement Material Specifications to be adopted for construction are as follows:

Lightly loaded pavements:

- $<1 \times 10^6$ ESA traffic: ARRB SR 41 / AP-T36-06

8.2 Flexible Pavement Thickness Design

The recommended flexible pavement thickness, pavement material and compaction specification are presented in **Table 7.5** and **Table 7.6**.

Table 7.4 – Summary of flexible pavement minimum thickness design

Pavement	Depth (mm)
AC10 Wearing course (to Council spec)	30
Primer seal	(10)
Base course (DGB20 or equivalent)	150
Subbase (DGS40 or equivalent)	180
Total thickness (mm)	360
Subgrade CBR	5.0%

*a construction tolerance of 10mm should be allowed for above the minimum thickness

NOTE: ASPHALT THICKNESS for lightly loaded traffic Check Table 12.1 (AUSTROADS): Typical asphalt layer thicknesses.

Table 7.5 – Flexible pavement compaction criteria

Pavement	Compaction criteria
AC10 wearing course (to Council spec)	NA
Primer seal	NA
Base course (DGB20 or equivalent)	98% Modified (AS 1289.5.2.1)
Subbase (DGS40 or equivalent)	95% Modified (AS 1289.5.2.1)
Select Fill Subgrade	100% Standard (AS 1289.5.1.1)
Subgrade	100% Standard (AS 1289.5.1.1)

8.3 Rigid Pavement Thickness Design

The recommended rigid pavement thickness, pavement material and compaction specifications are presented in **Table 7.6** and **Table 7.7** below.

Table 7.6 – Summary of rigid pavement minimum thickness design

Pavement	Depth (mm)
Base course (32 MPa concrete with SL72 reinforcement)	125
Subbase (crushed rock subbase min soaked CBR 80%, max PI = 6%, or equivalent)	100
Total thickness (mm)	225
Subgrade CBR	5.0%

*a construction tolerance of 10mm should be allowed for above the minimum thickness;

**or as specified by Council

NOTE: Refer Min Subbase thickness, Min Rigid Pavement thickness , min dowel bar diameter (5.2 Maitland City Council, Manual of Engineering Standards or AP-T36/06)

Table 7.7 – Rigid pavement compaction criteria

Pavement	Compaction Criteria
Base course (32 MPa concrete with SL72 reinforcement)	NA
Subbase (crushed rock subbase min soaked CBR 80%, max PI = 6%, or equivalent)	98% Modified (AS 1289.5.2.1)
Subgrade	100% Standard (AS 1289.5.1.1)

8.4 Pavement Drainage and Pavement Interfaces

The pavement thickness is dependent on the provision of adequate surface and subsurface drainage as specified by a qualified civil or pavement engineer. It is recommended that an intra pavement subsoil drain be installed at the interfaces between pavement types.

Where new pavement construction abuts existing pavement, care shall be taken to create a clean vertical construction joint along with a benched transition zone. The transition zone should be across a 0.5m distance and benched to tie in with existing profiles.

It is recommended that all construction joints should be located outside of wheel paths, and where practical should be located in the centre of the lanes or along edge lines.

It should be noted that when variable pavements are abutted then the potential for localised failure is greater. Care should be exercised in the placement and compaction of the subgrade and pavements in this area to maximise the performance of the pavement.

Consideration should also be given to sealing any cracks that may develop between existing and new pavements, benching to tie in pavements. The use of a strain relieving membrane at the interface may be appropriate in some cases.

8.5 Recommendations During Construction

Following excavation, site visits should be made by an experienced geotechnical engineer to inspect exposed subgrade and pavement conditions.

8.6 Earthworks

Any earthworks conducted at the site should be controlled in accordance with AS3798-2007 and guided by the sections below.

8.6.1 Site Preparation

New Pavement Construction:

It is recommended that the following be undertaken where controlled filling is to be undertaken:

- remove all topsoil, root effected zones, material assessed as unsuitable and other deleterious zones (noting the stripped soil is not considered suitable as engineered fill but may be considered for landscaping purposes)
- exposed suitable foundation areas should then be ripped 300mm and re-compacted to 100% standard maximum dry density (SMDD) at $\pm 2\%$ of optimum moisture content (OMC)
- the foundation area should then be proof rolled under the supervision of an experienced geotechnical consultant and any soft spots / heaving areas identified. If identified, these areas should be over excavated under the direction of the geotechnical consultant and replaced with engineered fill.

8.6.2 Controlled Fill

Any earthworks conducted at the site should be controlled in accordance with AS3798-2007. Based on the soil profile shown above in **Section** Error! Reference source not found., visual observations and in-situ Dynamic Cone Penetrometer (DCP) testing, the material encountered at the site is deemed suitable for controlled fill. If the sub-surface conditions encountered at the site during construction differ from those discussed in **Section** Error! Reference source not found. VC should be consulted to determine if the material is suitable for controlled fill. Similarly, any won material imported from external sites should consult VC to determine if the fill is suitable for controlled fill.

8.6.3 Compaction Criteria

Fill material should be compacted in near-horizontal uniform layers with a maximum compacted thickness of 300mm. It is important to ensure layers are placed in such a way that provides adequate drainage and prevent ponding during construction. The thickness of fill placed during construction should take into account the compaction equipment available.

The moisture of the fill material should be controlled within a specified range of OMC in order to achieve the compaction criteria. In general, soils should be compacted within a moisture range of $\pm 2\%$ of OMC.

For commercial developments the following compaction criteria applies:

- cohesive soils – 98% Minimum Density Ratio (standard compactive effort)
- non-cohesive soils – 75% Minimum Density Index

Where fill is placed on sloping ground, any slope wash/deleterious should be removed and the fill should be adequately keyed/benched into the sloping ground profile.

Geotechnical inspections and testing should be performed in accordance with Level 1 procedures with reference to AS3798-2007.

8.6.4 Excavations Conditions

Excavations within the fill, natural soils and extremely low to very low strength rock that was encountered during the investigations is thought to be achievable with conventional earthmoving equipment such excavators, backhoes, and dozers. Very low to low strength rock may also require ripper

tyes attached to excavator arms or dozers for effective excavation. Rock of low strength or greater may possibly require a 12-tonne excavator (or greater) with rock ripper or hydraulic rock hammer, depending on the degree of strength and fracturing in the rock. Excavations in rock would require minimising vibration to neighbouring residences and structures, else other methods may be required (for example pre-drilling the rock, rock sawing using diamond wire saw equipment, grinding or engaging a rock breaking and removal specialist).

Bored piers could be drilled using a 12-tonne excavator or greater with an attached auger. It is recommended that the bottom of bored pier holes should be cleaned out with the excavator fitted with a bucket attachment.

Excavations should be conducted in accordance with The Safe Work Australia “Excavation Work” Code of Practice October 2018.

<https://www.safeworkaustralia.gov.au/doc/model-codes-practice/model-code-practice-excavation-work>

Excavations can seriously affect the stability of adjacent buildings. Careful consideration must be taken in order to prevent the collapse of partial collapse of adjacent structures.

Construction material and equipment should not be placed within the zone of influence of an excavation unless a suitably qualified geotechnical engineer has designed ground support structures to withstand these loads. The zone of influence is dependent on the material encountered at the site and is the area in which possible failures can occur.

Refer to Council development guidelines before conducting any excavation works.

8.7 Batter Slopes

8.7.1 Temporary Batter Slopes

Temporary excavations in natural material or extremely low to very low strength rock may be near vertical provided that:

- the depth does not exceed 1.5m
- they are open for no more than 24hrs.
- no surcharge loading is applied to the surface within 2.5m of the excavation.
- no one enters the excavation e.g., workers.

All other temporary batter slopes during construction should not exceed 1H:1V in soils and 1H:4V in rock and benched, planned and managed in accordance with Safe Work Australia “Excavation Work” Code of Practice October 2018.

Specific geotechnical assessment will be required where temporary excavations exceed 2 m vertical height.

8.7.2 Permanent Batter Slopes

Recommended permanent batter slopes in general are as follows:

- 2H:1V in cohesive soils (e.g. clays) or extremely to very low weathered rock else retained by an engineered retaining wall
- 3H:1V in non-cohesive soils (e.g. sands) else retained by an engineered retaining wall
- 1H:1V in low strength rock or greater (permanent rock batters may be steepened to near vertical – subject to inspection by a qualified geotechnical engineer)

Flatter slopes of 3H:1V or flatter and/or provision of benching would be required for maintenance requirements and should be topsoiled (at least 100 mm thickness) and vegetated. The optimum type of grass species selected to promote growth should be based on local climatic conditions and soil types. Also, regular watering during establishment and during dry weather periods along with initial fertilisation to promote growth is recommended.

Specific geotechnical assessment will be required where permanent batters exceed 2 m vertical height.

9 Report Limitations

This report has been prepared by HCL for the specific site and purposes described within this report. HCL will accept no responsibility or liability for the use of this report by any third party, without the express consent of HCL or the Client, or for use at any other site or purpose than that described in this report.

This report and the services provided have been completed in accordance with relevant professional and industry standards of interpretation and analysis. This report must be read in its entirety without separation of pages or sections and without any alterations, other than those provided by HCL.

The scope of the investigation described in this report is based on information and plans provided to HCL by the Client as well as any additional limitations imposed by either the Client and / or site restraints. Such limitations may include but are not limited to budget restraints, the presence of underground services or accessibility issues to a site. Where the report has been prepared for a specific design proposal the information and interpretation may not be relevant if the design proposal is changed. HCL should be consulted if site plans, or design proposal is changed as the recommendations and / or opinions presented may not be suitable for the new revisions or variations made.

The conclusions, recommendations and opinions expressed within this report are subject to the specific conditions encountered and the limited geotechnical data gathered at the site during the time of the current investigation. The sub-surface conditions and results presented in this report are indicative of the conditions encountered at the discrete sampling and testing locations within the site at the time of the investigation and within the depths investigated. Variations in ground conditions may exist between the locations that were investigated, and the subsurface profile cannot be inferred or extrapolated from the limited investigation conducted by HCL. For this reason, the report must be regarded as interpretative, rather than a factual document.

Sub-surface conditions are subject to constant change and can vary abruptly as a result of human influences and /or natural geological and / or climatic processes and events. As such, conditions may exist at the site that could not be identified during or may develop after the current investigation has

been conducted and as such, may impact the accuracy of this report. HCL should be contacted for further consultation and site re-assessment should sub-surface conditions differ from those conditions identified in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by HCL.

Safety in design assessment is outside the current scope of work of this report. HCL has had no involvement in any design that relies upon the geotechnical advice contained in this report. HCL cannot be held liable for any loss of life or property damage arising from any hazards that arise from the geotechnical advice.

HCL recommends geotechnical reports older than 5 years from the date shown on the report, reports submitted for a previous (unrelated) development application on the site, or sites that have been altered by earthworks be reviewed by a qualified geotechnical consultant to confirm that the scope of the investigation undertaken for the report and the contents of the report are appropriate for the current development being proposed.

We are pleased to present this report and trust that the recommendations provided are sufficient for your present requirements. If you have any further questions about this report, please contact the undersigned.

For and on behalf of

Valley Civilab Pty Ltd, trading as Hunter Civilab

Reported by:

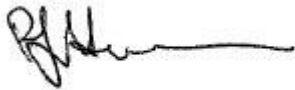


Jonacani Rabo

Senior Geotechnical Engineer

Bachelor of Engineering Technology (Mechanical)

GradCert of Engineering Technology (Civil)



Peter Hudson

Principal Geotechnical Engineer - Pavements

Bachelor of Engineering (Civil)

Grad Diploma of Municipal Engineering (Traffic & Transportation Planning)

Reviewed by:



Daniel Barnes

Principal Geotechnical Engineer

Bachelor of Engineering (Civil)/

Master of Applied Science (Civil)

References

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Annex A

Borehole Location Plan



Note: Overhead image taken from Nearmaps

Figure 1 – Overhead image at Heritage Drive, Chisholm (Whitewater Park) showing the approximate location of the Geotechnical boreholes.



Annex B



Hunter Civilab

Unit 3, 62 Sandringham Avenue Thornton NSW 2322

Phone: (02) 4966 1844

Geotechnical Log - Borehole

BH1

UTM : 56H	Driller Rig : 4WD Mounted Drill Rig	Job Number : G0244
Easting (m)	Driller Supplier : Hunter Civilab	Client : Maitland City Council
Northing (m)	Logged By : JR	Project : Eastern Precinct Community Centres Project, Chisholm
Ground Elevation : Not Surveyed	Reviewed By : KS	Location : Heritage Drive, Chisholm (Whitewater Park)
Total Depth : 2.2 m BGL	Date : 27/07/2023	Loc Comment :

Drilling Method	Water	DCP graph	Samples Disturbed	Depth (m)	Graphic Log	Classification Code	Material Description	Moisture	Consistency/Density	Soil Origin	Remarks				
Auger Drilling			ES	0.1		SM	Silty SAND, fine grained, dark grey / brown, (with organics)	M-D		Topsoil					
				0.3		SW	FILL: gravelly SAND, fine to medium grained, fine to medium sized gravel, pale grey / brown.	M		Fill					
				0.7		SC	FILL: clayey to gravelly SAND, fine to medium grained, fine to medium sized gravel, low plasticity clay, pale grey / brown, (floater).	M-D		Fill					
				1.0		CI-CH	Silty CLAY, medium to high plasticity, dark brown / dark grey, trace fine grained sand.	w > PL	St	Residual					
				1.4		CI-CH	As above, but pale grey / brown.		St-VSt	Residual					
				1.8		CL-CI	Sandy CLAY, low to medium plasticity, pale grey / white / mottled orange / red, fine grained sand, trace fine sized gravel, (with extremely weathered sandstone inclusions).	w < PL		Residual					
				2.2		SC SP	Clayey SAND, fine to medium grained, pale grey / brown / red, low plasticity clay, trace fine sized gravel, (extremely weathered sandstone material)	D	VD	Residual					
				BH1 refusal at 2.2 m											



Hunter Civilab

Unit 3, 62 Sandringham Avenue Thornton NSW 2322

Phone: (02) 4966 1844

Geotechnical Log - Borehole

BH3

UTM : 56H	Driller Rig : 4WD Mounted Drill Rig	Job Number : G0244
Easting (m)	Driller Supplier : Hunter Civilab	Client : Maitland City Council
Northing (m)	Logged By : JR	Project : Eastern Precinct Community Centres Project, Chisholm
Ground Elevation : Not Surveyed	Reviewed By : KS	Location : Heritage Drive, Chisholm (Whitewater Park)
Total Depth : 1.5 m BGL	Date : 27/07/2023	Loc Comment :

Drilling Method	Water	DCP Graph	Samples Disturbed	Depth (m)	Graphic Log	Classification Code	Material Description	Moisture	Consistency/Density	Soil Origin	Remarks	
Auger Drilling		1		1		ML	Sandy SILT, low plasticity, dark grey / brown, fine grained sand, (with organics).	w < PL		Topsoil		
		2	ES									
		2				SC SP	Clayey SAND, fine to medium grained, pale grey / orange / brown, low plasticity clay, trace fine sized gravel.	M	VD	Residual		
		10										
		15				CI-CH	Silty CLAY, medium to high plasticity, dark grey / brown / mottled orange.	w > PL	VSt	Residual		
		15										
		16				SC SP	Clayey SAND, fine to medium grained, pale grey white brown, low plasticity clay, (extremely weathered sandstone material)	M-D	VD	Residual		
		20	ES									
		23										
		22										
	20											
	R											
							BH3 refusal at 1.5 m					
				2								
				3								



Hunter Civilab

Unit 3, 62 Sandringham Avenue Thornton NSW 2322

Phone: (02) 4966 1844

Geotechnical Log - Borehole

BH4

UTM : 56H	Driller Rig : Ute Mounted Drill Rig	Job Number : G0244
Easting (m)	Driller Supplier : Hunter Civilab	Client : Maitland City Council
Northing (m)	Logged By : JR	Project : Eastern Precinct Community Centres Project, Chisholm
Ground Elevation : Not Surveyed	Reviewed By : KS	Location : Heritage Drive, Chisholm (Whitewater Park)
Total Depth : 1.2 m BGL	Date : 27/07/2023	Loc Comment :

Drilling Method	Water	DCP graph	Samples		Depth (m)	Graphic Log	Classification Code	Material Description	Moisture	Consistency/Density	Soil Origin	Remarks
			Disturbed	Bulk								
Auger Drilling		2	ES		0.1		OL	Clayey to sandy SILT, low plasticity, dark grey / brown, fine grained sand, with fine to medium sized gravel.	w < PL		Topsoil	
		2					CL-CI	Silty to sandy CLAY, low to medium plasticity, dark grey / brown, mottled orange, fine grained sand, with fine to medium sized gravel.	w ≈ PL	St-VSt	Residual	
		3										
		11										
		3										
		4										
		7										
		3		ES								
		3										
		3										
		12/20mm			CBR	1						
		R										
									BH4 refusal at 1.2 m			
					2							
					3							



Hunter Civilab

Unit 3, 62 Sandringham Avenue Thornton NSW 2322
 Phone: (02) 4966 1844

Geotechnical Log - Borehole

BH5

UTM : 56H	Driller Rig : Ute Mounted Drill Rig	Job Number : G0244
Easting (m)	Driller Supplier : Hunter Civilab	Client : Maitland City Council
Northing (m)	Logged By : JR	Project : Eastern Precinct Community Centres Project, Chisholm
Ground Elevation : Not Surveyed	Reviewed By : KS	Location : Heritage Drive, Chisholm (Whitewater Park)
Total Depth : 1.1 m BGL	Date : 27/07/2023	Loc Comment :

Drilling Method	Water	DCP graph	Samples		Depth (m)	Graphic Log	Classification Code	Material Description	Moisture	Consistency/Density	Soil Origin	Remarks
			Disturbed	Bulk								
Auger Drilling		2	ES		0.1		OL	Clayey to sandy SILT, low plasticity, dark grey / brown, fine grained sand, (with organics).	w < PL		Topsoil	
		2					CL-CI		Silty to sandy CLAY, low to medium plasticity, dark grey / brown, mottled orange, fine grained sand.	w > PL	S-F	
		1										
		2										
		0										
		12	ES									
		20										
		R										
			CBR		0.9		CL-CI	As above, but trace fine sized gravel.	w < PL	VSt	Residual	
					1							
								BH5 refusal at 1.1 m				
					2							
					3							

1 Introduction

The following notes are provided to be used in conjunction with Hunter Civilab's report to explain the terms and abbreviations used throughout the report.

2 Material Descriptions

Descriptions of soil and rock are generally in accordance with the Unified Soil Classification System and Australian Standard AS1726-2017 – Geotechnical Site Investigations. The descriptions of soil and rock are based on field tests and observations and are independent of any laboratory test results. The data presented throughout this report is as factual as possible. However, some interpretations is unavoidable.

2.1 Unified Soil Classification Group Symbols

Soils are generally assigned one of the following unified soil classification group symbols:

Table 2.1 - Unified Soil Classification Group Symbols

Symbol	Description	Symbol	Description
CH	Organic clays of high plasticity	Pt	Peat and other highly organic soils
OL	Organic silts of low plasticity	CH	Inorganic clays of high plasticity
MH	Inorganic silts of high plasticity	CI	Inorganic clays of low plasticity
ML	Inorganic silts of low plasticity	CL	Inorganic clays of low plasticity
GC	Clayey gravels	SC	Clayey sands
GM	Silty gravels	SM	Silty sands
GP	Poorly graded gravels	SP	Poorly graded sands
GW	Well graded gravels	SW	Well graded sands

2.2 Soil Description

Soils are described in general accordance with AS1726-2017, Section 6.1:

Table 2.2 - Particle Size Definitions (AS1726-2017, Table 1)

Component	Subdivision	Size (mm)
BOULDERS		>200
COBBLES		63 - 200
GRAVEL	Coarse	19 - 63
	Medium	6.7 - 19
	Fine	2.36 - 6.7
SAND	Coarse	0.6 - 2.36
	Medium	0.21 - 0.6
	Fine	0.075 - 0.21
SILT		0.002 - 0.075
CLAY		<0.002

Explanatory Notes & Abbreviations



Table 2.3 - Descriptive Terms for Accessory Soil Components (AS1726-2017, Table 2)

Designation of Components	In Coarse Grained Soils				In Fine Grained Soils	
	% Fines	Terminology	% Accessory Coarse Fraction	Terminology	% Sand / Gravel	Terminology
Minor	≤ 5	Add 'trace clay / silt' to description where applicable	≤ 15	Add 'trace sand / gravel' to description where applicable	≤ 15	Add 'trace sand / gravel' to description where applicable
	> 5, ≤ 12	Add 'with clay / silt' to description where applicable	> 15, ≤ 30	Add 'with sand / gravel' to description where applicable	> 15, ≤ 30	Add 'with sand / gravel' to description where applicable
Secondary	> 12	Prefix soil name as 'Silty' or 'Clayey', as applicable	> 30	Prefix soil name as 'Sandy' or 'Gravelly', as applicable	> 30	Prefix soil name as 'Sandy' or 'Gravelly', as applicable

Table 2.4 - Descriptive Terms for Plasticity (AS1726-2017, Table 6)

Descriptive Term	Range of Liquid Limit for SILT	Range of Liquid Limit for CLAY
Non-Plastic	Not applicable	Not applicable
Low Plasticity	≤ 50	≤ 35
Medium Plasticity	Not applicable	> 35 and ≤ 50
High Plasticity	> 50	> 50

Table 2.5 - Moisture Condition (AS1726-2017, Clause 6.1.7 (a))

Material	Term	Abbreviation	Field Description Terms
Coarse Grained Soil	Dry	D	Non-cohesive and free-running
	Moist	M	Soil feels cool, darkened in colour; Soil tends to stick together
	Wet	W	Soil feels cool, darkened in colour; Soil tends to stick together, free water forms when handling
Fine Grained Soil	Moist, dry of plastic limit	w < PL	Hard and friable or powdery
	Moist, near plastic limit	w ≈ PL	Soil can be moulded at a moisture content approximately equal to the plastic limit
	Moist, wet of plastic limit	w > PL	Soil usually weakened and free water forms on hands when handling
	Wet, near liquid limit	w ≈ LL	Near liquid limit
	Wet, wet of liquid limit	w > LL	Wet of liquid limit

Explanatory Notes & Abbreviations

Table 2.6 - Consistency Terms for Cohesive Soils (AS1726-2017, Table 11)

Consistency	Abbreviation	Field Guide to Consistency
Very Soft	VS	Exudes between the fingers when squeezed in hand
Soft	S	Can be moulded by light finger pressure
Firm	F	Can be moulded by strong finger pressure
Stiff	St	Cannot be moulded by fingers
Very Stiff	VSt	Can be indented by thumb nail
Hard	H	Can be indented with difficulty by thumb nail
Friable	Fr	Can be easily crumbled or broken into small pieces by hand

Table 2.7 - Relative Density of Non-Cohesive Soils (AS1726-2017, Table 12)

Relative Density	Abbreviation	Density Index (%)
Very Loose	VL	≤ 15
Loose	L	> 15 and ≤ 35
Medium Dense	MD	> 35 and ≤ 65
Dense	D	> 65 and ≤ 85
Very Dense	VD	> 85

Table 2.8 - Soil Origin (AS1726-2017, Clause 6.1.9)

Origin	Description
Residual Soil	Formed directly from in situ weathering of geological formations. These soils no longer retain any visible structure of fabric of the parent soil or rock material.
Extremely weathered material	Formed directly from in situ weathering of geological formations. Although this material is of soil strength, it retains the structure and / or fabric of the parent rock material.
Alluvial soil	Deposited by streams and rivers.
Estuarine soil	Deposited in coastal estuaries, and including sediments carried by inflowing rivers and streams, and tidal currents.
Marine soil	Deposited in a marine environment.
Lacustrine soil	Deposited in freshwater lakes.
Aeolian soil	Carried and deposited by wind.
Colluvial soil	Soil and rock debris transported down slopes by gravity, with or without the assistance of flowing water and generally deposited in gullies or at the base of slopes. Colluvium is often used to refer to thicker deposits such as those formed from landslides, whereas the term 'slopewash' may be used for thinner and more widespread deposits that accumulate gradually over longer geological timeframes.
Topsoil	Surface and / or near surface soils often, but not always, defined by high levels of organic material.
Fill	Material placed by anthropogenic processes.

Explanatory Notes & Abbreviations

2.3 Rock Description

Rocks are described in general accordance with AS1726-2017, Clause 6.2.

Table 2.9 - Rock Material Strength Classification (AS1726-2017, Table 19)

Strength	Abbreviation	Field Assessment
Very Low Strength	VLS	Material crumbles under firm blows with sharp end of pick; Can be peeled with sharp knife; Too hard to cut a triaxial sample by hand; Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	LS	Easily scored with a knife; Indentations 1mm to 3mm show in the specimen with firm blows of the pick point; Has dull sound under the hammer; A piece of core 150mm long by 50mm diameter may be broken by hand; Sharp edges of core may be friable and break during handling.
Medium Strength	MS	Readily scored with a knife; A piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	HS	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; Rock rings under hammer.
Very High Strength	VH	Hand specimen breaks with pick after more than one blow; Rock rings under hammer.
Extremely High Strength	EH	Specimen required many blows with geological pick to break through intact material; Rock rings under hammer.

Note: Material with strength less than 'Very Low' shall be described using soil characteristics. The presence of an original rock structure, fabric or texture should be noted, if relevant.

Table 2.10 - Classification of Material Weathering (AS1726-2017, Table 20)

Term	Abbreviation	Definition
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported. The material is described using soil descriptive terms.
Extremely Weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material structure and fabric of original rock are still visible. The material is described using soil descriptive terms.
Highly Weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognizable. Rock strength is significantly changed by weathering.
Moderately Weathered	MW	
Slightly Weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition of individual minerals or colour changes.

Explanatory Notes & Abbreviations

3 Drilling, In Situ Testing & Sampling Methodology

Table 3.1 - Drilling Methods





Abbreviation	Method
HA	Hand Auger
EX	Excavator bucket
AV	Auger drilling with steel 'V' bit
AT	Auger drilling with tungsten carbide bit
AB	Auger for bulk sampling
WB	Wash bore rotary drilling
NMLC	Rock coring using a NMLC core barrel
HQ	Rock coring using a HQ core barrel

Table 3.2 - Field Sampling and In Situ Testing Key

Abbreviation	In Situ Test	Abbreviation	Sample Type
DCP	Dynamic Cone Penetrometer (blows/100mm)	U	Undisturbed Sample (50mm)
PSP	Perth Sand Penetrometer (blow/100mm)	D	Disturbed Sample
SPT	Standard Penetrometer Test	B	Bulk Disturbed Sample
PP	Pocket Penetrometer Measurement (kPa)	ES	Environmental Sample
3,4,5 (example)	SPT blows per 150mm	W	Water Sample
N=9 (example)	STP 'blow count number' over 300mm after initial 150mm seating		
VS	Handheld Shear Vane Measurement (kPa)		
CPT	Cone Penetrometer Test		
IS50 (D) (A)	Point Load Index Value (reported in MPA) (D) = Diametric (A) = Axial		

4 Groundwater Observations

Table 4.1 - Water Comments Key

Water Comment	Symbol
Water Inflow	
Water / drilling fluid loss	
Measurement of standing water level	
Water Noted	



Annex C

Material Test Report

Report Number: P22767-216B
Issue Number: 1
Date Issued: 21/08/2023
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P22767
Project Name: Geotechnical Consulting Services
Project Location: Eastern Precinct Community Centres Project, Chisholm
Client Reference: G0244
Work Request: 12113
Sample Number: 23-12113A
Date Sampled: 27/07/2023
Dates Tested: 28/07/2023 - 03/08/2023
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Preparation Method: AS 1289.1.1 - Sampling and preparation of soils
Site Selection: Selected by Client
Sample Location: BH2, Depth: 0.4-0.65m



Hunter Civilab
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 Email: office@huntercivilab.com.au



Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: James Wyatt
 Laboratory Manager
 NATA Accredited Laboratory Number: 14975

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)

Iss (%)	3.0
Visual Description	Silty Clay trace soil grey/brown
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	
Variation to the test method: Readings between some shrink & swell measurements exceed 12 hours.	

Core Shrinkage Test

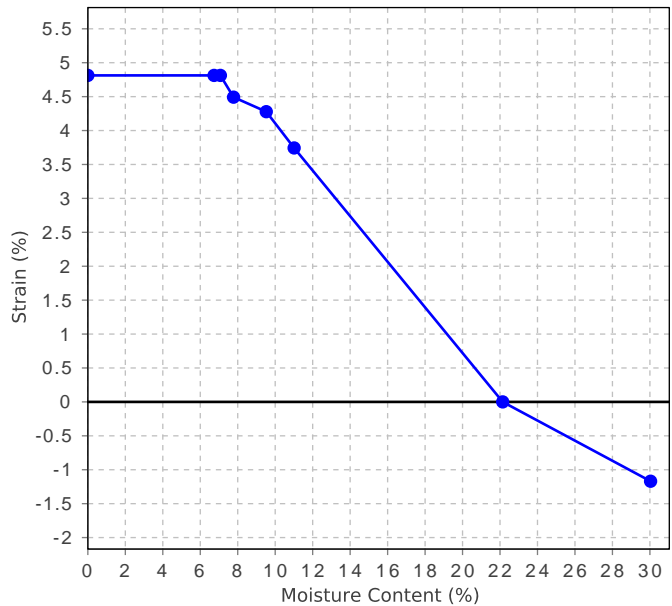
Shrinkage Strain - Oven Dried (%)	4.8
Estimated % by volume of significant inert inclusions	0
Cracking	Moderately Cracked
Crumbling	No
Moisture Content (%)	22.1

Swell Test

Initial Pocket Penetrometer (kPa)	225
Final Pocket Penetrometer (kPa)	175
Initial Moisture Content (%)	23.3
Final Moisture Content (%)	30.0
Swell (%)	1.2

* NATA Accreditation does not cover the performance of pocket penetrometer readings.

Shrink Swell



Material Test Report

Report Number: P22767-216B
Issue Number: 1
Date Issued: 21/08/2023
Client: Hunter Civilab
3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P22767
Project Name: Geotechnical Consulting Services
Project Location: Eastern Precinct Community Centres Project, Chisholm
Client Reference: G0244
Work Request: 12113
Sample Number: 23-12113B
Date Sampled: 27/07/2023
Dates Tested: 28/07/2023 - 09/08/2023
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Preparation Method: AS 1289.1.1 - Sampling and preparation of soils
Site Selection: Selected by Client
Sample Location: BH3, Depth: 0.9-1.2m



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Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: James Wyatt
Laboratory Manager
NATA Accredited Laboratory Number: 14975

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	33		
Plastic Limit (%)	18		
Plasticity Index (%)	15		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	7.5		
Cracking Crumbling Curling	Cracking		

Material Test Report

Report Number: P22767-216B
Issue Number: 1
Date Issued: 21/08/2023
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P22767
Project Name: Geotechnical Consulting Services
Project Location: Eastern Precinct Community Centres Project, Chisholm
Client Reference: G0244
Work Request: 12113
Sample Number: 23-12113C
Date Sampled: 27/07/2023
Dates Tested: 28/07/2023 - 18/08/2023
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Preparation Method: AS 1289.1.1 - Sampling and preparation of soils
Site Selection: Selected by Client
Sample Location: BH4, Depth: 0.2 - 1.0m



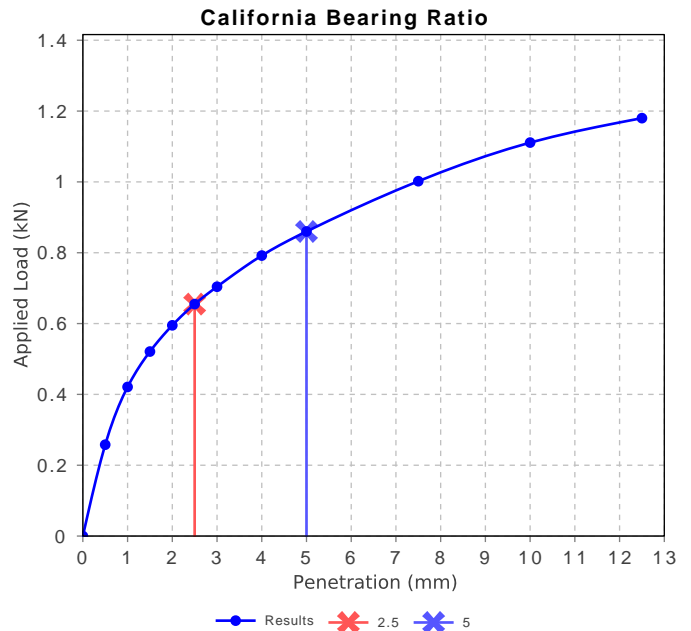
Hunter Civilab
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 Email: office@huntercivilab.com.au



Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: James Wyatt
 Laboratory Manager
 NATA Accredited Laboratory Number: 14975

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	5.0		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	vt		
Maximum Dry Density (t/m ³)	1.76		
Optimum Moisture Content (%)	18.0		
Laboratory Density Ratio (%)	99.5		
Laboratory Moisture Ratio (%)	103.5		
Dry Density after Soaking (t/m ³)	1.74		
Field Moisture Content (%)	18.3		
Moisture Content at Placement (%)	18.7		
Moisture Content Top 30mm (%)	19.7		
Moisture Content Rest of Sample (%)	19.0		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	238.2		
Swell (%)	0.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0.0		
Variation from Test Method	Moulded outside Moisture tolerance		



Material Test Report



Report Number: P22767-216B
Issue Number: 1
Date Issued: 21/08/2023
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P22767
Project Name: Geotechnical Consulting Services
Project Location: Eastern Precinct Community Centres Project, Chisholm
Client Reference: G0244
Work Request: 12113
Sample Number: 23-12113D
Date Sampled: 27/07/2023
Dates Tested: 28/07/2023 - 18/08/2023
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Preparation Method: AS 1289.1.1 - Sampling and preparation of soils
Site Selection: Selected by Client
Sample Location: BH5, Depth: 0.2 - 0.9m

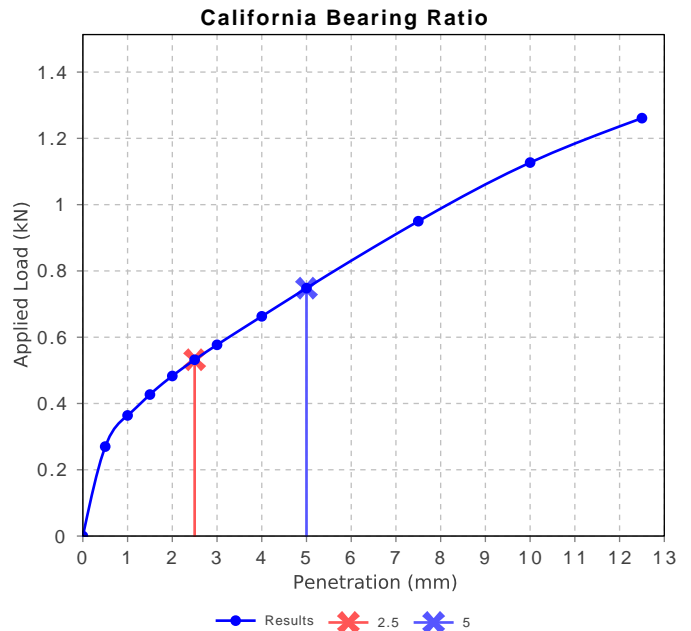
Hunter Civilab
 62 Sandringham Avenue Thornton NSW 2322
 Phone: (02) 4966 1844
 Email: office@huntercivilab.com.au



Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: James Wyatt
 Laboratory Manager
 NATA Accredited Laboratory Number: 14975

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	4.0		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	vt		
Maximum Dry Density (t/m ³)	1.68		
Optimum Moisture Content (%)	19.5		
Laboratory Density Ratio (%)	100.5		
Laboratory Moisture Ratio (%)	101.5		
Dry Density after Soaking (t/m ³)	1.66		
Field Moisture Content (%)	21.0		
Moisture Content at Placement (%)	19.8		
Moisture Content Top 30mm (%)	24.3		
Moisture Content Rest of Sample (%)	20.1		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	240.9		
Swell (%)	1.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0.0		





Annex D

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; landslide; 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 perpend).

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

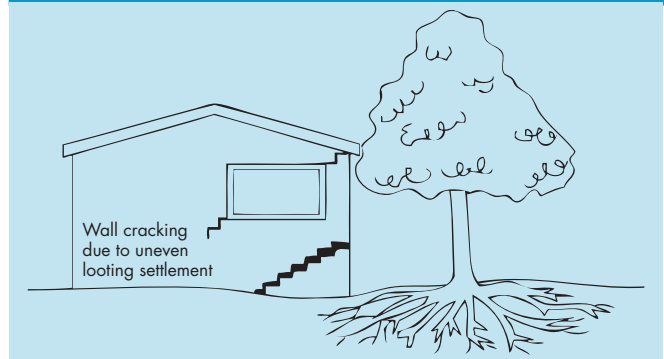
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the 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

Trees can cause shrinkage and damage



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.

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

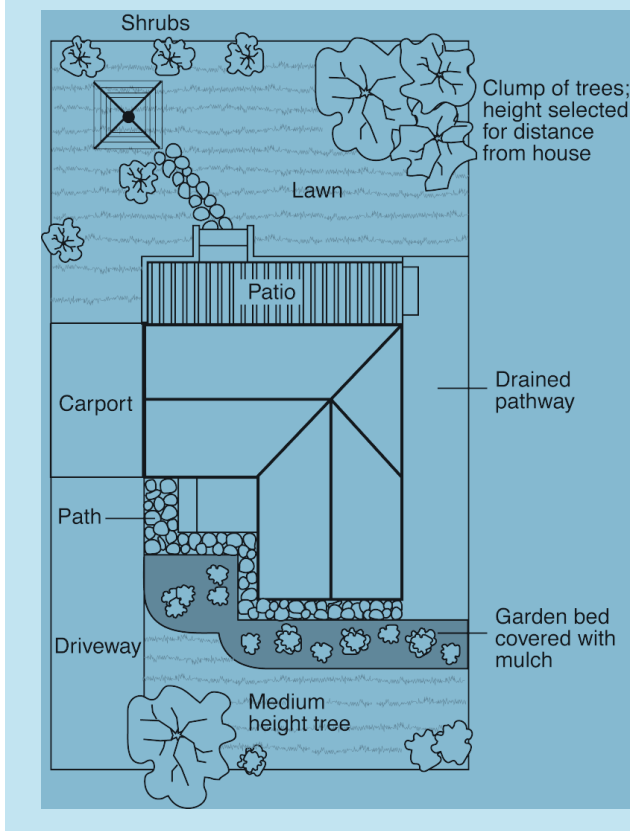
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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.

The information in this and other issues in the series was derived from various sources and was believed to be correct when published.

The information is advisory. It is provided in good faith and not claimed to be an exhaustive treatment of the relevant subject.

Further professional advice needs to be obtained before taking any action based on the information provided.

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