



Geotechnical Investigation

Lot 19 No.91 Gardiner Street

Rutherford

Report Ref: C1105-R-001-Rev0

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We confirm that the following report has been produced for Diana Pet Food, based on the described methods and conditions within.

For and on behalf of Hunter Civilab,



Nathan Roberts

Geotechnical Engineering Manager

Executive Summary

The following report details the geotechnical investigation undertaken by Hunter Civilab (HC) under the request of Diana Pet Food. The investigation was undertaken at Lot 19 No.91 Gardiner Street Rutherford on the 3/09/2021 and 13/09/2021 and consisted of a desktop study, a visual site assessment and intrusive excavations and testing.

The desktop study indicated that the site does not lie in an area of known acid sulfate soils.

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

The site is located on undeveloped rural bushland west of Rutherford and Telarah. The site is surrounded mostly by undeveloped land / farmlands, with a railway line bordering the site at the south, running east to west (city to country).

The subsurface profile generally consisted of alluvial Silty CLAY overlying residual CLAY's

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 H2, having a characteristic free surface movement of 60 – 75 mm. Therefore, a site classification of Class P - H2 is recommended for the site.

The site would be suitable for the use of both shallow and deep footings. Refer to **Section 8** 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 in-situ testing subgrade CBR of 3% which was adopted for the purpose of the pavement design. For flexible pavement design a traffic loading of 1.0×10^5 DESA's was adopted for a 30-year design life based on Maitland City Council engineering guidelines. For rigid pavement design a traffic loading of 1.0×10^5 DESA's was adopted for a 30-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 designs can be seen below in **Table 1**.

Table 1 - Summary of flexible and rigid pavement designs

Flexible pavement option	Rigid pavement option
Two Coat Seal OR 30 mm of Asphalt wearing course	175 mm of Concrete basecourse
150 mm of granular basecourse	150 mm of granular sub-base
200 mm of granular sub-base	

Refer to **Section 10** for the detailed pavement design including material and compaction requirements.

An infiltration test was undertaken and indicated a steady state saturation infiltration rate of 5mm/hr would be suitable for the site.

Laboratory test results for soil aggressivity for each location are summarised in the **Table 1.1** below.

Table 1.1 - Aggressivity Summary

Borehole	Depth (m)	Concrete	Steel
BH1	0.9 – 1.0	Mild	Moderate
BH5	0.6 – 0.7	Mild	Moderate
BH8	2.0 – 2.1	Mild	Moderate

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Annex C – Hunter Civilab Laboratory & Field Test Reports

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

1 Introduction

At the request of Diana Pet Food, Hunter Civilab (HC) have carried out a limited geotechnical investigation for the purpose of a site classification at Lot 19 No.91 Gardiner Street Rutherford. It is understood that the proposed development is to consist of the construction of proposed industrial warehouses / offices. The investigation works were undertaken in accordance with HC services agreement Q0063-Rev1, dated the 24th August 2021.

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

- surface and sub-surface conditions;
- laboratory test results;
- site preparation;
- excavation conditions;
- suitability of site soils for fill;
- Site Classification to AS 2870-2011;
- alternative footing types and foundation design parameters (Allowable Bearing Capacity (kPa) recommendations);
- retaining wall parameters (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);
- suitable options for pavement thickness design;
- general construction recommendations;
- infiltration test results; and
- infiltration rate recommendation – hydraulic conductivity.

2 Site Description

The site was located at Lot 19 No.91 Gardiner Street Rutherford. The site was bordered by railway tracks on the south west, and undeveloped land on all other sides.

At the time of investigation site was undeveloped.

Existing vegetation consisted of mature and growing trees, which had been previous cut down, medium to dense shrubs and grass.

Topographically the site slopes towards the east.

3 Preliminary Site Investigation

3.1 Geological and Soil Landscape Setting

Reference to the 1:250,000 Singleton Geological Map indicates that the site is underlain by the Rutherford formation consisting of mudstone, sandstone, shalestone and limestone.

Reference to the 1:250,000 Singleton Soil Landscape Map indicates that the site is located within the Branxton Landscape. The landscape is characterized by undulating rises to low hills and creek flats. Elevations range from 50 - 80 m with slopes gradients from 3 - 5% on local reliefs of 10 to 40m. Much of

this landscape has drainage lines spaced at 400 – 1,500 m intervals. The soil is known to consist of Yellow Podzolic Soils (Dy5.41) on midslopes with Red Podzolic Soils (Dr3.21) on crests. Yellow Soloths (Dy4.41, Dy3.41) occur on lower slopes and in drainage lines. Alluvial Soils (sands - Uc1.22) occur in some creeks with Siliceous Sands (Uc4.22) on flats within large valleys. Some acid topsoil problems are encountered in the area. The vegetation in the landscape is mainly cleared for grazing purposes, with native pastures. Some areas of uncleared native bushland contain mainly spotted gum, red ironbark and narrow-leaved red ironbark, with swamp oak in the drainage lines.

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 Methodology

Fieldwork was undertaken on the 03/09/2021 and 13/09/2021 and consisted of:

- a visual assessment of the existing surface of the site and surrounding area;
- locating borehole by approximate measurements from existing site features;
- the drilling of 8 x boreholes (BH1 – BH8) to depths of up to 3.0m;
- the driving of 8 x Dynamic Cone Penetrometer (DCP) probes to depths of up to 2.7m;
- undertaking of 2 x Double Ring Infiltrometer tests;
- recovery of 2 x undisturbed soil samples for laboratory testing;
- recovery of 4 x disturbed soil samples for laboratory testing;
- recovery of 3 x bulk soil samples for laboratory testing;

Laboratory testing consisted of:

- 2 x Shrink Swell Index tests;
- 1x Atterberg Limits tests;
- 3x California Bearing Ratio tests; and
- 3x Aggressivity tests.

5 Subsurface Conditions

The subsurface soil conditions encountered at the site have been summarised into the following units:

UNIT 1 – TOPSOIL:

- Sandy SILT, brown

UNIT 2A – ALLUVIAL:

- Silty Sandy CLAY, brown, soft to firm
- Silty CLAY, brown / orange / grey, soft to firm

UNIT 2B – ALLUVIAL:

- Silty Sandy CLAY, brown, stiff to very stiff
- Silty CLAY, brown, firm to stiff

UNIT 2C – RESIDUAL:

- Sandy CLAY, brown, stiff to very stiff
- Silty Sandy CLAY, brown / dark brown / red / grey, very stiff
- Silty CLAY, brown / orange / grey / mottled red, stiff to very stiff

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	Depth (m)	Depth (m)			
		UNIT 1	UNIT 2A	UNIT 2B	UNIT 2C
BH1	3.0	0.0–0.3	-	0.3–2.20	2.20–3.0
BH2	2.0	0.0–0.3	0.3–0.5	0.5–2.0	-
BH3	2.0	0.0–0.1	0.1–0.5	0.5–2.0	-
BH4	2.0	0.0–0.3	0.6–1.2	0.3–0.6 1.2–2.0	-
BH5	3.0	0.0–0.1	-	0.1–1.2	1.2–3.0
BH6	3.0	0.0–0.3	0.3–0.7	0.7–1.5	1.5–3.0
BH7	3.0	0.0–0.2	-	0.2–2.0	2.0–3.0
BH8	3.0	0.0–0.2	0.2–0.5	0.5–1.5	1.5–3.0

Both groundwater and surface water were not encountered at the site.

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

6 Laboratory Test Results

2x undisturbed, 1 x disturbed and 3x Bulk samples were recovered from the boreholes. The samples were transported to Hunter Civilab's NATA accredited soil testing laboratory for analysis.

3x disturbed samples were recovered from the boreholes. The samples were transported for external laboratory testing and analysis.

The laboratory test results are summarised below in **Table 6.1** to **Table 6.3** below.

Table 6.1 - Shrink Swell Index test results

Borehole	Depth (m)	Soil description	I _{ss} (%)
BH5	1.0 – 1.25	Silty CLAY	6.5
BH8	0.50 – 0.75	Silty CLAY	7.6

Table 6.2 - Atterberg Limit test results

Borehole	Depth (m)	Soil description	Plasticity Index (%)	Linear Shrinkage (%)
BH1	0.6 – 0.75	Silty CLAY	19	8.5

Table 6.3 – California Bearing Ratio test results

Borehole	Depth (m)	FMC (%)	Swell (%)	OMC (%)	MDD (t/m ³)	CBR (%)
BH2	0.5 – 0.8	23.9	2.0	24.0	1.58	2.0
BH3	0.5 – 0.8	28.0	6.0	22.0	1.58	0.5
BH4	0.5 – 0.8	26.0	2.0	22.5	1.62	3.0

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

7 Site Classification

7.1 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 (y_s) (mm)	Site classification AS2870-2011	Underlying soil / geology
0	Class A	SAND or ROCK site (non-reactive)
0 – 20mm	Class S	CLAY (slightly reactive)
20 – 40mm	Class M	CLAY (moderately reactive)
40 – 60mm	Class H1	CLAY (highly reactive)
60 – 75mm	Class H2	CLAY (highly reactive)
> 75mm	Class E	CLAY (extremely reactive)

Sites subjected to deep-seated moisture change are modified with the addition of “-D”. As defined by AS2870-2011 other sites should be classified as a Class P (Problem) site. These sites include sites with:

- 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).

7.2 Site Classification

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 5**, 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)
Class P	Class H2	60 – 75 mm

The site was classified as a Class P due to the presence of trees / recently cleared and uprooted trees that may create abnormal moisture conditions.

Based on the subsurface profile and the results of the laboratory testing a site reactivity of Class H2 has been assigned to the Class P site.

Classification of the site has not taken into account the effects of abnormal moisture conditions. If the site undergoes any earthworks operations, the site shall be reclassified in accordance with AS2870-2011.

7.3 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.

7.4 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.

8 Footing Recommendations

The site is suitable for the use of both shallow and deep footing systems dependant on the development and structural bearing pressure required. Refer to **Section 8.1** and **Section 8.2** below for recommended allowable bearing pressure parameters.

8.1 Shallow Footings

A maximum allowable bearing capacity of 100kPa is recommended at the site for shallow level footings founded within stiff clay 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 piers / taken to bedrock to reduce the risk of differential settlement.

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 7.2** above.

8.2 Deep Footings

The site is suitable for bored piers with an approximate allowable end bearing pressures and shaft adhesion estimated below in **Table 8.1**.

Table 8.1 - Summary of allowable end bearing pressures and shaft adhesion for deep footings

Soil strata	Typical depth encountered (m)	Allowable shaft adhesion (kPa)	Allowable end bearing pressure (kPa)
UNIT 2B – Stiff Alluvial / Residual	0.8 – 2.0	5	150
UNIT 2C – Stiff to very stiff Residual	2.0 – 3.0	25	300

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

8.3 Footing Construction

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.

8.4 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 E** for the following:

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

9 Retaining Walls

Recommended site soil parameters for retaining wall design at the site are provided in **Table 9.1** below.

Table 9.1 - Recommended retaining wall design soil parameters

Parameter	Supported material		
	UNIT 2A Soft to Firm	UNIT 2B Stiff	UNIT 2C Stiff to Very Stiff
γ (kN/m ³)	17	20	22
Φ' (°)	10	22	26
C' (kPa)	0	0	5
C_u (kPa)	30	50	150
K_a	0.704	0.455	0.390
K_p	1.420	2.198	2.561
K_o	0.826	0.625	0.562

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

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

For retaining walls surcharge loads from uphill structures should be considered and it is recommended that a minimum surcharge of 5kPa be adopted for this purpose. Retaining walls in excess of 1m high should be designed by a qualified structural engineer, with adequate subsurface and surface drainage provided behind the retaining wall.

10 Pavement Thickness Design

10.1 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.

Adopted design CBR:

Based on laboratory test results as described in **Section 6**, and in-situ subgrade results values adopted were as follows:

- 3.0% for in-situ natural clay subgrades;

Adopted Traffic Loadings:

As per Maitland City Council Engineering Guidelines the following traffic loadings were adopted for design:

Flexible Pavements:

- 1×10^5 Design ESA's.

Rigid Pavements:

- 1×10^5 Design ESA's.

Adopted Design Life:

As per Maitland City Council Engineering Guidelines the following design life is was adopted for design:

Flexible Pavements:

- 30-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

Contractor should also confirm Maitland City Council material specification requirements and seek written approval of proposed materials or mix design prior to supplying the proposed material.

In addition, it is noted that bound layers should preferably be placed in one layer.

10.2 Flexible Pavement Thickness Design

The recommended flexible pavement thickness, pavement material and compaction specification are presented in **Table 10.1** and **Table 10.2** below.

Table 10.1 – Summary of flexible pavement minimum thickness design

Pavement	Depth (mm)
14/7 Two Coat Bituminous Seal (OR)	(21)
AC10 Wearing course (to Council spec)	30
Primer seal	(10)
Base course (DGB20 or equivalent)	150
Subbase (DGS40 or equivalent)	200
Total thickness (mm)	380
In-situ Subgrade CBR	3.0%*

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

*Due to the presence of low CBR material with high swell % across the site, it is recommended that the top 150mm of exposed subgrade be stabilised by tyning with 2% lime by dry weight.

Table 10.2 – Flexible pavement compaction criteria

Pavement	Compaction criteria
14/7 Two Coat Bituminous Seal (OR)	NA
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)
Subgrade (with 2% lime Stabiliser)	100% Standard (AS 1289.5.1.1)

10.3 Rigid Pavement Thickness Design

The recommended rigid pavement thickness, pavement material and compaction specifications are presented in **Table 10.3** and ***Due to the presence of low CBR material with high swell % across the site, it is recommended that the top 150mm of exposed subgrade be stabilised by tyning with 2% lime by dry weight.**

Table 10.4 below.

Table 10.3 – Summary of rigid pavement minimum thickness design

Pavement	Depth (mm)
Base course (32 MPa concrete with SL92 reinforcement)	175
Subbase (crushed rock subbase min soaked CBR 80%, max PI = 6%, or equivalent)	150
Total thickness (mm)	325
In-situ Subgrade CBR	3.0%*

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

*Due to the presence of low CBR material with high swell % across the site, it is recommended that the top 150mm of exposed subgrade be stabilised by tyning with 2% lime by dry weight.

Table 10.4 – Rigid pavement compaction criteria

Pavement	Compaction Criteria
Base course (32 MPa concrete with SL92 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)
Subgrade (with 2% lime Stabiliser)	100% Standard (AS 1289.5.1.1)

10.4 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.

10.5 Recommendations During Construction

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

11 Earthworks

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

11.1 Site Preparation

New Pavement Construction:

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

- 1) 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);
- 2) exposed suitable subgrade areas should then be ripped 300mm and re-compacted to 100% standard maximum dry density (SMDD) at $\pm 2\%$ of optimum moisture content (OMC);
- 3) 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.

11.2 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 5**, 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 5**, HC should be consulted to determine if the material is suitable for controlled fill. Similarly, any won material imported from external sites should consult HC to determine if the fill is suitable for controlled fill.

11.2.1 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 road developments the following compaction criteria applies:

- Subgrade Soils – 100% Standard Maximum Dry Density to $\pm 2\%$ of OMC;
- General fill – 98% Standard Maximum Dry Density;
- Non-cohesive soils:
 - Base 98% modified maximum dry density (MMDD) at -2% to 0% of optimum moisture content (OMC);
 - Sub-base 95% modified maximum dry density (MMDD) at -2% to 0% of optimum moisture content (OMC).

Reference should also be made to council development guidelines for compaction criteria for different traffic loading.

A suitably qualified geotechnical professional must be consulted to determine that the specified compaction has been achieved.

12 Double Ring Permeability Test

12.1 Test Method

The soil permeability of the clays was tested in accordance with ASTM D3385-09 "Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer".

The test procedure involved:

1. Driving a 400mm diameter outer ring into ground surface;
2. Driving a 300mm diameter inner ring in the middle of the 400mm ring;
3. Fill both rings with water to near the top;
4. Measure the depth of the water within the inner ring at regular time intervals;
5. Maintain a constant water level in the outer ring;
6. Continuing the testing until steady state has been achieved, refilling the inner ring as required when water has been exhausted.

The drop in water level in the reservoir was recorded until the change in drop became steady. Steady state was achieved when the drop was $\pm 10\%$ of the previous drops.

12.2 Infiltration Testing Results

The Double Ring Infiltrometer described with ASTM D3385-09 "Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer" was used to determine the saturated conductivity.

Using the test method described in **Section 12.1** above, the "steady state" soil permeability was calculated for test location 1 & 2. The results can be seen in **Table 12.1** below.

Table 12.1 – Summary of infiltration testing results

Location	Average Steady State Rate		
	mm/min	mm/hr	m/day
Test 1	0.05	3.05	0.07
Test 2	0.08	4.70	0.11

The soil permeability testing results and calculations can be seen in **Annex C**.

12.3 Discussion

Based on the results of the laboratory testing it is recommended that a steady state saturated infiltration rate of 5mm/hr would be suitable for the site.

12.4 Aggressivity of Soils to Buried Structures

12.4.1 Laboratory Test Results

3 x disturbed representative samples were recovered from the boreholes. The samples were transported to Sydney Analytical Laboratory's NATA accredited soil testing laboratory for analysis. The laboratory test results are summarised below in **Table 12.2**

Table 12.2 - Summary of soil aggressivity results

Sample & Depth (m)	Chloride (ppm)	Sulphate (ppm)	pH	Resistivity (ohm.cm)	Concrete Exposure Classification	Steel Exposure Classification
BH1 (0.9 – 1.0)	1200	360	5.1	0.0014	Mild	Moderate
BH5 (0.6 – 0.7)	1900	700	4.6	0.0007	Mild	Moderate
BH8 (2.0 – 2.1)	1200	410	4.9	0.0012	Mild	Moderate

The external laboratory test reports can be found in **Annex D**.

12.4.2 Discussion of Aggressivity Results

Based on the results from the chemical laboratory testing of the soil sample collected on the site, and with reference to Table 6.4.2 (A) & (C) and 6.5.2 (A) & (C) in AS2159-2009, the exposure classification for on structural elements falls into **Mild** towards concrete and **Moderate** towards steel.

Concrete piles should be designed in accordance with AS 2159-2009 Table 6.4.3. A minimum concrete strength of 50MPa for precast and prestressed piles and 32MPa for cast in situ concrete piles must be used. The minimum required cover to reinforcement is 20mm for precast piles and 60mm for cast in situ piles over a 50-year design life (or 30mm for precast piles and 75mm for cast in situ piles over a 100-year design life).

Exposed steel should be designed in accordance with AS 2159-2009 Table 6.5.3 a minimum steel uniform corrosive allowance of 0.02mm to 0.04mm/year should be allowed for.

12.5 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 tyres 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 March 2015.

<https://www.safeworkaustralia.gov.au/system/files/documents/1705/mcop-excavation-work-v3.pdf>

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.

12.6 Batter Slopes

12.6.1 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 March 2015.

12.6.2 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).

13 Report Limitations

This report has been prepared by Hunter Civilab (HC) for the specific site and purposes described within this report. HC will accept no responsibility or liability for the use of this report by any third party, without the express consent of HC 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 HC.

The scope of the investigation described in this report is based on information and plans provided to HC 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. HC 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 HC. 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. HC should be contacted for further consultation and site re-assessment should sub-surface conditions differ from those conditions identified in this report.

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:



Jope Turner

Undergraduate Geotechnical Engineer

Reviewed by:



Nathan Roberts

Geotechnical Engineering Manager
Bachelor of Engineering (Civil)

References:

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Annex A



*NOTE: Overhead image shown was taken from Nearmaps

Figure 1 – Plan of the development at Lot 19 No.91 Gardiner Street, Rutherford showing the approximate location of the Geotechnical boreholes.



Annex B



BOREHOLE LOG REPORT

HOLE NO: BH2
FILE / JOB NO: C1105
SHEET: 1 OF 1

CLIENT: Diana Pet Food
PROJECT: Proposed Industrial Development
LOCATION: Lot 109 No.91 Gardiner Street, Rutherford

POSITION:	SURFACE ELEVATION:	INCLINATION: 90°
DRILLING METHOD: Drill Rig	CONTRACTOR: HCL	DRILLER: SH
DATE LOGGED: 03/09/2021	DATE SAMPLED: 03/09/2021	LOGGED BY: UM / JT
		CHECKED BY: NR

TESTING & SAMPLING				MATERIAL					
Water	DCP AS 1289.6.3.2-1997		Field Tests	Samples	Depth (m)	Graphic Log Classification Symbol	MATERIAL DESCRIPTION Soil Type, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	Moisture Condition Consistency/ Relative Density	STRUCTURE & Other Observations
	Depth (m)	Blows							
	0.2 - 0.3	2			0.2	ML	TOPSOIL: Sandy SILT, low plasticity, brown, fine grained sand	-PL to >PL	TOPSOIL
	0.3 - 0.4	2			0.3		Silty Sandy CLAY, low to medium plasticity, brown, fine grained sand	<PL	ALLUVIUM
	0.4 - 0.5	1			0.5	CL-CI	Silty CLAY, high plasticity, brown	S to F	
	0.5 - 0.6	2		B 0.50-0.80	0.5				
	0.6 - 0.7	2							
	0.7 - 0.8	2			1.0				F to St
	0.8 - 0.9	2							
	0.9 - 1.0	3							
	1.0 - 1.1	3							
	1.1 - 1.2	3							
	1.2 - 1.3	3				CH		>PL to -PL	
	1.3 - 1.4	5							
	1.4 - 1.5	5			1.5				
	1.5 - 1.6	4							
	1.6 - 1.7	9							St to VSt
	1.7 - 1.8	Terminated							
					2.0		Terminated at 2.00 m		
					2.5				
					3.0				

Additional Comments	CLASSIFICATION SYMBOLS & SOIL DESCRIPTION Based on Unified Classification System <div style="border: 1px solid black; padding: 5px; width: fit-content;"> WATER Water table Water inflow </div>	SAMPLES & FIELD TESTS U - Undisturbed Sample D - Disturbed Sample ES - Environmental Sample B - Bulk Disturbed Sample MC - Moisture Content PP - Pocket Penetrometer SPT - Standard Penetration Test VS - Vane Shear	MOISTURE D - Dry M - Moist W - Wet <PL - Moist, below PL ~PL - Moist, approx. PL >PL - Moist, above PL ~LL - Wet, approx. LL >LL - Wet, above LL PL - Plastic Limit LL - Liquid Limit	CONSISTENCY/ RELATIVE DENSITY VS - Very Soft S - Soft F - Firm St - Stiff VSt - Very Stiff H - Hard VL - Very Loose L - Loose MD - Medium Dense D - Dense VD - Very Dense
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BOREHOLE LOG REPORT

HOLE NO: BH3
 FILE / JOB NO: C1105
 SHEET: 1 OF 1

CLIENT: Diana Pet Food
 PROJECT: Proposed Industrial Development
 LOCATION: Lot 109 No.91 Gardiner Street, Rutherford

POSITION:	SURFACE ELEVATION:	INCLINATION: 90°
DRILLING METHOD: Drill Rig	CONTRACTOR: HCL	DRILLER: SH
DATE LOGGED: 03/09/2021	DATE SAMPLED: 03/09/2021	LOGGED BY: UM / JT
		CHECKED BY: NR

TESTING & SAMPLING				MATERIAL						
Water	DCP AS 1289.6.3.2-1997		Field Tests	Samples	Depth (m)	Graphic Log	Classification Symbol	MATERIAL DESCRIPTION Soil Type, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	Moisture Condition Consistency/ Relative Density	STRUCTURE & Other Observations
	Depth (m)	Blows								
							ML	0.10m	TOPSOIL: Sandy SILT, low plasticity, brown, fine grained sand	TOPSOIL
							CH		Silty CLAY, high plasticity, brown, trace fine grained sand	ALLUVIUM
	0.2 - 0.3	1					CH			S to F
	0.3 - 0.4	1					CH			
	0.4 - 0.5	2					CH			
	0.5 - 0.6	2			0.5		CH	0.50m	Silty CLAY, high plasticity, brown	St
	0.6 - 0.7	3		B 0.50-0.80			CH			
	0.7 - 0.8	3					CH			
	0.8 - 0.9	3					CH			
	0.9 - 1.0	5					CH			
	1.0 - 1.1	5			1.0		CH			~PL or >PL
	1.1 - 1.2	5					CH			
	1.2 - 1.3	6					CH			
	1.3 - 1.4	6					CH			
	1.4 - 1.5	8					CH			St to VSt
	1.5 - 1.6	Terminated					CH			
					2.0		CH	2.00m	Terminated at 2.00 m	
							CH			
					2.5		CH			
							CH			
					3.0		CH			

Additional Comments	CLASSIFICATION SYMBOLS & SOIL DESCRIPTION Based on Unified Classification System Water table Water inflow	SAMPLES & FIELD TESTS U - Undisturbed Sample D - Disturbed Sample ES - Environmental Sample B - Bulk Disturbed Sample MC - Moisture Content PP - Pocket Penetrometer SPT - Standard Penetration Test VS - Vane Shear	MOISTURE D - Dry M - Moist W - Wet <PL - Moist, below PL ~PL - Moist, approx. PL >PL - Moist, above PL ~LL - Wet, approx. LL >LL - Wet, above LL PL - Plastic Limit LL - Liquid Limit	CONSISTENCY/RELATIVE DENSITY VS - Very Soft S - Soft F - Firm St - Stiff VSt - Very Stiff H - Hard VL - Very Loose L - Loose MD - Medium Dense D - Dense VD - Very Dense
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BOREHOLE LOG REPORT

HOLE NO: BH4
FILE / JOB NO: C1105
SHEET: 1 OF 1

CLIENT: Diana Pet Food
PROJECT: Proposed Industrial Development
LOCATION: Lot 109 No.91 Gardiner Street, Rutherford

POSITION:	SURFACE ELEVATION:	INCLINATION: 90°
DRILLING METHOD: Drill Rig	CONTRACTOR: HCL	DRILLER: SH
DATE LOGGED: 03/09/2021	DATE SAMPLED: 03/09/2021	LOGGED BY: UM / JT
		CHECKED BY: NR

TESTING & SAMPLING				MATERIAL						
Water	DCP AS 1289.6.3.2-1997		Field Tests	Samples	Depth (m)	Graphic Log	Classification Symbol	MATERIAL DESCRIPTION Soil Type, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	Moisture Condition Consistency/Relative Density	STRUCTURE & Other Observations
	Depth (m)	Blows								
	0.2 - 0.3	3			0.30m		ML	TOPSOIL: Sandy SILT, low plasticity, brown, fine grained sand	-PL to >PL	TOPSOIL
	0.3 - 0.4	3						Silty CLAY, high plasticity, dark brown / orange / grey	>PL	ALLUVIUM
	0.4 - 0.5	3					CH			
	0.5 - 0.6	2		D 0.50-0.80	0.5			As above but decreasing moisture		
	0.6 - 0.7	1								
	0.7 - 0.8	1								
	0.8 - 0.9	1								S to F
	0.9 - 1.0	2								
	1.0 - 1.1	2			1.0					
	1.1 - 1.2	4								
	1.2 - 1.3	4								
	1.3 - 1.4	4							<PL to ~PL	
	1.4 - 1.5	4								
	1.5 - 1.6	3			1.5					St to VSt
	1.6 - 1.7	3								
	1.7 - 1.8	4								
	1.8 - 1.9	5								
	1.9 - 2.0	4			2.0					
	2.0 - 2.1	5						Terminated at 2.00 m		
	2.1 - 2.2	7								
	2.2 - 2.3	6								
	2.3 - 2.4	10								
	2.4 - 2.5	9								
	2.5 - 2.6	9			2.5					
	2.6 - 2.7	11								
	2.7 - 2.8	Terminated								
					3.0					

Additional Comments	CLASSIFICATION SYMBOLS & SOIL DESCRIPTION Based on Unified Classification System WATER Water table Water inflow	SAMPLES & FIELD TESTS U - Undisturbed Sample D - Disturbed Sample ES - Environmental Sample B - Bulk Disturbed Sample MC - Moisture Content PP - Pocket Penetrometer SPT - Standard Penetration Test VS - Vane Shear	MOISTURE D - Dry M - Moist W - Wet <PL - Moist, below PL ~PL - Moist, approx. PL >PL - Moist, above PL ~LL - Wet, approx. LL >LL - Wet, above LL PL - Plastic Limit LL - Liquid Limit	CONSISTENCY/RELATIVE DENSITY VS - Very Soft S - Soft F - Firm St - Stiff VSt - Very Stiff H - Hard VL - Very Loose L - Loose MD - Medium Dense D - Dense VD - Very Dense
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BOREHOLE LOG REPORT

HOLE NO: BH7
FILE / JOB NO: C1105
SHEET: 1 OF 1

CLIENT: Diana Pet Food
PROJECT: Proposed Industrial Development
LOCATION: Lot 109 No.91 Gardiner Street, Rutherford

POSITION:	SURFACE ELEVATION:	INCLINATION: 90°
DRILLING METHOD: Drill Rig	CONTRACTOR: HCL	DRILLER: SH
DATE LOGGED: 03/09/2021	DATE SAMPLED: 03/09/2021	LOGGED BY: UM / JT
		CHECKED BY: NR

TESTING & SAMPLING				MATERIAL					
Water	DCP AS 1289.6.3.2-1997		Field Tests	Samples	Depth (m)	Graphic Log Classification Symbol	MATERIAL DESCRIPTION Soil Type, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	Moisture Condition Consistency/ Relative Density	STRUCTURE & Other Observations
	Depth (m)	Blows							
	0.0 - 0.1	5				ML	TOPSOIL: Sandy SILT, low plasticity, brown, fine grained sand	~PL to >PL	TOPSOIL
	0.1 - 0.2	5							
	0.2 - 0.3	3							
	0.3 - 0.4	5							
	0.4 - 0.5	4							
	0.5 - 0.6	4			0.5	CL	Silty CLAY, low plasticity, dark brown	<PL to ~PL	St
	0.6 - 0.7	4							
	0.7 - 0.8	4							
	0.8 - 0.9	5							
	0.9 - 1.0	6							
	1.0 - 1.1	6			1.0		Silty Sandy CLAY, low to medium plasticity, orange / red / grey / brown, fine grained sand		
	1.1 - 1.2	6							
	1.2 - 1.3	7							
	1.3 - 1.4	Terminated							
					1.5	CI			
					2.0		Sandy CLAY, low plasticity, orange / mottled grey / brown, fine grained sand	<PL	VSt
									RESIDUAL SOIL
					2.5	CL			
					3.0		Terminated at 3.00 m		

Additional Comments	CLASSIFICATION SYMBOLS & SOIL DESCRIPTION Based on Unified Classification System WATER Water table Water inflow	SAMPLES & FIELD TESTS U - Undisturbed Sample D - Disturbed Sample ES - Environmental Sample B - Bulk Disturbed Sample MC - Moisture Content PP - Pocket Penetrometer SPT - Standard Penetration Test VS - Vane Shear	MOISTURE D - Dry M - Moist W - Wet <PL - Moist, below PL ~PL - Moist, approx. PL >PL - Moist, above PL ~LL - Wet, approx. LL >LL - Wet, above LL PL - Plastic Limit LL - Liquid Limit	CONSISTENCY/RELATIVE DENSITY VS - Very Soft S - Soft F - Firm St - Stiff VSt - Very Stiff H - Hard VL - Very Loose L - Loose MD - Medium Dense D - Dense VD - Very Dense
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Annex C

Material Test Report

Report Number: P21557-22A
Issue Number: 1
Date Issued: 13/09/2021
Client: Hunter Civilab
3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200D
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 13/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: BH1, Depth: 0.6-0.75m



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



Accredited for compliance with ISO/IEC 17025 - Testing

A handwritten signature in black ink, appearing to read "Grant Burgess".

Approved Signatory: Grant Burgess
Geologist

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 (%)	34		
Plastic Limit (%)	15		
Plasticity Index (%)	19		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	8.5		
Cracking Crumbling Curling	Cracking		

Material Test Report

Report Number: P21557-22B
Issue Number: 1
Date Issued: 16/09/2021
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200E
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 16/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Remarks: Variation to the test method: Readings between some shrink & swell measurements exceed 12 hours.
Sample Location: BH5, Depth: 1.0-1.25m
Material Source: U50



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



Accredited for compliance with ISO/IEC 17025 - Testing

Grant Burgess

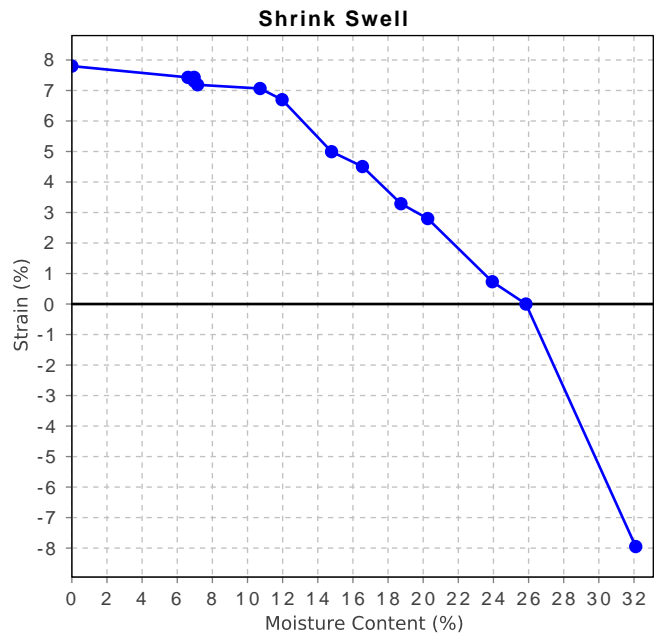
Approved Signatory: Grant Burgess
 Geologist

NATA Accredited Laboratory Number: 14975

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	6.5
Visual Description	CLAY trace silt, brown
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	7.8
Estimated % by volume of significant inert inclusions	2
Cracking	Slightly Cracked
Crumbling	No
Moisture Content (%)	25.9

Swell Test	
Initial Pocket Penetrometer (kPa)	210
Final Pocket Penetrometer (kPa)	50
Initial Moisture Content (%)	28.0
Final Moisture Content (%)	32.1
Swell (%)	8.0
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	



Material Test Report

Report Number: P21557-22B
Issue Number: 1
Date Issued: 16/09/2021
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200F
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 16/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Remarks: Variation to the test method: Readings between some shrink & swell measurements exceed 12 hours.
Sample Location: BH8, Depth: 0.5-0.75m
Material Source: U50



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



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Grant Burgess

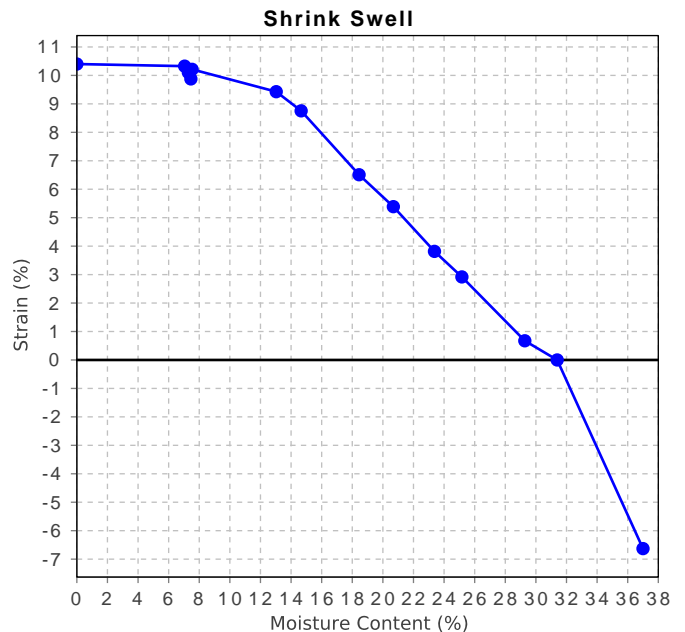
Approved Signatory: Grant Burgess
 Geologist

NATA Accredited Laboratory Number: 14975

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	7.6
Visual Description	CLAY trace silt, brown
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	10.4
Estimated % by volume of significant inert inclusions	0
Cracking	Slightly Cracked
Crumbling	No
Moisture Content (%)	31.4

Swell Test	
Initial Pocket Penetrometer (kPa)	110
Final Pocket Penetrometer (kPa)	50
Initial Moisture Content (%)	33.2
Final Moisture Content (%)	37.0
Swell (%)	6.6
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	



Material Test Report

Report Number: P21557-22C
Issue Number: 1
Date Issued: 05/10/2021
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200A
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 23/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: BH2, Depth: 0.5-0.8m



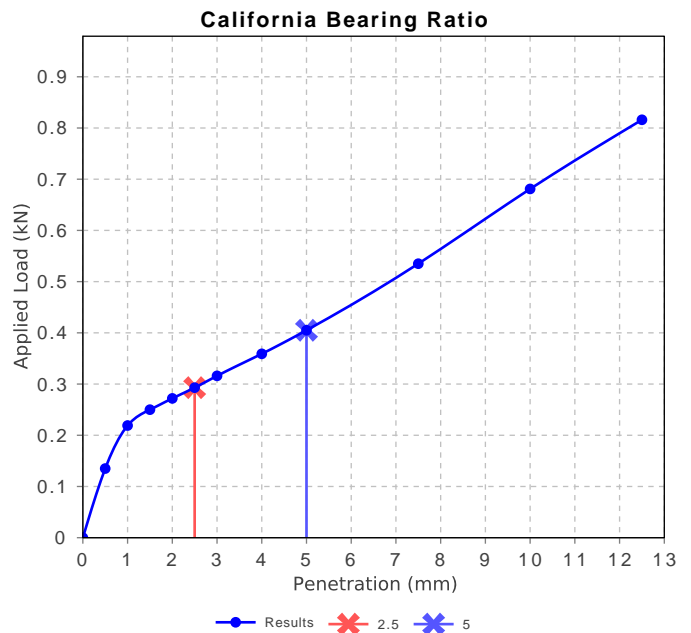
Hunter Civilab
 62 Sandringham Avenue Thornton NSW 2322
 Phone: (02) 4966 1844
 Email: office@huntercivilab.com.au



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 %	2.0		
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)	1.58		
Optimum Moisture Content (%)	24.0		
Laboratory Density Ratio (%)	100.0		
Laboratory Moisture Ratio (%)	96.0		
Dry Density after Soaking (t/m ³)	1.55		
Field Moisture Content (%)	23.9		
Moisture Content at Placement (%)	23.3		
Moisture Content Top 30mm (%)	28.4		
Moisture Content Rest of Sample (%)	24.0		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	189.8		
Swell (%)	2.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Material Test Report

Report Number: P21557-22C
Issue Number: 1
Date Issued: 05/10/2021
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200B
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 23/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: BH3, Depth: 0.5-0.8m



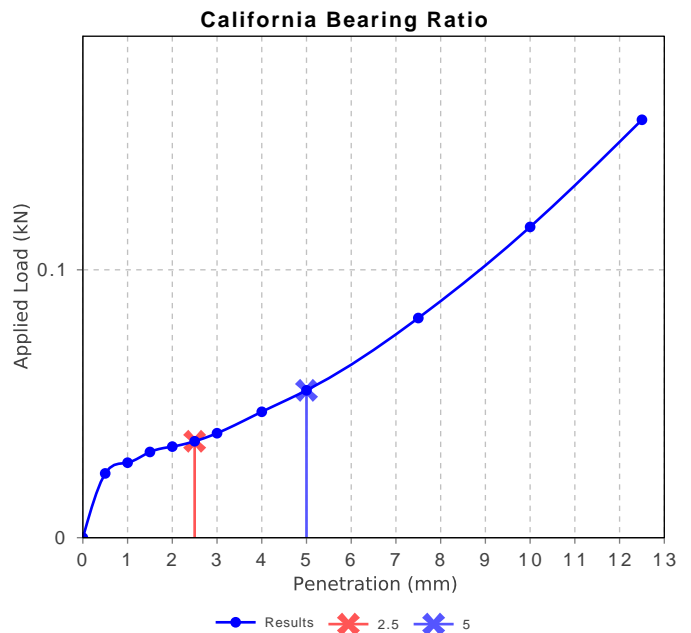
Hunter Civilab
 62 Sandringham Avenue Thornton NSW 2322
 Phone: (02) 4966 1844
 Email: office@huntercivilab.com.au



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	5 mm		
CBR %	0.5		
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)	1.58		
Optimum Moisture Content (%)	22.0		
Laboratory Density Ratio (%)	97.0		
Laboratory Moisture Ratio (%)	101.0		
Dry Density after Soaking (t/m ³)	1.45		
Field Moisture Content (%)	28.0		
Moisture Content at Placement (%)	22.1		
Moisture Content Top 30mm (%)	40.5		
Moisture Content Rest of Sample (%)	26.5		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	190.2		
Swell (%)	6.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		
Variation from Test Method	moulded outside method tolerance.		



Material Test Report

Report Number: P21557-22C
Issue Number: 1
Date Issued: 05/10/2021
Client: Hunter Civilab
 3/62 Sandringham Avenue, Thornton New South Wales 2322
Contact: Nathan Roberts
Project Number: P21557
Project Name: Geotechnical Consulting Services
Project Location: Lot 109 No. 91 Gardiner Street, Rutherford
Client Reference: C1105
Work Request: 5200
Sample Number: 21-5200C
Date Sampled: 03/09/2021
Dates Tested: 07/09/2021 - 30/09/2021
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: BH4, Depth: 0.5-0.8m



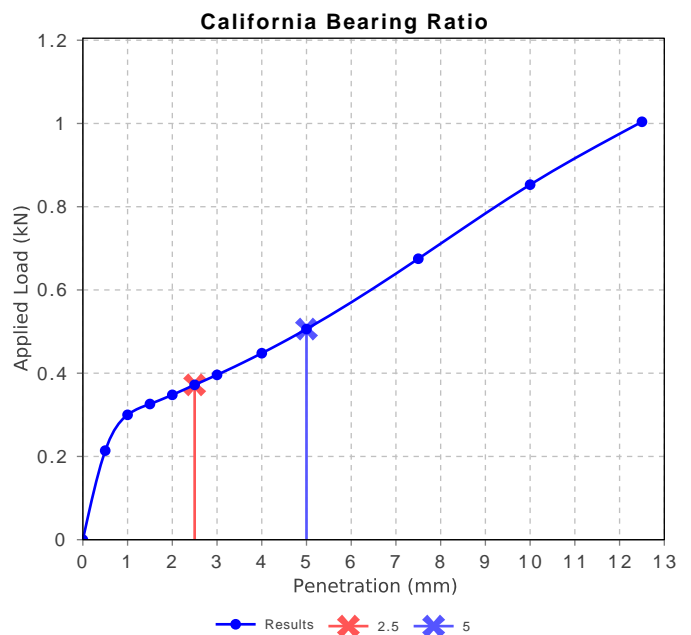
Hunter Civilab
 62 Sandringham Avenue Thornton NSW 2322
 Phone: (02) 4966 1844
 Email: office@huntercivilab.com.au



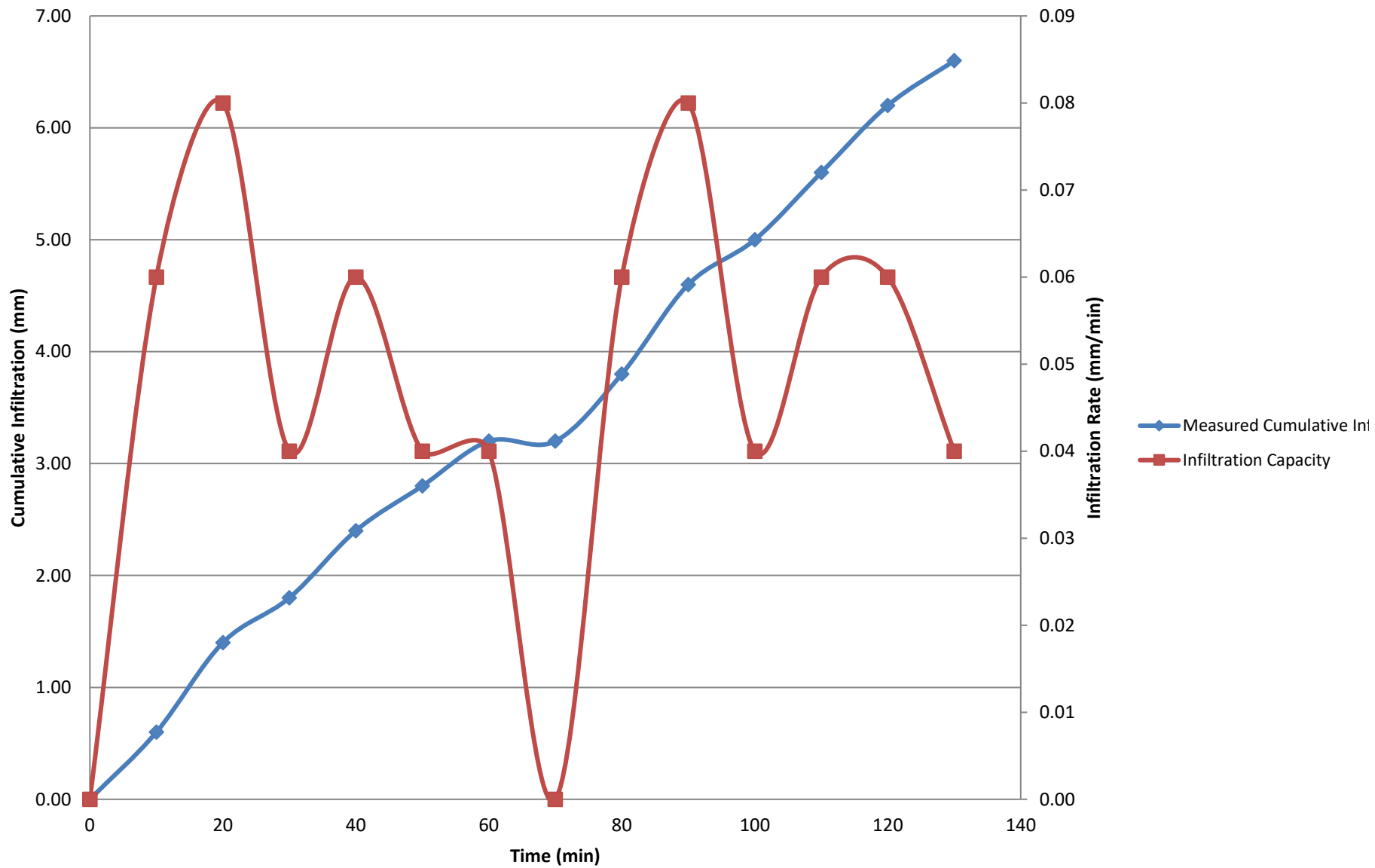
Accredited for compliance with ISO/IEC 17025 - Testing

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 %	3.0		
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)	1.62		
Optimum Moisture Content (%)	22.5		
Laboratory Density Ratio (%)	99.5		
Laboratory Moisture Ratio (%)	102.0		
Dry Density after Soaking (t/m ³)	1.57		
Field Moisture Content (%)	26.0		
Moisture Content at Placement (%)	22.8		
Moisture Content Top 30mm (%)	26.4		
Moisture Content Rest of Sample (%)	22.9		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	67.0		
Swell (%)	2.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Cumulative Infiltration and Infiltration Rate



DOUBLE RING INFILTRATION TEST

Client: Diana Pet Food

Location: Lot 109 No.91 Gardiner Street, Rutherford

Date: 13/09/2021

Job No: C1105

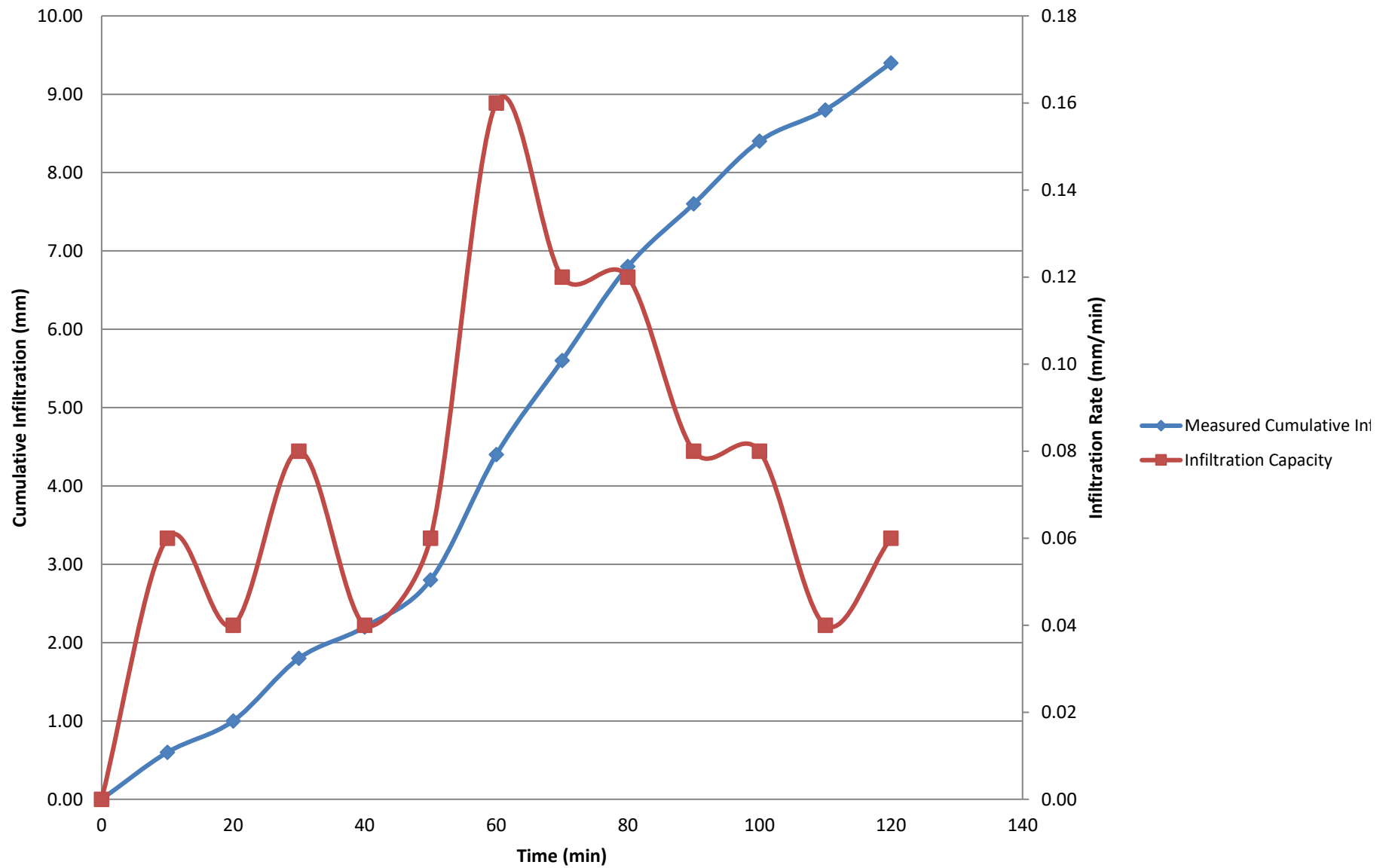
Test Location: Test 1

Logged By: JT/KS

Double Ring Permeability test was undertaken as per ASTM D3385-09 "Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer"

Time (min)	Water Level (mm)				Time Interval (min)	Infiltration (mm)	Cumulative Infiltration (mm)	Infiltration Rate (mm/min)	Infiltration Rate (mm/hour)	Infiltration Rate (m/day)
	Initial Reading	1st Refill Reading	2nd Refill Reading	3rd Refill Reading						
0	38.20	-	-	-	-	-	0.00	-	-	-
10	38.80	-	-	-	10.00	0.60	0.60	0.06	3.60	0.09
20	39.60	-	-	-	10.00	0.80	1.40	0.08	4.80	0.12
30	40.00	-	-	-	10.00	0.40	1.80	0.04	2.40	0.06
40	40.60	-	-	-	10.00	0.60	2.40	0.06	3.60	0.09
50	41.00	-	-	-	10.00	0.40	2.80	0.04	2.40	0.06
60	41.40	38.20	-	-	10.00	0.40	3.20	0.04	2.40	0.06
70	-	38.20	-	-	10.00	0.00	3.20	0.00	0.00	0.00
80	-	38.80	-	-	10.00	0.60	3.80	0.06	3.60	0.09
90	-	39.60	-	-	10.00	0.80	4.60	0.08	4.80	0.12
100	-	40.00	-	-	10.00	0.40	5.00	0.04	2.40	0.06
110	-	40.60	-	-	10.00	0.60	5.60	0.06	3.60	0.09
120	-	41.20	-	-	10.00	0.60	6.20	0.06	3.60	0.09
130	-	41.60	-	-	10.00	0.40	6.60	0.04	2.40	0.06

Cumulative Infiltration and Infiltration Rate



DOUBLE RING INFILTRATION TEST

Client: Diana Pet Food

Location: Lot 109 No.91 Gardiner Street, Rutherford

Date: 13/09/2021

Job No: C1105

Test Location: Test 2

Logged By: JT/KS

Double Ring Permeability test was undertaken as per ASTM D3385-09 "Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer"

Time (min)	Water Level (mm)				Time Interval (min)	Infiltration (mm)	Cumulative Infiltration (mm)	Infiltration Rate (mm/min)	Infiltration Rate (mm/hour)	Infiltration Rate (m/day)
	Initial Reading	1st Refill Reading	2nd Refill Reading	3rd Refill Reading						
0	61.00	-	-	-	-	-	0.00	-	-	-
10	61.60	-	-	-	10.00	0.60	0.60	0.06	3.60	0.09
20	62.00	-	-	-	10.00	0.40	1.00	0.04	2.40	0.06
30	62.80	-	-	-	10.00	0.80	1.80	0.08	4.80	0.12
40	63.20	-	-	-	10.00	0.40	2.20	0.04	2.40	0.06
50	63.80	-	-	-	10.00	0.60	2.80	0.06	3.60	0.09
60	65.40	-	-	-	10.00	1.60	4.40	0.16	9.60	0.23
70	66.60	-	-	-	10.00	1.20	5.60	0.12	7.20	0.17
80	67.80	-	-	-	10.00	1.20	6.80	0.12	7.20	0.17
90	68.60	-	-	-	10.00	0.80	7.60	0.08	4.80	0.12
100	69.40	-	-	-	10.00	0.80	8.40	0.08	4.80	0.12
110	69.80	-	-	-	10.00	0.40	8.80	0.04	2.40	0.06
120	70.40	-	-	-	10.00	0.60	9.40	0.06	3.60	0.09



Annex D

**SYDNEY
ANALYTICAL
LABORATORIES**

Page 1 of 3

Office:
PO BOX 48
ERMINGTON NSW 2115

Laboratory:
1/4 ABBOTT ROAD
SEVEN HILLS NSW 2147
Telephone: (02) 9838 8903
Fax: (02) 9838 8919
A.C.N. 003 614 695
A.B.N. 81 829 182 852
NATA No: 1884

ANALYTICAL REPORT for:

HUNTER CIVILAB

UNIT 3/62 SANDRINGHAM AVE
THORNTON 2322

ATTN: JAMES WYATT

JOB NO: SAL280060
CLIENT ORDER: P21557
DATE RECEIVED: 22/09/21
DATE COMPLETED: 30/09/21
TYPE OF SAMPLES: SOILS
NO OF SAMPLES: 3



.....
Issued on 30/09/21
Lance Smith
(Chief Chemist)

ANALYTICAL REPORT

JOB NO: SAL280060
CLIENT ORDER: P21557

SAMPLES	pH Sat.	COND uS/cm	Cl %	SO4 %
1 BH1/0.9-1.0m	5.1	730	0.12	0.036
2 BH5/0.6-0.7m	4.6	1420	0.19	0.070
3 BH8/2.0-2.1m	4.9	820	0.12	0.041
MDL	0.1	10	0.001	0.001
Method Code	C23	WA2	C24	C6
Preparation	P4	P4	P4	P4

RESULTS ON DRY BASIS
DATE OF COLLECTION: 03/09/21
WORK REQUEST No.: 5200

ANALYTICAL REPORT

JOB NO: SAL280060
CLIENT ORDER: P21557

METHODS OF PREPARATION AND ANALYSIS

The tests contained in this report have been carried out on the samples as received by the laboratory. In the case where an analyte or group of analytes are received outside of recommended holding times, the analysis will proceed and the report annotated. Analysis is carried out within analyte holding times where possible.

P4 Sample dried, sieved at 9.5mm, split and crushed to -425um

C23 pH - RMS T123

WA2 Conductivity-In House Method A8.25 (1:5 soil/water extract)
 Determined by APHA 2510B

C24 Water Soluble Chloride - RMS T1010
 Determined by APHA 4110B

C6 Water Soluble Sulphate - AS1289.4.2.1
 Determined by APHA 4110B



Annex E

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

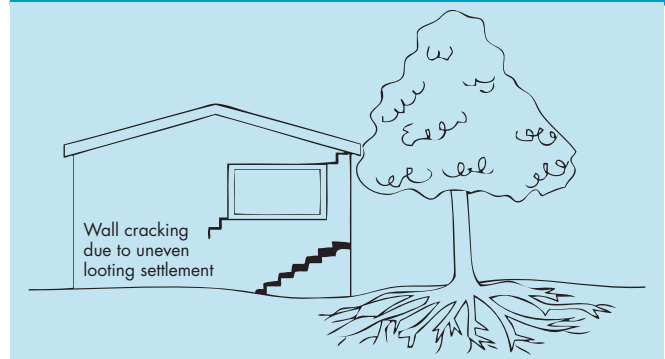
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the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

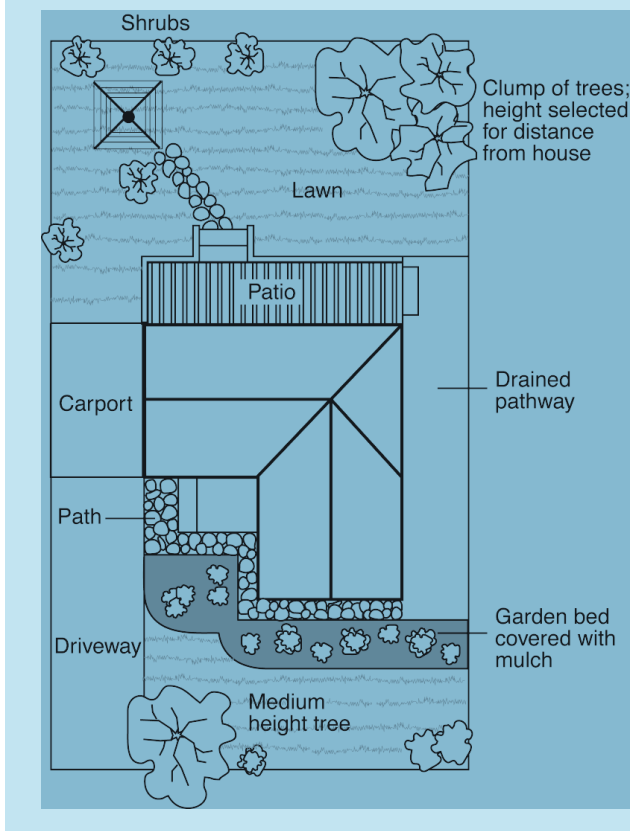
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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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