TGE Taylor Geotechnical Engineering
Geotechnical Civil Engineers & Project Managers

REPORT ON GEOTECHNICAL INVESTIGATION

PROPOSED RETAIL AND RESIDENTIAL DEVELOPMENT

55 KALANG ROAD ELANORA HEIGHTS

CLIENT: A & A CABRERA AND A & T PAPANDREA PARTNERSHIP

PROJECT: TGE21732

DATE: 22 SEPTEMBER 2017

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LRT
Report TGE21732
22 September 2017

REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED RETAIL RESIDENTIAL DEVELOPMENT
55 KALANG ROAD ELANORA HEIGHTS

1. INTRODUCTION

This report details the results of a geotechnical investigation undertaken at 55 Kalang Road Elanora Heights. Ray Fitz-Gibbon & Associates Pty Ltd, architects for the project, requested the investigation on behalf of the property owners, A & A Cabrera and A & T Papandrea Partnership. The investigation was carried out by Taylor Geotechnical Engineering Pty Limited as per Proposal tgeP1721 dated 7th June 2017.

It is understood that the proposed development will comprise demolition of the existing residential and retail shops and construction of a new four level retail residential building with one basement level. The aim of the investigation was to provide information on site and subsurface conditions to assist with design and planning.

The investigation comprised photographic survey, test bores, insitu testing and engineering inspection and assessment. Details of the fieldwork are given in the report, together with comments relating to design and construction practice.

2. SITE DESCRIPTION

The site is located on the eastern side of Kalang Road in Elanora Heights and has the shape and dimensions as shown on Drawing 1 – Site Pan in Appendix 1 with an area of approximately 579.5 m^2. Ground slopes fall to the east with an average slope of 4-5 degrees from front to rear boundaries. The site is bounded by neighbouring properties to the north, south and east with frontage to Kalang Road to the west. A bitumen paved right of carriageway runs along the northern boundary of the site with a two and three level brick retail/residential building located in the western section of the site and a bitumen paved area occupying the majority of the site on the eastern side of the building and a grassed area located adjacent to the eastern (rear) boundary. Views of the site are shown in Photos 1 and 2 in Appendix 3.

Reference to the Sydney 1:100,000 Geological Sheet indicates that the site is underlain by Hawkesbury Sandstone, of the Triassic Period. The Hawkesbury Sandstone formation typically comprises medium to coarse grained quartz sandstone with very minor shale and laminite lenses. The rocks of this formation typically weather to form low and moderately reactive sandy clay soils but highly reactive clay soils are possible.

The results of the fieldwork confirmed that Hawkesbury Sandstone bedrock underlies the site at relatively shallow depth.

3. FIELD WORK METHODS

The fieldwork comprised the drilling of three test bores. The test bores were drilled with a Dingo mounted drilling rig fitted with 100 mm diameter continuous flight augers. A dynamic penetrometer test (DPT) was conducted at each bore location, testing from the surface level to a maximum depth of 2.4 m or prior refusal. The test was conducted in accordance with test method AS 1289.F3.2 and an experienced geotechnical engineer logged the bores on site with strata identification made from the auger cuttings.

4. FIELD WORK RESULTS

Details of the conditions encountered in the test bores are given in the test bore report sheets in Appendix 2 and are summarised below. The bores were drilled to depths ranging from 1.4 to 2.2 metres in the approximate locations shown on Drawing 1 in Appendix 1.

Bore 1 encountered silty sand topsoil overlying sandy filling to 0.6 m then clayey sand to a depth of 2.0 m where very low strength sandstone was encountered. The bore was terminated at a depth of 2.2 m due to auger refusal on low strength sandstone.

Bore 2 encountered sandy filling to 0.6 m underlain by sand to 1.2 m where very low strength sandstone was encountered. Bore 2 was terminated at a depth of 1.9 m due to auger refusal on low strength sandstone.

Bore 3 encountered sandy filling to 0.2 m overlying clayey sand to 0.8 m where extremely low strength sandstone was encountered to 1.2 m then very low strength sandstone. The bore was terminated at a depth of 1.4 m due to auger refusal on low strength sandstone.

The dynamic penetrometer tests indicated that the natural sandy soils were generally in a loose grading to medium dense condition at the time of the investigation and that refusal depth corresponded with the depth of the underlying sandstone bedrock encountered in each of the test bores.

Groundwater seepage was not observed in any of the bores at the time of investigation but the soils were generally in a moist condition. Seepage would be expected during excavation for the basement, particularly after rain but the groundwater table is expected to be well below the base excavation level for this development but drainage provision should be made during construction and in the long term for the life of the development.

5. PROPOSED DEVELOPMENT

It is understood that the proposed development will comprise construction of a new four level retail residential building with a single basement level for parking and storage. Reference to preliminary development application architectural plans by Ray Fitz-Gibbon & Associates Pty Ltd, Job No. J107, Drawings DA02 to DA10 indicates that the proposed basement floor level will vary between RL 90.00 to 90.90, the ground floor level will be at RL 94.080, the first floor level will be at RL 97.25 and the second floor level will be at RL 100.40.

6. COMMENTS

6.1 Inferred Geological Profile

Based on the results of the field work the inferred geological profile underlying the site consists of surficial sandy soils and filling overlying weathered sandstone bedrock grading in strength from extremely low to possibly medium and high strength within the expected range of excavation. Bedrock was encountered at depths of approximately 1.2-2.0 m below existing ground surface levels.

6.2 Stability Risk Assessment

The results of the geotechnical investigation indicated that there is no evidence of recent instability and that currently there are no landslide hazards that would pose an unacceptable risk to property or life on this or any immediate upslope properties. It is expected that the proposed development will be constructed in a manner that will not increase the risk of instability to this or any adjoining sites. This will involve the control of stormwater and provision of adequate shoring measures for proposed excavations (if required).

Assessment of the site has been made in accordance with the methods and requirements as outlined by the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management paper titled 'Landslide Risk Management Concepts and Guidelines' May 2002 and the Australian Geomechanics Society Landslide Taskforce, Landslide Practice Note Working Group paper titled 'Practice Note Guidelines for Landslide Risk Management 2007'.

6.3 Site Preparation and Earthworks

It is understood that approximately 4 m of excavation will be required at the western end of the building platform area and up to 1.0 m at the eastern end of the building platform area in order to achieve the proposed design levels for the basement and rear on-grade parking area. Prior to any cut/filling operations site preparation should include the stripping of all topsoils and vegetation such as grasses and low lying shrubs. The results of the fieldwork indicated that the existing filling and natural surficial sandy soils on site may be suitable for reuse as filling material in any situation where a significant raising of the ground surface level is proposed, but preference should be given to use of any excavated sandstone bedrock. Any imported filling proposed on either building or pavement areas should consist of granular material such as crushed sandstone or similar while backfilling of service or drainage trenches should be done with sand. Filling material should be placed in layers not exceeding 250 mm maximum loose thickness and the material moisture conditioned to within 2% of standard optimum by the addition or removal of water, as appropriate. Each layer should be compacted to a density ratio not less than 95% standard maximum density, increasing to 98% standard maximum over the final two layers and 100% standard maximum for pavement areas.

6.4 Excavation

Based on the design levels indicated on the preliminary development application architectural plans supplied, it is understood that excavation of up to 4 m required at the western end of the building platform area in order to achieve the proposed design levels for the basement level. Conventional earthmoving equipment such as an excavator fitted with a digging bucket, is normally used to excavate residual soils and filling. The use of rippers and hydraulic rock breakers will be required to excavate low or better strength sandstone bedrock or ironstone layers. It is expected that much of the material that will require excavation will consist of sandy soils and weathered sandstone bedrock ranging in strength from very low grading to medium strength and possibly high strength with depth.

Vibration levels are controlled by rock strength and the size of the rock hammer used to excavate the material, therefore if medium or better strength bedrock is encountered and large hydraulic rock hammers are used, precautions will need to be put in place to limit site vibration levels. The use of rock sawing techniques prior to breaking out with rock hammers will reduce vibration associated with the use of rock hammers in medium or higher strength bedrock.

As medium and high strength sandstone may be encountered, requiring the use of hydraulic rock hammer equipment for efficient excavation, it is recommended that a vibration monitor or monitors be set up onsite to check that vibration levels (peak particle velocity levels) are kept below the recommended peak particle velocity. Although a peak particle velocity of 10 mm/sec is recommended by the relevant Australian Standard (AS2187) this level is for ground vibrations due to blasting with explosives which are generally of short duration, experience has shown that cosmetic damage to masonry structures may occur with peak particle velocities of less than 10 mm/sec associated with bulk excavation where longer duration vibrations are generated. If vibration levels exceed 5 mm/sec cosmetic damage to neighbouring masonry structures may result. If the neighbouring structures are of significant age or show signs of foundation movement, then vibration levels should be kept below 3 mm/sec. Vibration limits are for the founding level of adjacent structures.

Large excavation equipment is likely to be required for efficient excavation of medium and high strength sandstone if encountered for this project. As a rough guide, based on previous experience monitoring excavation of medium or higher strength sandstone in the Sydney region, vibration levels are generally kept below 5 mm/sec if the excavator fitted with hydraulic hammer equipment operates at a distance greater than 3 m away from any neighbouring masonry structures for a 300 kg hammer, 6 m for a 600 kg hammer and 20 m for a 900 kg hammer. If the hydraulic hammer equipment is required to operate within these distances then the hammer should be used in short durations with the hammer pointed away from the structure in question (if possible) and the size of the hammer should be minimised or saw cutting and ripping techniques should be used instead of hammers.

If localised excavation to a depth of greater than 1.5 m is required, then the sides of the excavation must be either retained or trimmed to a gradient that will ensure stability in both the short term during construction and the long term. The following table lists suggested batter slopes for materials likely to be encountered during excavation.

Table 1 - Batter Slopes

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	**Long Term/ Permanent**
Compacted filling	1.5:1	2.5:1
Residual Sandy soils	1.5:1	2:1
Clayey Siltstone / Sandstone (extremely & very low strength)	1:1	1.5:1
Siltstone / Sandstone (low strength)	0.5:1	0.75:1 *
Siltstone / Sandstone (medium or better strength)	Vertical *	0.25:1 *

* Dependent upon jointing and the absence of unfavourably oriented joints

6.5 Retaining Walls

Where space limitations preclude the battering of either cut or filled slopes, it will be necessary to provide support to the cut or filled embankments using an appropriate "engineer designed" retaining wall system. Retaining walls will be required for the basement levels. The basement walls will need to be designed as retaining walls with soldier pier walls being a cost effective option.

Lateral earth pressures for a cantilevered wall, or a wall restrained by a single row of ground anchors may be calculated using the following triangular earth pressure distribution:

$H_z = K \gamma z$

Where: H_z = horizontal pressure at depth z

γ = unit weight of soil (20 kN/m^3) or rock (22 kN/m^3)

K = lateral earth pressure coefficient

Pressures acting on retaining walls can be calculated based on the parameters listed in Table 2 for the materials likely to be retained. Surcharge loads due to neighbouring buildings or road corridors should also be considered in the design of any retaining wall.

Table 2 - Retaining Structures Design Parameters

Material	Unit Weight (kN/m^3)	Friction Angle Long Term (Drained)	Cohesion (Drained) (kPa)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
				Active (K_a)	At Rest (K_o)	
Residual clayey soils and well compacted clayey filling	20	$\phi' = 25°$	c'=5	0.35	0.5	2.0
Silty Sands (Loose)	18	$\phi' = 30°$	c'=0	0.35	0.5	3.0
Extremely low strength rock	22	$\phi' = 30°$	c'=10	0.25	0.4	200 kPa
Very low and low strength rock (jointed)	22	$\phi' = 35°$	c'=20	0.20	0.3	400 kPa
Low strength rock	22	$\phi' = 38°$	c'=50	0.1		2000 kPa
Medium strength rock	22	$\phi' = 40°$	c'=250	0.0**		4000 kPa
High Strength Rock	24	$\phi' = 40°$	c'=500	0.0**		6000 kPa

* Ultimate design values

** 0.1 if highly fractured

Retaining walls should be designed for free draining granular backfill and appropriate surface and subsoil drains to either divert or intercept groundwater flow which otherwise could provide surcharging on the walls and additional pressures.

A soldier pier wall with shotcrete infill panels would be an appropriate shoring and retaining system for the proposed development. Depending on the specific design of the basement retaining wall around the site, temporary anchors may or may not be required. It will depend on the stiffness of the proposed walls and embedment depth of the piers as to whether anchors will be necessary. If the wall is outside the zone of influence of adjacent building foundations and services and some wall movement can be tolerated, consideration could be given to using a cantilevered soldier pier wall with pier sockets developed in the medium and high strength sandstone.

The following table provides design parameters for the materials likely to be intersected by the sockets of the piers for the retaining walls.

Table 3 – Pier Design Parameters

SOIL PROFILE	ALLOWABLE SKIN FRICTION (kPa)	ALLOWABLE BASE BEARING (kPa)	ULTIMATE SKIN FRICTION (kPa)	ULTIMATE BASE BEARING (kPa)
Weathered Rock – Extremely low to very low strength.	75	700	100	1500
Weathered Rock – Low strength.	100	1500	300	5000
Weathered Rock – medium strength.	300	3500	750	25000
Weathered Rock – Medium to high strength.	450	6000	1000	60000

A geotechnical strength reduction factor ($ø_g$) of 0.5 is recommended for limit state design. Retaining walls should be designed for full hydrostatic pressure if significant drainage is not installed between the soldier piers. Appropriate surface and subsoil drains (such as strip drains) should be implemented to either divert or intercept groundwater flow which otherwise could provide surcharging on the walls and additional pressures.

6.6 Ground Anchors

Should anchors be required, the anchoring of piles can be accomplished by the use of prestressed rock anchors. It is recommended that these be inclined (usually at 30^o below horizontal) to allow anchoring in the higher strength rock with a free length extending behind a line rising at 45^o from the base of the excavation, or no less than 3 m (whichever is greater) to allow for stressing of the anchor. Anchor design can be based on using a maximum allowable bond stress of 50 kPa for the extremely low strength sandstone, 100 kPa for the very low strength sandstone, 300 kPa for low strength sandstone, 500 kPa for medium strength sandstone and 1000 kPa for high strength sandstone. These values assume that the anchor holes are sufficiently cleaned and free of clay smear and loose debris.

Once the anchors have been installed and the grout allowed adequate curing time, it is recommended that they be proof stressed to at least 125% of their nominal Working Load and then locked off at 80% of Working Load up until the time that the anchors are decommissioned. It is recommended that regular checks should be carried out (such as lift off tests) to ensure that the load is maintained in the anchors throughout the construction period and is not lost due to creep effects.

It should be noted that the permission of neighbouring property owners will be required in order to install ground anchors beneath their properties.

6.7 Foundations

Given that weathered sandstone is likely to be exposed or at relatively shallow depth after excavation over much of the building area, then it is recommended that the foundations for the development found directly on the weathered bedrock. The use of shallow piers or pad footings, founding in the weathered sandstone would be appropriate, with the foundations dimensioned based on founding in at least very low strength sandstone with an allowable bearing capacity of 800 kPa, increasing to 1500 kPa for footings founded in low strength bedrock and 3500 kPa for footings founding on medium strength sandstone.

A geotechnical engineer should inspect and verify the founding strata for all footings at the time of construction. No water should be allowed to ingress the footing excavations prior to concreting as water ingress will soften clay soils or clayey bedrock and reduce the allowable bearing pressure.

Additional information on residential foundations is supplied in CSIRO BTF 18 which is enclosed in Appendix 4.

6.8 Earthquake Site Factor

As the proposed building will be founded on rock, a site sub-soil class of B_e is recommended and a hazard factor of 0.08 should be adopted for earthquake design in accordance with AS 1170.4-2007, the Australian Standard Structural Design Actions Part 4 - Earthquake actions in Australia.

6.9 Site Drainage

The nature of the proposed development means that the development will be affected by both overland and sub-surface flows. In order to maintain site stability over the design life of the development, it will be essential to incorporate both upslope (at street level) and subsurface drainage. Any proposed drainage system should be designed by a suitable qualified professional.

6.10 Geotechnical Verification

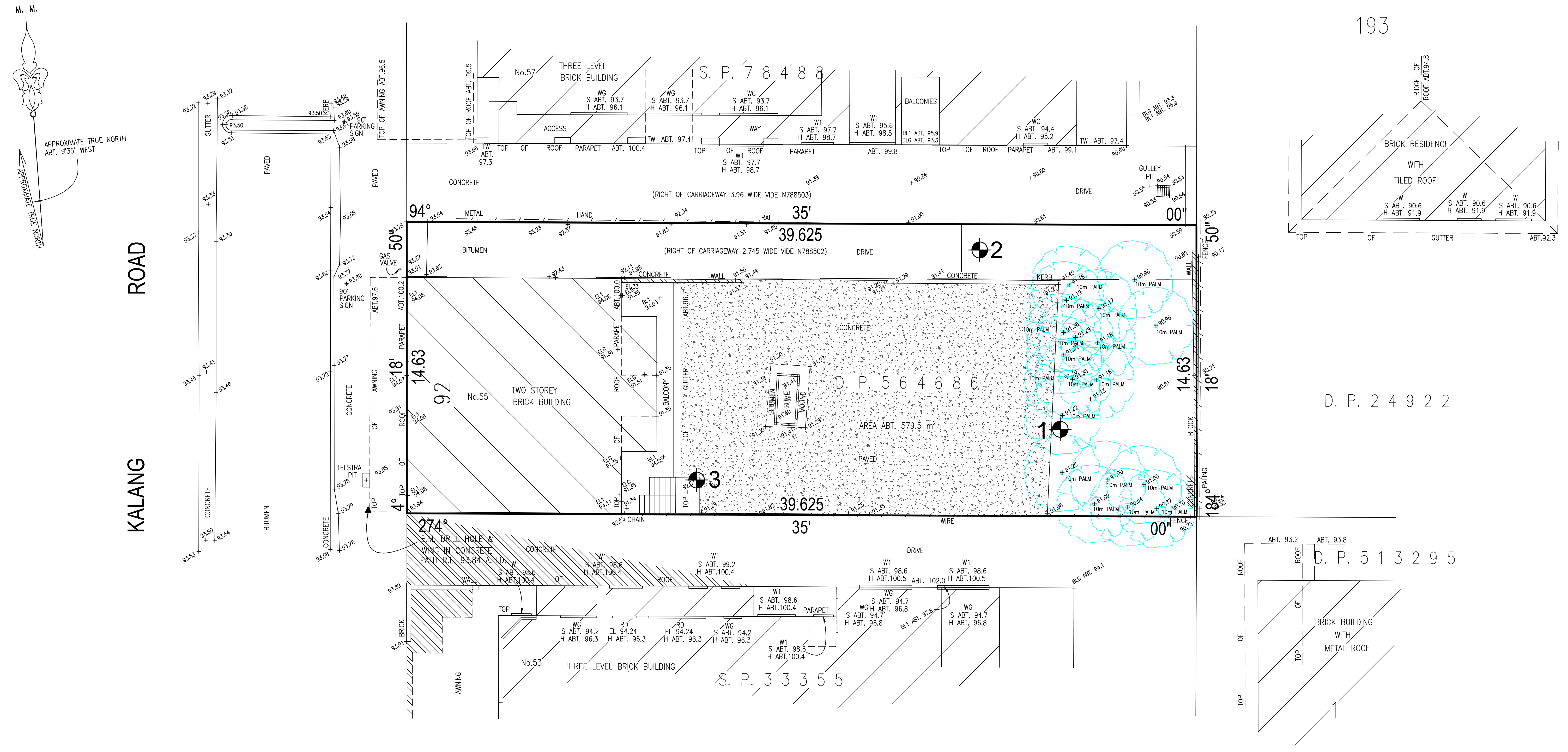
In order to verify design bearing capacities and founding strata a certification schedule will be required. A geotechnical engineer or engineering geologist should inspect and verify the founding strata for any footings at the time of construction to ensure that they comply with the certification schedule. If the founding strata are not inspected at the time of construction, then geotechnical certification cannot be provided. Councils are now generally requiring geotechnical certification of foundations prior to final approval of new developments.

TAYLOR GEOTECHNICAL ENGINEERING PTY LIMITED,

LTaylor

Lachlan Taylor
MIE Aust. CPEng. NER.
Principal Geotechnical Engineer

Appendix 1



Approximate location of test bore.

Note: Survey carried out by others.

TGE *Taylor Geotechnical Engineering* Geotechnical Civil Engineers & Project Managers	*Taylor Geotechnical Engineering Pty Ltd* *ABN: 22 120 426 155* *PO Box 1906* *Phone: (02) 9999 6625* *Dee Why NSW 2099* *Fax: (02) 9999 6650* *taylorgeotech@bigpond.com* *Mobile: 0407 984 128*	Drawing 1 - Site Plan Proposed Retail Residential Development 55 Kalang Road Elanora Heights	Project: TGE21732 Date: 22 September 2017 Scale: As Shown

Appendix 2

TEST BORE REPORT

CLIENT: A & A Cabrera & A & T Papandrea Partnership — DATE: 18-Aug-2017 — Bore No: **1**

PROJECT: Proposed Retail Residential Development — PROJECT No.: TGE21732 — 1 of 1

LOCATION: 55 Kalang Road Elanora Heights — SURFACE LEVEL: RL = 91.2*

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Blows/150mm N Value	Core Recovery%
0.00	TOPSOIL - Grey brown, fine grained silty sand.				
0.20	FILLING - Orange brown & grey brown, fine to medium grained sand with a trace of clay.				
0.60	CLAYEY SAND - Medium dense, orange brown, fine to medium grained clayey sand with ironstone gravel.				
2.00	SANDSTONE - Very low strength, orange brown, yellow brown and red brown, fine to medium grained sandstone.				
2.20	TEST BORE DISCONTINUED AT 2.2 METRES. Auger refusal on low strength sandstone.				

RIG: Dingo Mounted

TYPE OF BORING: 100mm diameter auger

GROUND WATER OBSERVATIONS: No Free Groundwater Observed.

REMARKS:*RL interpolated from survey plan.

DRILLER: Contractor

LOGGED: Taylor

CHECKED: dkT

SAMPLING & IN SITU TESTING LEGEND

D = Disturbed auger sample

B = Bulk sample

Ux = x mm dia. Tube Sample

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TEST BORE REPORT

CLIENT: A & A Cabrera & A & T Papandrea Partnership   DATE: 18-Aug-2017   Bore No: **2**

PROJECT: Proposed Retail Residential Development   PROJECT No.: TGE21732   1 of 1

LOCATION: 55 Kalang Road Elanora Heights   SURFACE LEVEL: RL = 91.4*

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Blows/150mm N Value	Core Recovery%
0.00	FILLING - Dark brown, fine grained silty sand.				
0.20	FILLING - Red brown & grey brown, fine to medium grained sand with ironstone gravel.				
0.60	SAND - Medium dense, yellow brown, fine to medium grained sand with some silt & a trace of clay.				
1.20	SANDSTONE - Very low strength, orange brown and red brown, fine to medium grained sandstone with ironstone banding.				
1.90	TEST BORE DISCONTINUED AT 1.9 METRES. Auger refusal on low strength sandstone.				

RIG: Dingo Mounted   DRILLER: Contractor

TYPE OF BORING: 100mm diameter auger   LOGGED: Taylor

GROUND WATER OBSERVATIONS: No Free Groundwater Observed.   CHECKED:

REMARKS:*RL interpolated from survey plan.

SAMPLING & IN SITU TESTING LEGEND

D = Disturbed auger sample

B = Bulk sample

Ux = x mm dia. Tube Sample

Taylor Geotechnical Engineering

TEST BORE REPORT

CLIENT: A & A Cabrera & A & T Papandrea Partnership DATE: 18-Aug-2017 Bore No: **3**

PROJECT: Proposed Retail Residential Development PROJECT No.: TGE21732 1 of 1

LOCATION: 55 Kalang Road Elanora Heights SURFACE LEVEL: RL = 92.5*

Depth (m)	Description of Strata	Sampling & In Situ Testing			
		Type	Depth (m)	Blows/150mm N Value	Core Recovery%
0.00	FILLING - Grey brown & dark brown silty sand with some bitumen.				
0.20	CLAYEY SAND - Loose to medium dense, yellow brown, fine to medium grained, damp to wet, clayey sand.				
0.80	SANDSTONE - Extremely low strength, orange brown & red brown, fine to medium grained clayey sandstone.				
1.20	SANDSTONE - Very low strength, orange brown, light grey and red brown, fine to medium grained sandstone.				
1.40	TEST BORE DISCONTINUED AT 1.4 METRES. Auger refusal on low strength sandstone.				

RIG: Dingo Mounted DRILLER: Contractor

TYPE OF BORING: 100mm diameter auger LOGGED: Taylor

GROUND WATER OBSERVATIONS: No Free Groundwater Observed. CHECKED:

REMARKS:*RL interpolated from survey plan.

SAMPLING & IN SITU TESTING LEGEND

D = Disturbed auger sample

B = Bulk sample

Ux = x mm dia. Tube Sample

Taylor Geotechnical Engineering

RESULTS OF DYNAMIC PENETROMETER TESTS

CLIENT: A & A Cabrera & A & T Papandrea Partnership

DATE: 18 August 2017

PROJECT: Proposed Retail Residential Development

PROJECT No: TGE21732

LOCATION: 55 Kalang Road Elanora Heights

SHEET: 1 of 1

	PENETRATION RESISTANCE BLOWS / 150mm									
TEST LOCATION	**1**	**2**	**3**							
DEPTH (m)										
0.00 - 0.15	3	2	2							
0.15 - 0.30	6	2	3							
0.30 - 0.45	14	7	2							
0.45 - 0.60	17	7	4							
0.60 - 0.75	5	6	2							
0.75 - 0.90	6	6	3							
0.90 – 1.05	5	3	6							
1.05 – 1.20	5	4/100mm	9							
1.20 – 1.35	2		10/50mm							
1.35 – 1.50	2									
1.50 – 1.65	2									
1.65 – 1.80	7									
1.80 – 1.95	10									
1.95 – 2.10	5/30mm									
2.10 – 2.25										
2.25 – 2.40										
2.40 – 2.55										
2.55 – 2.70										
2.70 – 2.85										
2.85 – 3.00										

TEST METHOD:AS 1289.F3.2, CONE PENETROMETER YES

AS 1289.F3.3, FLAT END PENETROMETER

TESTED BY: Taylor

REMARKS:

Taylor Geotechnical Engineering

Appendix 3

Photo 1 – View of site from Kalang Road, looking east.

Photo 2 – View of site from rear boundary, looking west.

Appendix 4

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 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, 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 with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

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

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

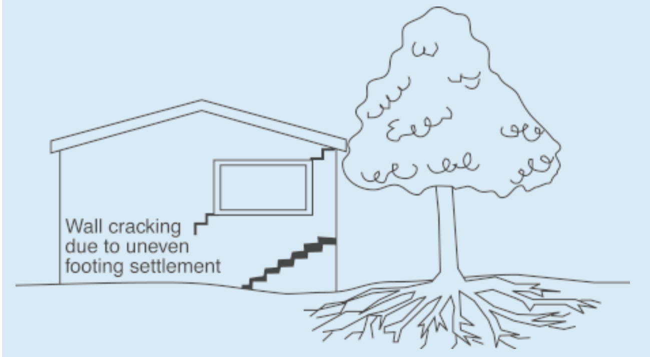
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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

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

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

Trees can cause shrinkage and damage



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

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 cause 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.

AS 2870 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.

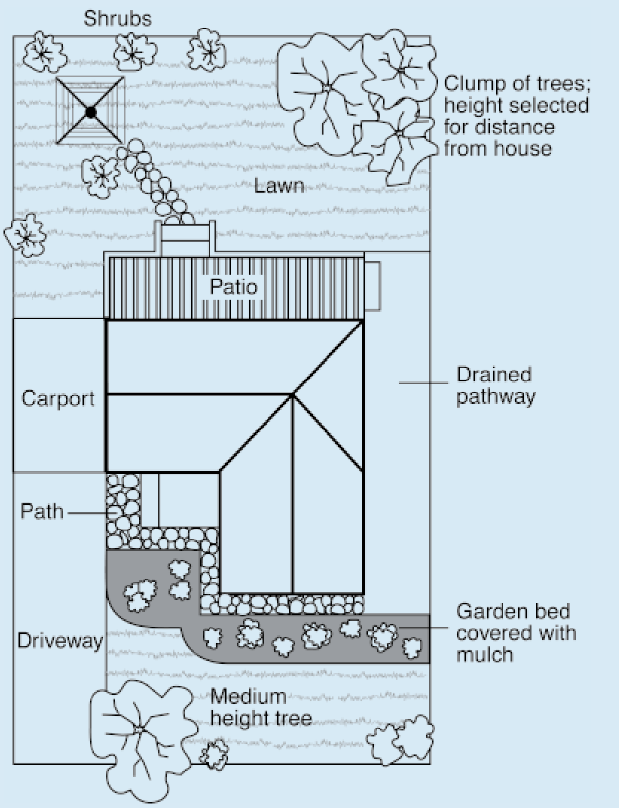
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

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 depend on number of cracks	4

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

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