Geotechnical Consultants Australia

Maree Jaloussis & Michael Hayes

Geotechnical Investigation Report

Proposed Development at:
107 Iris Street
Beacon Hill NSW 2100

G19377-1

3rd December 2019

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Report Distribution

Geotechnical Investigation Report

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1. INTRODUCTION

1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 107 Iris Street Beacon Hill NSW 2100 (the site). The investigation was commissioned by Maree Jaloussis & Michael Hayes (the client), and was carried out on the 29th November 2019.

The purpose of the investigation was to assess the subsurface conditions over the site at the borehole and testing locations (where accessible and feasible), and provide necessary recommendations from a geotechnical perspective for the proposed development.

The findings presented in this report are based on our subsurface investigation and our experience with subsurface conditions in the area. This report presents our assessment of the geotechnical conditions, and has been prepared to provide preliminary advice and recommendations to assist in the preparation of preliminary designs and construction of the ground structures for the proposed development.

For your review, **Appendix A** contains a document prepared by GCA entitled "Important Information About Your Geotechnical Report", which summarises the general limitations, responsibilities, and use of geotechnical engineering reports.

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises subdivision of the site into two (2) lots, followed by construction of a two (2) storey residential dwelling within the front portion of the site, accompanied by an attached secondary dwelling and garage. We note that no underground infrastructures (i.e. basement, etc.) are projected as part of the proposed development.

The Finished Floor Levels (FFL)s of the proposed developments ground floor level are set to be at Reduced Levels (RL)s of 132.850m to RL134.470m Australian Height Datum (AHD).

Based on this information and existing site topography and levels, cut and fill anticipated to be required for construction of the proposed development, with locally deeper excavations for the proposed building footings and service trenches also expected to be required as part of the proposed development.

It should be noted that excavation depths are expected to vary across the site, and have been inferred based off the existing site levels shown on the site survey plan and proposed development FFLs on the architectural drawings, referenced in Section 1.3 below.

1.3 Provided Information

The following relevant information was provided to GCA prior to the geotechnical investigation and during preparation of this report:

- Architectural drawings prepared by Lifestyle Home Designs, titled "Subdivision & New Home 107 Iris Street Lot – 18 D.P. 19022 Beacon Hill NSW 2100", referenced project No. 1836, included drawing nos. DA 01 to DA 04 inclusive, and dated November 2019.
- Site survey plan prepared by Survcorp Consulting Surveyors Pty Ltd, titled "Plan Showing Physical Features and Levels at No. 107 Iris Street, Beacon Hill Part Lot 18 in DP 19022", referenced No. 3435, and dated 2nd October 2018.

1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the borehole and testing locations (where accessible and feasible), and to provide professional advice and recommendations on the following based on requirements provided to GCA by the client:

- General assessment of any potential geotechnical issues that may affect any surrounding infrastructures, buildings, council assets, etc., along with the proposed development.
- Excavation conditions and recommendations on excavation methods in soils and rocks, to restrict any ground vibrations.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site (for ultimate limit state and serviceability loads).
- Groundwater levels which may be determined during the geotechnical investigation.
- Preliminary site lot classification in accordance with Australian Standards (AS) 2870-2011.
- General geotechnical advice on site preparation, filling and subgrade preparation.

1.5 Scope of Works

Fieldwork for the geotechnical investigation was undertaken by an experienced geotechnical engineer, following in general the guidelines outlined in AS 1726-2017. The scope of works included:

- Submit and review Dial Before You Dig (DBYD) plans, and any other plans provided by the client of existing buried services on the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected boreholes and testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible), and identify any relevant features of the site.
- Hand augering of one (1) borehole at a selected location within the site (where accessible and feasible), using hand operated equipment, to a practical refusal depth of approximately 0.4m existing ground level (bgl) within the site. The borehole is identified as borehole BH1.
- Dynamic Cone Penetrometer (DCP) testing immediately adjacent to borehole BH1, and at selected locations within the site (where accessible and feasible), using hand operated equipment, to varying practical refusal depths of approximately 0.2m to 0.65m bgl. The tests are identified as DCP1 to DCP5 inclusive.
    - The approximate locations of the borehole and DCP tests are shown on **Figure 1, Appendix B** of this report.
- Collection of soil samples during fieldwork for any laboratory tests which may be required.
- Reinstatement of borehole BH1 with available soil displaced during augering.
- Preparation of this geotechnical engineering report.

1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained during hand augering and DCP testing at the selected borehole and testing locations within the site (where accessible and feasible). It is recommended that further geotechnical inspections should be carried out during construction to confirm the subsurface conditions across the site and foundation bearing capacities have been achieved.

Consideration should also be given to additional machine drilled boreholes and rock strength testing carried out to confirm the ground conditions and estimated rock strength underlying the site, and to help assist in final designs of the proposed development. This recommendation should be confirmed by the project geotechnical engineer and structural engineer during/following design stages of the proposed development.

2. SITE DESCRIPTION

2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 1 below.

Table 1. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site is located within a residential area along Iris Street carriageway, approximately 170m north of Warringah Road thoroughfare.
Site Address	107 Iris Street Beacon Hill NSW 2100
Approximate Site Area[1]	2,254m^2
Local Government Authority	Northern Beaches Council
Site Description	At the time of the investigation a residential dwelling was present within the rear portion of the site, accompanied by associated concrete pavements and an in-ground swimming pool. The remaining site area, including the proposed development (investigation) area within the front portion of the site, was predominately covered in grass, vegetation and a number of mature trees scattered throughout. Sandstone outcrops were also present and exposed within the front portion of the site, as well as a number of retaining walls.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, etc.)	• Middle Creek – 190m north-west of the site.
Site Surroundings	The site is located within an area of residential use, and is bounded by: • Iris Street carriageway to the north. • Residential property at No. 45 Oxford Falls Road to the east. • Residential properties at No. 18 and No. 20 Dareen Street to the south. • Residential property at No. 105 Iris Street to the west.

[1]Site area is approximate and based off the site survey plan referenced in Section 1.3.

2.2 Topography

The local topography surrounding the site generally falls towards the north to north-east, and towards the east. The site topography also generally slopes towards the north to north-east, with levels within the site varying from approximately RL126.43m to RL137.93m AHD.

It should also be noted that the site topography, levels and slopes are approximate and based off the site survey plan referenced in Section 1.3 of this report, along with observations made during the geotechnical investigation, and reference to NSW Six Maps (https://maps.six.nsw.gov.au/). The actual topography in areas inaccessible during the site investigation, including areas under the existing infrastructures, along with the site and local topography and levels are expected to vary from those outlined in this report.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Sydney 1:100,000 Geological Series Sheet 9130 First Edition, dated 1983, by the Geological Survey of New South Wales, indicates the site is located within a geological region generally underlain by Triassic Aged Hawkesbury Sandstone (Rh). The Hawkesbury Sandstone typically comprises "medium to coarse grained quartz sandstone".

The site is also situated approximately 80m to 90m north-east of a geological boundary/region generally underlain by Triassic Aged Hawkesbury Sandstone of "shale lenses" formation (Rhs), which normally comprises "shale, laminite".

A review of the regional maps by the NSW Government Environment and Heritage indicates the site is generally located within the Lambert (la) landscape group, which is recognised by undulating to rolling rises and low hills on Hawkesbury sandstone. Soils of the Lambert group typically have very high soil erosion hazard, rock outcrop, seasonally perched water tables, shallow, highly permeable soil, and very low soil fertility. Local reliefs are generally 20m to 120m, with slopes typically of approximately 20% in gradient and rock outcrops greater than 50%. Soils of the Lambert group are generally slightly (pH 6.0) to extremely (pH 3.5) acidic.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site during this geotechnical investigation are summarised in Table 2 below, and are interpreted from the assessment results. It should be noted that Table 2 presents a summary of the overall site conditions, and reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes detailed in **Appendix C**. Any rock description has been based on Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

It should be noted that soil consistency/strength assessed by during DCP testing in the site are approximate and variances should be expected throughout. Due to the variable ground conditions throughout the site, it is recommended that confirmation of the subsurface materials be carried out during construction, or by additional boreholes and rock strength testing. It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Based on the geotechnical investigation, along with our experience and observations made within the site and local region, it is inferred that bedrock of variable strength and weathering is underlying the majority of the proposed development area within the front portion of the site at relatively shallower depths of 0.2m to 0.7m (varying throughout), and certain areas with exposed sandstone outcrops.

Visual geotechnical assessment of the exposed sandstone outcrops within the site indicated the presence of generally highly weathered and medium estimated strength (or better) bedrock. Defects predominately in the form of joints throughout the bedrock were observed to be present, which we anticipate to become generally loose during excavation works. Vegetation and mature trees covered portions of the exposed bedrock.

We note that the actual assessment of the defects and potential weathered zones within the underlying bedrock were not carried out during the geotechnical investigation. Therefore, the estimated rock strength is approximate and soley based on a visual assessment by a geotechnical engineer, and variances should be expected throughout.

Although no natural soils were encountered, their presence should not be precluded within the site and during construction, predominately at locations and depths not assessed during the geotechnical investigation.

Table 2. Summary of Subsurface Conditions

			Borehole ID	**BH1**
Unit	**Unit Type**	**Description**	**Estimated Consistency/ Strength**	**Depth/Thickness of Unit (m)**
			Approximate RL at Borehole Location (m AHD)	RL134.1
1	Fill	Silty SAND, fine grained, gravel inclusions.	N/A	0.0 – 0.35
2	Bedrock[1]	SANDSTONE, fine grained, extremely weathered.	Extremely low[2]	0.35 – 0.4

[1]Confirmation of the actual composition, continuity, strength and depth of the underlying bedrock should be carried out by a geotechnical engineer by additional borehole drilling and rock strength testing, or by inspection during construction.
[2]Bedrock strength inferred to become generally low to medium estimated strength (or better), shortly below practical hand auger refusal depth within the site.
Notes:
- N/A = Not Applicable.
- Clay seams, and defects, fractured and extremely weathered zones are expected to be present throughout the underlying bedrock, predominately at depths and locations unobserved during the geotechnical investigation.
- Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

A summary of the inferred subsurface conditions encountered and inferred during DCP testing are summarised in Table 3 below, with the DCP testing results attached in **Appendix E**. Ground conditions depicted in Table 3 below are inferred based on the DCP testing results, and confirmation should be carried out by additional testing or during construction by inspection. It should also be noted that the underlying subsurface conditions should be confirmed during construction of the proposed development as site conditions may vary throughout the site.

It should also be noted that DCP tests and higher blow counts encountered may be affected by factors such as gravels, ironstone bands, well consolidated soils and highly cemented sands, and other deleterious materials which may be present within the underlying soils, along with tree rootlets extending throughout the soils from trees and vegetation within the vicinity. These results should be read in conjunction with the boreholes, and geotechnical confirmation should be carried out during construction by inspection or by additional borehole drilling and testing as site conditions may vary.

Table 3. Summary of Inferred Subsurface Conditions From DCP Testing

	DCP ID	**DCP1**	**DCP2**	**DCP3**	**DCP4**	**DCP5**
Unit	**Unit Type**	**Depth/Thickness of Unit (m)**				
1	*Inferred* Fill[2]	0.0 – 0.25	0.0 – 0.35	0.0 – 0.35	0.0 – 0.65	0.0 – 0.2
2	*Inferred* Bedrock[3]	0.3	0.4	0.4	0.7	0.2

[1]Assumed fill thickness based on DCP blow counts and observations made during the geotechnical investigation. Thickness of the fill layer is expected to vary from those indicated in Table 3.
[2]Inferred bedrock composition, continuity, strength and depth should be confirmed by a geotechnical engineering either prior to construction by additional boreholes and rock strength testing, or during construction by inspection. Bedrock inferred to be present at or shortly below the practical DCP testing refusal depths.
Notes:
- Clay seams, defects and fractured and extremely weathered zones are expected to be present throughout the underlying inferred bedrock, predominately at depths and locations unobserved during the geotechnical investigation.
- Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

3.2 Groundwater

No groundwater was encountered or observed at all testing locations within the site to a maximum depth of approximately 0.65m bgl, or through the exposed sandstone outcrops within the site. It is noted that borehole BH1 was immediately backfilled following completion of augering which precluded longer term monitoring of groundwater levels within the site.

Groundwater which may be present within the site is expected to be in the form of seepage through the voids within the underlying fill material, and through the pore spaces between particles of unconsolidated natural soils (if present), or through networks of fractures and solution openings in consolidated bedrock underlying the site. Although no groundwater was encountered or observed during the geotechnical investigation, its presence should not be precluded within the site and during construction.

It should be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties. Groundwater monitoring should be carried out during construction, to assess any groundwater inflows within the site.

4. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

4.1 Dilapidation Survey

It is recommended that prior to demolition, excavation and construction, a detailed dilapidation survey be carried out on all adjacent buildings, structures, council assets, road reserves and infrastructures that fall within the vicinity of the proposed development. A dilapidation survey will record the condition of existing defects prior to any works being carried out within the site. Preparation of a dilapidation report should constitute as a "Hold Point".

4.2 General Geotechnical Issues

The following aspects have been considered main geotechnical issues for the proposed development:

- Preliminary site lot classification.
- Excavation conditions.
- Foundations

Based on results of our assessment, a summary of the geotechnical aspects above and recommendations for construction and designs are presented below.

4.3 Preliminary Site Lot Classification

Based on the geotechnical investigation and observations made during the site investigation, fill material is expected to be underlain by bedrock at relatively shallower depths across the proposed development area of approximately 0.2m to 0.7m (varying throughout), and certain areas with exposed sandstone outcrops. Due to the site and subsurface conditions, no laboratory testing was carried out on any natural soils present underlying the proposed development area.

The governing site lot classification in accordance with AS 2870-2011 has been identified as **"Class P" (Problematic Site)** for the overall site, due to the presence of existing infrastructures and matures trees within and adjoining the site, causing abnormal and changing moisture conditions.

Based on the augered borehole and DCP tests carried out within the site, and assessment of the exposed sandstone outcrops within the site, AS 2870-2011 indicates the site may be classified as a **"Class S"** site, for design and construction of the proposed developments foundation system founded below any topsoil, slopewash, fill or other deleterious material, being entirely on the bedrock underlying the proposed development area (subject to confirmation).

This classification is solely based on assessment of the subsurface conditions are the selected borehole and testing locations within the site, and confirmation should be carried out as outlined in this report.

Foundation design and construction should be carried out as outlined in Section 4.6 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying fill material, natural soils and bedrock should be made prior to construction by additional borehole drilling and rock strength testing, or by inspection during construction.

Where ground conditions vary from those outlined at the boreholes and testing locations, and confirmation of the actual depth of underlying fill material, natural soils and bedrock has not been carried out by a geotechnical engineer as outlined in this report, and where the building foundations are not proposed to be constructed on the bedrock underlying the site, GCA should be contacted immediately, and the building foundations be designed and constructed as a "Class P" site.

Footing designs should take into consideration the effect of recent removal and planting of trees, along with any future tree removal within the vicinity of the proposed development on soil moisture conditions. Sufficient time should be given for soil moisture to re-equilibrate following any removal or planting of trees within the proposed development area, or specific engineering assessment and design will be required on the foundation design.

Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

Based on the site lot classification outlined above, it is recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix F**.

4.4 Excavation

Cut and fill are expected to be required for construction of the proposed development, with locally deeper excavations also anticipated to be required for the proposed building footings and service trenches across the site.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavation will extend through Unit 1 (fill) and Unit 2 (bedrock), throughout the majority of the proposed development area, as discussed in Section 3, and outlined in Table 2 and Table 3 above.

The possibility for encountering higher strength bedrock (i.e. medium to high estimated strength, or better) should not be precluded during excavation/construction, predominately where deeper excavations are required across the site, and in areas and at depths not assessed during the geotechnical investigation, due to the limited investigation carried out within the site.

Consultation should be made with subcontractors to discuss the feasibility and capability of machinery for the proposed development for the existing site conditions.

4.4.1 Excavation Assessment

Excavation through softer soils and extremely low to low estimated strength bedrock should be feasible using conventional earth moving excavators, typically medium to large hydraulic excavators. Smaller sized excavators may encounter difficulty in high strength bands of soils and rocks which may be encountered. Where high strengths bands are encountered, rock breaking or ripping should be allowed for. Removal of the existing pavements and associated infrastructures within the site are also expected to require larger excavators and rock breaking and ripping.

Excavation of medium to higher estimated strength bedrock, which may be encountered across the site, would require higher capacity excavators, bulldozers or similar, for effective removal of the rock. This excavation will require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment. Furthermore, excavation for the proposed building footings and service trenches may require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment, with the possibility of rock saw cutting.

Should rock hammering be used for the excavation in the underlying bedrock, excavation should be carried out away from the adjoining structures, with vibrations transmitted being monitored to maintain vibrations within acceptable limits. Rock saw cutting should be carried out (where required), around the perimeter of excavations, prior to any rock breaking commencing.

Demolition, excavation and construction activities (or the like) will generate both vibration and noise, whilst being carried out within the site. Vibration control measures should be implemented as part of the construction process. All excavation works should be carried out in accordance with the NSW WorkCover code of practice for excavation work.

4.5 Vibration Monitoring and Controls

Particular care will be required to ensure that adjacent buildings and infrastructures (i.e. buildings, road reserves, etc.) are not damaged during demolition, excavation and construction activities (or the like) due to excessive vibrations. Therefore, appropriate methods should be adopted which will limit ground vibrations to limits not exceeding the following maximum Peak Particle Velocity (PPV) for adjacent structures:

- Sensitive and/or historical structures – **2mm/sec**
- Residential and/or low rise structures – **5mm/sec**
- Unreinforced and/or brick structures – **10mm/sec**
- Reinforced and/or steel structures – **25mm/sec**
- Commercial and/or industrial buildings – **25mm/sec**

Vibrations transmitted by the use of rock hammers are unacceptable and not recommended. To minimise vibration transmission to any adjoining infrastructures, and to ensure vibration limits remain within acceptable limits, rock saw cutting using a conventional excavator with a mounted rock saw (or similar) should be carried out as part of excavation prior to any rock breaking commencing. Although rock hammering is unacceptable and not recommended, if necessary during excavation, it is recommended that hammering be carried out horizontally along pre-cut rock boulders or blocks provided by rock saw cutting, and should remain within limits acceptable. This should be monitored at all times during excavation.

The effectiveness of all the above-mentioned approaches must be confirmed by the results of vibration monitoring. The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavations are restricted to the values indicated in Table 4 below.

Table 4. Rock Breaking Equipment Recommendations

Distance From Adjoining Structures (m)	Maximum PPV 5mm/sec		Maximum PPV 10mm/sec[1]	
	Equipment	Operating Limit (Maximum Capacity %)	Equipment	Operating Limit (Maximum Capacity %)
1.5 to 2.5	Jack Hammer Only (hand operated)	100	300kg Rock Hammer	50
2.5 to 5.0	300kg Rock Hammer	50	300kg Rock Hammer	100
			600kg Rock Hammer	50
5.0 to 10.0	300kg Rock Hammer	100	600kg Rock Hammer	100
	600kg Rock Hammer	50	900kg Rock Hammer	50

[1]Vibration monitoring is recommended for the use of a maximum PPV of 10mm/sec.

A vibration monitoring plan is recommended to be developed to monitor construction activities, and their effects on adjoining infrastructures. A vibration monitoring plan may be carried out attended or unattended. An unattended vibration monitoring must be fitted with alarms in the form of strobe lights, sirens or live alerts sent to the vibration monitoring supervisor, which are activated when the vibration limit is exceeded.

A geotechnical engineer should be contacted immediately if vibrations during construction or in adjacent structures exceed the values outlined above, and work should immediately cease. It is recommended a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 4.1. This should be considered a "Hold Point".

4.6 Foundations

Based on the geotechnical investigation and observations made during the geotechnical investigation within the proposed development area, fill material as discussed in Section 2 above are expected to underlie the majority of the proposed development area, overlying sandstone bedrock at relatively shallower depths throughout.

The possibility for encountering higher strength bedrock (i.e. medium to high estimated strength, or better) in areas of deeper excavation across the site should not be precluded, providing the ground conditions are confirmed by a geotechnical engineer by additional borehole drilling and rock strength testing, or during construction by inspection.

Variable strength natural soils (if present) and fill material are likely to result in total and differential settlement under working load, and not adequately support shallow foundations for the proposed development within the site. Removal of the fill material within the proposed development area should be carried out prior to construction of the proposed building foundation system.

It is noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site. It is therefore recommended that confirmation of the underlying ground conditions be confirmed by a geotechnical engineer prior to construction by additional borehole drilling and testing, or during construction by inspection.

4.6.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a suitable foundation system comprising shallow foundations typically containing pad and/or strip footings, or a cast in-situ reinforced concrete raft slab constructed on appropriate strength, and consistent and competent sandstone bedrock underlying the site are likely to be adopted for the proposed development.

Shallow foundations should include local slab thickening to support internal walls and columns, where a raft foundation should include slab thickening to provide strip and pad footings for the support of the internal walls and columns, respectively. The use of settlement reduction piles with increased sock depths may also be considered, in order to increase the resistance against lateral loading induced by earthquake or winds, and to achieve higher bearing capacities than at the basement FFL.

Installation of piles (where adopted) should be complemented by inspections carried out by a geotechnical engineer during construction. The actual depth and embedment of the piles should be assessed by the project structural engineer, with all structural elements also inspected and approved by a suitably qualified structural engineer. Confirmation of the actual subsurface conditions underlying the proposed development area should be carried out by a geotechnical engineer during construction. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing ground conditions. Consultation should be made with specialist subcontractors to discuss the feasibility of piles and machinery for the existing site conditions.

It should be noted that due to the potential variable bedrock conditions throughout the site following bulk excavation, precaution should be taken for the design of the building foundation system, taking into consideration the preliminary geotechnical design parameters in Table 5 below.

Higher allowable bearing capacities may be justified subject to confirmation by inspection during construction, or by additional borehole drilling and rock strength testing. Where higher estimated strength bedrock is encountered during construction, GCA should be contacted to re-assess the preliminary allowable bearing capacities provided in this report. Adoption of higher preliminary bearing capacities for the design of the proposed development outlined in Table 5 should be confirmed by a geotechnical engineer, as discussed in this report.

Given the potential for variable ground conditions and soil reactivity across the site (as discussed in Section 4.3), it is recommended that all foundations are constructed on consistent and competent bedrock throughout, in order to provide uniform support and reduce the potential for differential settlements. This could be attained by strip or pad footings where the suitable bearing capacity is achieved or exposed at bulk level excavation, and pile foundations elsewhere. Reference should be made to the estimated levels of the subsurface conditions outlined in this report, and compared to the final bulk excavation levels across the site.

Installation of piles may be required where the axial and working loads transmitted through the building walls and columns exceed the bearing pressure of the bedrock exposed at the proposed developments FFLs. These should be socketed into consistent and appropriate bedrock underlying the site. For cases where resistance against lateral loading induced by earthquakes or winds, and to achieve higher bearing capacities, piles may also be required. Piles sufficiently socketed into higher strength bedrock may achieve higher allowable bearing capacities, subject to confirmation by a geotechnical engineer by additional borehole drilling and rock strength testing, or by inspection during construction.

Where ground conditions vary from those outlined in this report, GCA should be contacted immediately for further advise.

Table 5 provides preliminary recommended geotechnical design parameters.

Table 5. Preliminary Recommended Geotechnical Design Parameters

Unit Type/Material		Maximum Allowable (Serviceability) Values (kPa)		
		End Bearing Pressure[1]	**Shaft Adhesion (Compression)**	**Shaft Adhesion (Tension)**
Fill (Unit 1)		N/A	N/A	N/A
Bedrock (Unit 2)[2]	**EL – VL**	800	50	25
	L[3, 4]	1,500	100	50
	M[3, 4]	2,000	200	100

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep (pile) foundations.
[2]Confirmation of the underlying bedrock composition, continuity, strength and depth should be confirmed by additional borehole drilling and rock strength testing, or during construction by a geotechnical engineer.
[3]Preliminary only, and inferred to be present within the site at depth. Subject to confirmation by a geotechnical engineer during construction by inspection, or by additional borehole drilling and rock strength testing.
[4]At least Class IV Sandstone, or better. Subject to confirmation by a geotechnical engineer, as discussed in this report.
Notes:

- EL = Extremely Low estimated strength, VL = Very Low estimated strength, L = Low estimated strength, M = Medium estimated strength.
- N/A = Not Applicable. Not recommended for the proposed development.
- The composition, depth, strength and continuity of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.
- It is recommended that geotechnical inspections on the foundations are completed by a geotechnical engineer to determine the material and confirm the required bearing capacity has been achieved

4.6.2 Geotechnical Comments

Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing site conditions. It should be noted that higher bearing capacities may be justified for the proposed foundations subject to confirmation by inspection during construction, or by additional borehole drilling and rock strength testing.

Foundations located within the “zone of influence” of any services or sensitive structures should be supported by a piled foundation. The depths of the piles should extend below the “zone of influence” and should ignore any shaft adhesion. Appropriate measures should be taken to ensure that any services or sensitive structures located within the “zone of influence” of the proposed development are not damaged during and following construction.

Specific geotechnical advice should also be obtained for footing deigns and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009.

It is recommended that suitable drainage and the use of impermeable surfaces be implemented as a precaution as part of the design and construction of the proposed development in order to divert surface water away from the building, and help eliminate or minimise surface water infiltration to minimise moisture within the soils. Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

The design and construction of the foundations should take into consideration the potential of flooding. All foundation excavations should be free of any loose debris and wet soils, and if groundwater seepage or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered, or possible groundwater seepage during installation of bored piles within the site, it is recommended that consideration be given to other piling methods such as Continuous Flight Auger (CFA) piles.

Shaft adhesion may be applied to socketed piles adopted for foundations provided the socketed shaft lengths conform to appropriate classes of bedrock (subject to confirmation) in accordance with Pells et. al, and shaft sidewall cleanliness and roughness are to acceptable levels. Shaft adhesion should be ignored or reduced within socket lengths that are smeared or fail to satisfy cleanliness requirements (i.e. at least 80%). It is recommended that where piles penetrate expansive soils present within the site, which are susceptible to shrink and swell due to daily and seasonal moisture, shaft adhesion be ignored due to the potential of shrinkage cracking. Pile inspections should be complemented by downhole CCTV camera.

We recommend that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the designed socket materials have been reached and the required bearing capacity has been achieved. The geotechnical engineer should also determine any variations between the boreholes carried out and inspected locations. Inspections should be carried out in dewatered foundations for a more accurate examination, and inspections should be carried out under satisfactory WHS requirements. Geotechnical inspections for verification capacities of the foundations should constitute as a “Hold Point”.

4.7 Filling

Where filling is required, the following recommended compaction targets should be considered:

- Place horizontal loose layers not more than 150mm thickness over the prepared subgrade.
- Compact to a minimum dry density ratio not less than 98% of the maximum dry density for the building platforms.
- The moisture content during compaction should be maintained at ±2% of the Optimal Moisture Content (OMC).
- The upper 150mm of the subgrade should be compacted to a dry density ratio not less than 100% of the maximum dry density.

Any soils which are imported onto the site for the purpose of filling and compaction of the excavated areas should be free of deleterious materials and contamination. The imported soils should also include appropriate validation documentation in accordance with current regulatory authority requirements. The design and construction of earthworks should be carried out in accordance with AS 3798-2007 and AS 1289. Inspections of the prepared subgrade should be carried out by a geotechnical engineer, and should include proof rolling as a minimum. These inspections should be established as “Hold Points”.

4.8 Subgrade Preparation

The following are general recommendations on subgrade preparation for earthworks, slab on ground constructions and pavements:

- Remove existing fill and topsoil, including all materials which are unsuitable from the site.
- Excavate natural soils and rock.
    - Excavated material may be used for engineered fill.
    - Rock may be used for subgrade material underlying pavements.
- Any natural soils (predominately clayey soils) exposed at the bulk excavation level should be treated and have a moisture condition of 2% OMC. This should be followed by proof rolling and compaction of the upper 150mm layer.
    - Any soft or loose areas should be removed and replaced with engineered or approved fill material.

- Any rock exposed at the bulk excavation level should be clear of any deleterious materials (and free of loose or softened materials). As a guideline, remove an additional 150mm from the bulk excavation level.
- Ensure the foundations and excavated areas are free of water prior to concrete pouring.
- Areas which show visible heaving under compaction or proof rolling should be excavated at least 300mm and replaced with engineered or approved fill, and compacted to a minimum dry density ratio not less than 98% of the maximum dry density.

5. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

Following completion of the geotechnical investigation and report, GCA recommends the following additional work to be carried out:

- Dilapidation survey report on adjacent properties and infrastructures.
- Constant supervision and monitoring of any excavation within the proposed development area.
- The depth and strength of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection, predominately in areas not inspected during the geotechnical investigation.
- Geotechnical inspections of foundations (shallow and piles) to confirm the preliminary allowable bearing capacities have been achieved.
- Monitoring of any groundwater inflows during construction within the site.
- Classification of all excavated material transported from the site.
- A meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

6. LIMITATIONS

Geotechnical Consultants Australia Pty Ltd (GCA) has based its geotechnical assessment on available information obtained prior and during the site inspection/investigation. The geotechnical assessment and recommendations provided in this report, along with the surface, subsurface and geotechnical conditions are limited to the inspection and test areas during the site inspection/investigation, and then only to the depths investigated at the time the work was carried out. Subsurface conditions can change abruptly, and may occur after GCA's field testing has been completed.

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site inspection/investigation vary substantially during construction, and from GCA's recommendations and conclusions, GCA should be contacted immediately for further testing and advice. This may be carried out as necessary, and a review of recommendations and conclusions may be provided at additional fees. GCA's advice and accuracy may be limited by undetected variations in ground conditions between sampling locations.

GCA does not accept any liability for any varying site conditions which have not been observed, and were out of the inspection or test areas, or accessible during the time of the investigation. This report and any associated information and documentations have been prepared solely for **Maree Jaloussis & Michael Hayes**, and any misinterpretations or reliances by third parties of this report shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be religated to GCA.

This report should be read in full, including all conclusions and recommendations. Consultation should be made to GCA for any misundertandings or misinterpretations of this report.

For and behalf of

Geotechnical Consultants Australia Pty Ltd (GCA)

Joe Nader
B.E. (Civil – Construction), Dip.Eng.Prac., MIEAust., PEng, AGS, ISSMGE
Cert. IV in Building and Construction
Geotechnical Engineer
Director

7. REFERENCES

Pells P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998.

AS 3600-2009 Concrete Structures. Standards Australia.

AS 1726-2017 Geotechnical Site Investigation. Standards Australia.

AS 3798-2007 Guidelines on Earthworks for Commercial and Residential Developments. Standards Australia.

AS 1289 Methods for Testing Soils for Engineering Purposes. Standards Australia.

AS 2870-2011 Residential Slabs and Footings. Standards Australia.

AS 2159-2009 Piling - Design and Installation. Standards Australia.

NSW WorkCover "Code of Practice – Excavation Work" (July 2015).

NSW Department of Mineral Resources (1983) Sydney 1:100,000 Geological Series Sheet 9130 (Edition 1) Geological Survey of New South Wales. Department of Mineral Resources.

NSW Government Environment and Heritage, Soil and Land Information, Sydney 1:100,000 Soil Landscape Series Sheet 9130la.

NSW Planning Portal.

NSW Six Maps.

eSPADE NSW Environment & Heritage.

APPENDIX A

Important Information About Your
Geotechnical Report

This geotechnical report has been prepared based on the scopes outlined in the project proposal. The works carried out by Geotechnical Consultants Australia Pty Ltd (GCA), have limitations during the site investigation, and may be affected by a number of factors. Please read the geotechnical invesitgation report in conjunction with this "Important Information About Your Geotechnical Report".

Geotechnical Services Are Performed for Specicif Projects, Clients and Purposes.
Due to the fact that each geotechnical investigation is unique and varies from sites, each geotechnical report is unique, and is prepared soley for the client. A geotechnical report may satisfy the needs of structural engineer, where is will not for a civil engineer or construction contractor. No one except the client should rely on the geotechnical report without first conferring with the specific geotechnical consultant who prepared the report. The report is prepared for the contemplated project or original purpose of the investigation. No one should apply this report to any other or similar project.

Reading The Full Report.
Do not read selected elements of the report or tables/figures only. Serious problems have occurred because those relying on the specially prepared geotechnical invesitgation report did not read it all in full context.

The Geotechnical Report is Based on a Unique Set of Project And Specific Factors.
When preparing a geotechnical report, the geotechnical engineering consultant considers a number of unique factors for the specific project. These typially include:

- Clients objectives, goals and risk management preferences;
- The general proposed development or nature of the structure involved (size, location, etc.); and
- Future planned or existing site improvements (parking lots, roads, underground services, etc.);

Care should be taken into identifying the reason of the geotechnical report, where you should not rely on a geotechnical engineering report that was:

- Not prepared for your project;
- Not prepared for the specific site;
- Not prepared for you;
- Does not take into consideration any important changes made to the project; or
- Was carried out prior to any new infrastructure on your subject site.

Typical changes that can affect the reliabiliy if an existing geotechical investigation report include those that affect:

- The function of the proposed structure, where it may change from one basement level to two basement levels, or from a light structure to a heavy loaded structure;
- Location, size, elevation or configuration of the proposed development;
- Changes in the structural design occur; or
- The owner of the proposed development/project has changed.

The geotecnical engineer of the project should always be notified of any changes – even minor – and be asked to evaluate if this has any impact. GCA does not accept responsibility or liability for problems that occur because its report did not consider developments which it was not informed of.

Subsurface Conditions Can Change
This report is based on conditions that existed at the time of the investigation, at the locations of the subsurface tests (i.e. boreholes) carried out during the site investigation. Subfurface conditions can be affected and modified by a number of factores including, but not limited to, the passage of time, man-made influences such as construction on or adjacent to the site, by natural forces such as floods, groundwater fluctuations or earthquakes. GCA should be contacted prior to submitting its report to determine if any further testing may be required. A minor amount of additional testing may prevent any major problems.

Geotechnical Findings Are Professional Opinions
Results of subsurface conditions are limited only to the points where the subsurface tests were carried out, or where samples were collected. The field and laboratory data is analysed and reviewed by a geotechnical engineer, who then applys their professional experience and recommendations about the site's subsurface conditions. Despite investigation, the actual subsurface conditions may differ – in some cases significantly – from the results presented in the geotechnical investigation report, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface anomalies and details.

Therefore, the recommendations in this report can only be used as preliminary. Retaining GCA as your geotechnical consultants on your project to provide construction observations is the most effective method of managing the risks associated with unanticipated subsurface conditions.

Geotechnical Report's Recommendations Are Not Final

Because geotechnical engineers provide recommendations based on experience and judgement, you should not overrely on the recommendations provided – they are not final. Only by observing the actual subsurface conditions revealed during construction may a geotechnical engineer finalise their recommendations. GCA does not assume responsibility or liability for the report's recommendations if no additional observations or testing is carried out.

Geotechnical Report's Are Subject to Misinterpretations

The project geotechnical engineer should consult with appropriate members of the design team following submission of the report. You should review your design teams plans and drawings, in conjunction with the geotechnical report to ensure they have all be incorporated. Due to many issues arising from misinterpretation of geotechnical reports between design teams and building contractors, GCA should participate in pre-construction meetings, and provide adequate construction observations.

Engineering Borehole Logs And Data Should Not be Redrawn

Geotechnical engineers prepare final borehole and testing logs, figure, etc. based on results and interpretation of field logs and laboratory data following the site investigation. The logs, figure, etc. provided in the geotechnical report should never be redrawn or altered for inclusion in any other documents from this report, includined architectural or other design drawings.

Providing The Full Geotechnical Report For Guidance

The project design teams, subcontactors and building contractors should have a copy of the full geotechnical investigation report to help prevent any costly issues. This should be prefaced with a clearly written letter of transmittal. The letter should clearly advise the aforementioned that the report was prepared for proposed development/project requirements, and the report accuracy is limited. The letter should also encourage them to confer with GCA, and/or carry out further testing as may be required. Providing the report to your project team will help share the financial responsibilities stemming from any unanticipated issues or conditions in the site.

Understanding Limitation Provisions

As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part of the geotechnical report, (in most cases) a 'limitations' explanatory provision is included, outlining the geotechnical engineers' limitations for your project – with the geotechnical engineers responsibilites to help other reduce their own. This should be read closely as part of your report.

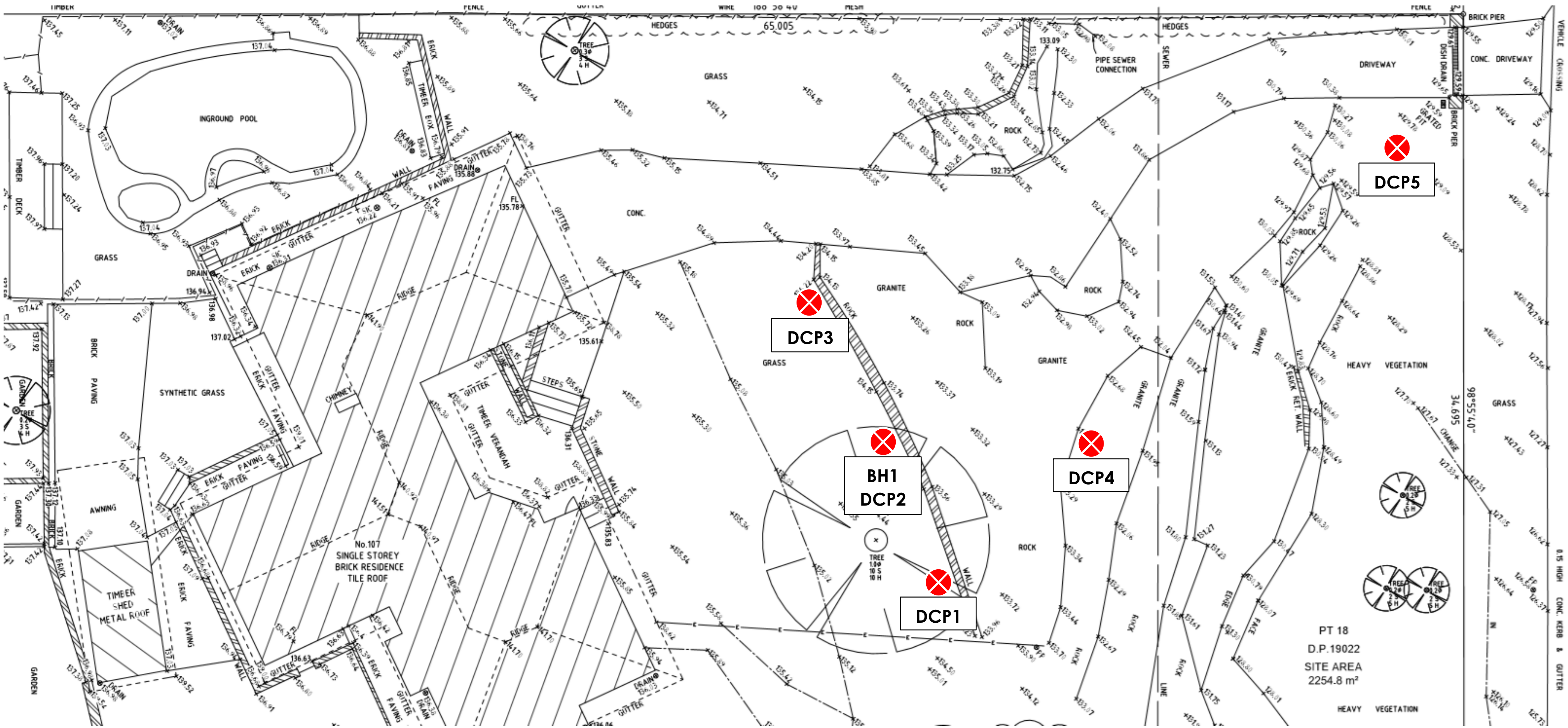
Other Limitations

GCA will not be liable to revise or update the report to take into account any events or circumstances (seen or unforeseen), or any fact occurring or becoming apparent after the date of the report. This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of GCA. The report should not be used if there have been changes to the project, without first consulting with GCA to assess if the report's recommendations are still valid. GCA does not accept any responsibility for problems that occur due to project changes which have not been consulted.

APPENDIX B

Legend:

Approximate Borehole/DCP Testing Location



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GN/GA
		Maree Jaloussis & Michael Hayes	Date: 03/12/2019
	Job No.: G19377-1	107 Iris Street Beacon Hill NSW 2100	Scale: NTS

Image source: Site survey plan prepared by Survcorp Consulting Surveyors Pty Ltd, titled "Plan Showing Physical Features and Levels at No. 107 Iris Street, Beacon Hill Part Lot 18 in DP 19022", referenced No. 3435, and dated 2nd October 2018.

APPENDIX C

Geotechnical Consultants Australia

Explanation of Notes, Abbreviations and Terms Used on Borehole and Test Pit Reports

DRILLING/EXCAVATION METHOD

Method	Description
AS	Auger Screwing
BH	Backhoe
CT	Cable Tool Rig
EE	Existing Excavation/Cutting
EX	Excavator
HA	Hand Auger
HQ	Diamond Core-63mm
JET	Jetting
NMLC	Diamond Core –52mm
NQ	Diamond Core –47mm
PT	Push Tube
RAB	Rotary Air Blast
RB	Rotary Blade
RT	Rotary Tricone Bit
TC	Auger TC Bit
V	Auger V Bit
WB	Washbore
DT	Diatube

PENETRATIION/EXCAVATION RESISTANCE

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

L **Low Resistance.** Rapid penetration possible with little effort from the equipment used.

M **Medium Resistance.** Excavation possible at an acceptable rate with moderate effort required from the equipment used.

H **High Resistance.** Further penetration is possible at a slow rate and required significant effort from the equipment.

R **Refusal or Practical Refusal.** No further progress possible within the risk of damage or excessive wear to the equipment used.

WATER

Water level at date shown

Partial water loss

Water inflow

Complete water loss

Groundwater not observed: The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

Groundwater not encountered: No free-flowing (springs or seepage) was intercepted, although the soil may be moist due to capillary water. Water may be observed in low permeable soils if the test pits/boreholes had been left open for at least 12-24 hours.

MOISTURE CONDITION (AS 1726-1993)

Dry - Cohesive soils are friable or powdery
Cohesionless soil grains are free-running

Moist - Soil feels cool, darkened in colour
Cohesive soils can be moulded
Cohesionless soil grains tend to adhere

Wet - Cohesive soils usually weakened
Free water forms on hands when handling

For cohesive soils the following codes may also be used:

MC>PL Moisture Content greater than the Plastic Limit.
MC~PL Moisture Content near the Plastic Limit.
MC<PL Moisture Content less than the Plastic Limit.

SAMPLING AND TESTING

Sample	Description
B	Bulk Disturbed Sample
DS	Disturbed Sample
Jar	Jar Sample
SPT*	Standard Penetration Test
U50	Undisturbed Sample –50mm
U75	Undisturbed Sample –75mm

*SPT (4, 7, 11 N=18). 4, 7, 11 = Blows per 150mm. N= Blows per 300mm penetration following 150mm sealing.
SPT (30/80mm). Where practical refusal occurs, the blows and penetration for that interval is recorded.

ROCK QUALITY

The fracture spacing is shown where applicable and the Rock Quality Designation (RQD) or Total Core Recovery (TCR) is given where:

$$TCR\ (\%) = \frac{\text{length of core recovered}}{\text{length of core run}}$$

$$RQD\ (\%) = \frac{\text{Sum of Axial lengths of core > 100mm long}}{\text{length of core run}}$$

ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

GCA

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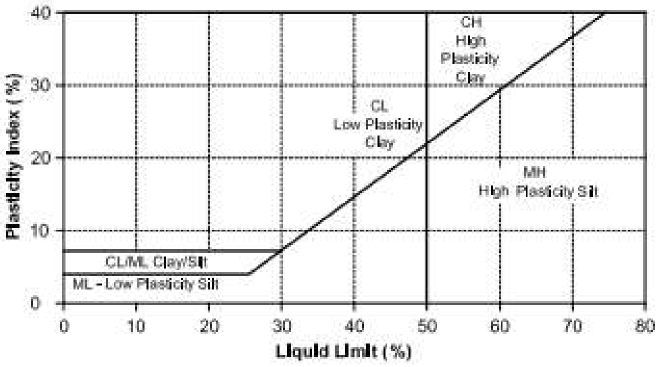
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

Soil and Rock is classified and described in reports of boreholes and test pits using the preferred method given in AS 1726-1993, Appendix A. The material properties are assessed in the field by visual/tactile methods. The appropriate symbols in the Unified Soil Classification are selected on the result of visual examination, field tests and available laboratory tests, such as, sieve analysis, liquid limit and plasticity index.

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

Name	Subdivision	Size
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 μm to 2.36 mm
	medium	200 μm to 600 μm
	fine	75 μm to 200 μm

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-1993)

Strength	Symbol	Undrained Shear Strength, C_u (kPa)
Very Soft	VS	< 12
Soft	S	12 to 25
Firm	F	25 to 50
Stiff	St	50 to 100
Very Stiff	VSt	100 to 200
Hard	H	> 200

PLASTICITY

Description of Plasticity	LL (%)
Low	<35
Medium	35 to 50
High	>50

COHESIONLESS SOILS - RELATIVE DENSITY

Term	Symbol	Density Index	N Value (blows/0.3 m)
Very Loose	VL	0 to 15	0 to 4
Loose	L	15 to 35	4 to 10
Medium Dense	MD	35 to 65	10 to 30
Dense	D	65 to 85	30 to 50
Very Dense	VD	>85	>50

UNIFIED SOIL CLASSIFICATION

USC Symbol	Description
GW	Well graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
ML	Silt of low plasticity
CL	Clay of low plasticity
OL	Organic soil of low plasticity
MH	Silt of high plasticity
CH	Clay of high plasticity
OH	Organic soil of high plasticity
Pt	Peaty Soil

ROCK MATERIAL WEATHERING

Symbol	Term	Definition
RS	Residual Soil	Soil definition on extremely weathered rock; the mass structure and substance are no longer evident; there is a large change in volume but the soil has not been significantly transported
EW	Extremely Weathered	Rock is weathered to such an extent that it has 'soil' properties, i.e. It either disintegrates or can be remoulded in water
HW / DW	Highly Weathered *Distinctly Weathered (as per AS 1726)*	The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable.
MW	Moderately Weathered	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable
SW	Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR	Fresh	Rock shows no sign of decomposition or staining

ROCK STRENGTH (AS 1726-1993 and ISRM)

Term	Symbol	Point Load Index $Is_{(50)}$ (MPa)
Extremely Low	EL	<0.03
Very Low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1
High	H	1 to 3
Very High	VH	3 to 10
Extremely High	EH	>10

ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

Term	Defect Spacing	Bedding
Extremely closely spaced	<6 mm	Thinly Laminated
	6 to 20 mm	Laminated
Very closely spaced	20 to 60 mm	Very Thin
Closely spaced	0.06 to 0.2 m	Thin
Moderately widely spaced	0.2 to 0.6 m	Medium
Widely spaced	0.6 to 2 m	Thick
Very widely spaced	>2 m	Very Thick

Type	Definition
B	Bedding
J	Joint
HJ	Horizontal to Sub-Horizontal Joint
F	Fault
Cle	Cleavage
SZ	Shear Zone
FZ	Fractured Zone
CZ	Crushed Zone
MB	Mechanical Break
HB	Handling Break

Planarity	Roughness
P – Planar	C – Clean
Ir – Irregular	Cl – Clay
St – Stepped	VR – Very Rough
U - Undulating	R – Rough
	S – Smooth
	Sl – Slickensides
	Po – Polished
	Fe – Iron

Coating or Infill	Description
Clean (C)	No visible coating or infilling
Stain	No visible coating or infilling but surfaces are discoloured by mineral staining
Veneer	A visible coating or infilling of soil or mineral substance but usually unable to be measured (<1mm). If discontinuous over the plane, patchy veneer
Coating	A visible coating or infilling of soil or mineral substance, >1mm thick. Describe composition and thickness
Iron (Fe)	Iron Staining or Infill.

APPENDIX D

Geotechnical Consultants Australia Pty Ltd
info@geoconsultants.com.au
www.geoconsultants.com.au

BOREHOLE NUMBER BH1

CLIENT Maree Jaloussis & Michael Hayes
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G19377-1
PROJECT LOCATION 107 Iris Street Beacon Hill NSW 2100

DATE STARTED 29/11/19 **COMPLETED** 29/11/19 **R.L. SURFACE** 134.1 **DATUM** m AHD
DRILLING CONTRACTOR Geotechnical Consultants Australia Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Hand Operated Equipment **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
HA	Not Encountered During Augering	134.0				Silty SAND, fine grained sand, brown to dark brown, grey, some fine to medium grained gravel, grass rootlets, moist.		FILL
						SANDSTONE, fine grained, grey to pale grey, brown, extremely weathered, extremely low estimated strength, moist.		BEDROCK
		133.5	0.5			Practical hand auger refusal at 0.4m bgl. Borehole BH1 terminated at 0.4m		
			1.0					

APPENDIX E

DYNAMIC CONE PENETOMETER RESULTS

Client:	Maree Jaloussis & Michael Hayes	**Test Date:**	28/11/2019
Address:	107 Iris Street Beacon Hill NSW 2100	**Job No.:**	G19377-1

Depths (mm bgl)	**DCP No.**				**Depths (mm bgl)**	**DCP No.**			
	1	**2**	**3**	**4**		**5**			
0-100	1	1	1	1	0-100	1			
100-200	2	2	2	2	100-200	1			
200-300	1/50mm	1	1	2	200-300	**Bouncing**			
300-400	**Bouncing**	1/50mm	1/50mm	1	300-400				
400-500		**Bouncing**	**Bouncing**	2	400-500				
500-600				1	500-600				
600-700				1/50mm	600-700				
700-800				**Bouncing**	700-800				
800-900					800-900				
900-1000					900-1000				
1000-1100					1000-1100				
1100-1200					1100-1200				
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3700-3800					3700-3800				
3800-3900					3800-3900				
3900-4000					3900-4000				

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Tested:	**GN**		**Sheet:**	**1 of 1**

APPENDIX F

Foundation Maintenance and Footing Performance: A Homeowner's Guide

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).

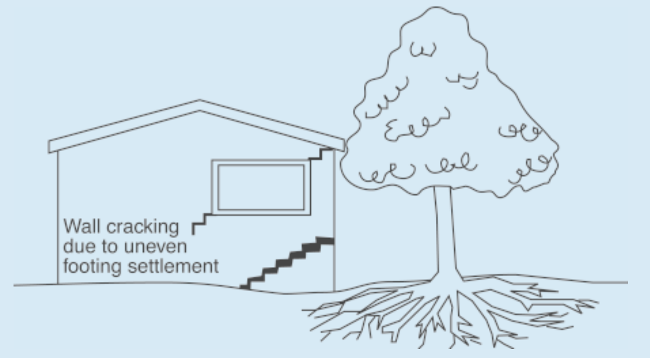
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

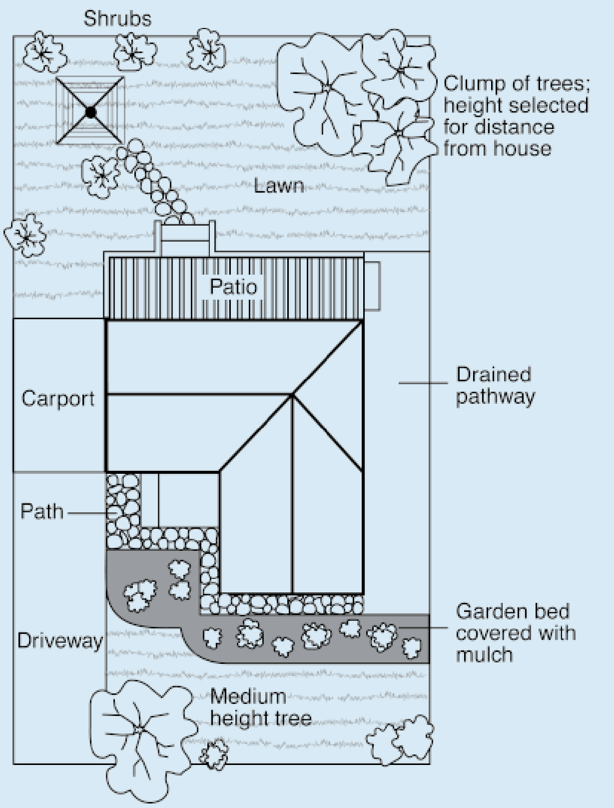
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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.

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