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Report on
Geotechnical Investigation

John Palmer Public School Upgrade
85 The Ponds Boulevard, The Ponds

Prepared for
NSW Department of Education

Project 94624.01
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Report on Geotechnical Investigation
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1. Introduction

This geotechnical report accompanies an Environmental Impact Statement (EIS) pursuant to Part 4 of the Environmental Planning and Assessment Act 1979 (EP&A Act) in support of a State Significant Development Application (SSD - 23330227).

The development is for upgrading works comprising alterations and additions to John Palmer Public School at 85 The Ponds Boulevard, The Ponds. The site is legally described as Lot 1 DP 1131340.

The site is roughly rectangular in shape, with a total area of 29,830m^2 and street frontages to Pebble Crescent to the west, Jetty Street to the south and The Ponds Boulevard to the east. The Ponds Shopping Centre adjoins the northern property boundary of the school.

This report addresses the relevant Secretary's Environmental Assessment Requirements (SEARs), specifically the assessment of salinity and a salinity management plan.

The investigation included the drilling of boreholes and laboratory testing of selected samples. The details of the field work and laboratory testing are presented in this report, together with comments and recommendations for design and construction.

DP carried out a detailed site investigation (DSI) for contamination in conjunction with this geotechnical investigation, the results of which are presented in a separate report (DP Report Reference 94624.01.R.002.Rev1).

2. Previous Investigations

DP has carried out the following previous investigations on-site:

- Geotechnical Investigation for "John Palmer Public School Upgrade: 85 The Ponds Boulevard, The Ponds" dated 8 September 2020 (Project Reference 94624.00).
- Detailed Site Investigation for Contamination for "John Palmer Public School Upgrade: 85 The Ponds Boulevard, The Ponds" dated 8 September 2020 (Project Reference 94624.00).
- Preliminary Contamination Assessment for "Proposed Primary School: The Ponds Boulevard, Second Ponds Creek" dated 30 April 2007 (Project Reference 44777).
- Geotechnical Investigation for "Proposed Primary School: The Ponds Boulevard, Second Ponds Creek" dated 2 March 2007 (Project Reference 41177A).

The relevant boreholes from these investigations have been indicated on Drawing 1, Appendix B and the borehole logs included in Appendix C.

3. Site Description

The proposed new three-storey buildings, extensions to existing buildings and new carpark are to be located over the northern and eastern portions of John Palmer Public School ("the site") located at 85 The Ponds Boulevard, The Ponds. The two new three-storey building and carpark comprise irregular shaped areas of approximately 2700 m^2, the extensions to existing building Block B comprise a total area of approximately 350 m^2, a new library an area of approximately 600 m^2.

The proposed new three-storey building, landscaping and pavements are to be located in the north-eastern corner of John Palmer Public School ("the site") located at 85 The Ponds Boulevard, The Ponds. The site covers an irregular shaped area of approximately 0.5 hectares.

At the time of the field work the site was occupied by landscaped areas, open grassed areas and concrete pavements. Some mature trees were observed on-site up to an estimated height of 20 m.

The site is bounded by the following structures;

- North - The Ponds Shopping Centre to the north. The Shopping Centre includes a two-level basement.
- East - The Ponds Boulevard
- South and west - existing school buildings to the south and an open grassed area and shade structure.

The site topography generally slopes down to the north-west at gradients estimated to be less than 3° with the maximum elevation at about RL 59.7 (m AHD) in the south-east corner and the minimum elevation at about RL 58.1 (m AHD) in the north-west corner. Second Ponds Creek is located about 270 m west of the site.

Table 1 presents site identification details and a location plan showing the approximate site area is presented in Figure 1.

Table 1: Site Identification

Item	Details
Allotment Identification	Lot 1 DP 1131340
Street Address	85 The Ponds Boulevard
Locality	The Ponds, NSW
Site Area	Full school site 3.0 hectares (approximately) Development area 0.2 hectare (approximately)
Local Government Area	Blacktown City Council (BCC)
Zoning	SP2 Educational Establishment

Planning Instrument(s)	BCC DCP (2015); LEP (2015)
Current Landuse	Primary School
Current Owner	Minister for Education and Training

Figure 1: Site Location Plan (Source: Metro Map)

4. Published Data

4.1 Geology

Reference to the Penrith 1:100 000 scale Geological Series Sheet indicates that the site is underlain by Ashfield Shale of Triassic Age. Ashfield Shale typically comprises dark grey to black shale, siltstone and laminite which weathers to a residual clay profile of medium to high plasticity.

4.2 Hydrogeology

The closest surface water receptor to the site is Second Ponds Creek located about 270 m west of the site.

Based on the local topography, groundwater is anticipated to flow to the west towards Seconds Ponds Creek.

A search of the NSW Department of Primary Industries Water (DPI Water) online map of registered groundwater works was undertaken as part of the investigation. The search carried out on 30 June 2021 identified one registered groundwater borehole within 500 m of the site that contained groundwater information. A standing groundwater level of 6.5 m was measured in the groundwater borehole located about 420 m south-west of the site.

4.3 Soil Landscape

Reference to the Penrith 1: 100 000 scale Soil Landscape Series Sheet indicates that the site is located within the Blacktown soil landscape group. The Blacktown Group is characterised by moderately reactive, highly plastic subsoil with poor drainage characteristics.

4.4 Acid Sulfate Soils

Review of published mapping indicates that the site is in an area of 'no known occurrence of acid sulfate soils'. The NSW Acid Sulfate Soils Manual 1998 published by the Acid Sulfate Soils Management Advisory Committee (ASSMAC) indicates that ASS (and Potential Acid Sulfate Soils – PASS) normally occur in alluvial or estuarine soils below RL 5 m AHD although occasionally are encountered up to RL 12 m AHD. Considering the ASS mapping and given that the site soils are at site elevations above RL 50 m AHD, it is considered unlikely that ASS is present on-site.

4.5 Salinity

The Department of Infrastructure, Planning and Natural Resources (DIPNR) "Map of Salinity Potential in Western Sydney 2002" suggests that the site is in an area of "moderate salinity potential" with a higher potential in the lower elevations areas in close proximity to the Second Ponds Creek system.

5. Field Work

5.1 Methods

The field work included the following:

- Drilling of twenty two boreholes (Bores 9 to 110, 115 to 19, 122 to 124 and 201 to 205) using a track mounted drilling rig with 110 mm diameter spiral flight augers. The boreholes were drilled to depths of between 1.0 m and 5.6 m. Standard penetration tests (SPTs) were completed at regular depths within the overburden.
- Extension of four boreholes (Bores 101, 102, 201 and 202) into the underlying bedrock using NMLC coring techniques to obtain continuous 50 mm diameter core samples of the rock for identification and strength testing purposes. The boreholes were drilled to depths of 5.6 m to 11.56 m.
- Drilling of ten boreholes (Bores 111 to 114, 120, 121, 203, 206, 207 and 208) using a 3.5-tonne excavator fitted with a 150 mm diameter auger attachment. The boreholes were drilled to depths of between 0.3 m and 2.9 m.
- Dynamic Cone Penetrometer (DCP) tests adjacent to selected boreholes. DCPs were carried out to 1.2 m depth or prior refusal in accordance with AS1289.6.3.2.
- Drilling of two test bores (Bores 209 and 210), initially using a concrete corer and then a hand auger to depths of 0.3 m to 0.4 m;

Undisturbed and disturbed samples were collected from the boreholes to assist with logging and for laboratory testing.

The ground surface levels (measured in 'metres above Australian Height Datum AHD') together with the Eastings and Northings at the borehole locations were determined by using a high precision Differential GPS which is accurate to approximately 0.1 m. The locations of the boreholes are shown on Drawing 1 in Appendix B.

5.2 Results

The borehole logs from the investigation are provided in Appendix D. Notes defining classification methods and terms used to describe the soils and rocks are included in Appendix A. The subsurface conditions encountered underlying the site of the can be summarised as follows:

- Topsoil:
  - silty clay, clayey silt or sandy silt topsoil to depths of 0.08 m to 0.2 m in all boreholes except Bore 206;
- Fill:
  - asphaltic concrete ranging from 20 mm to 40 mm thick at Bores 101 and 110 overlying roadbase gravel to 0.4 m depth;
  - silty clay topsoil fill with gravel, sand and vegetation to depths of 50 mm to 150 mm in all boreholes except Bores 101, 110 and 112;
  - silty clay or gravelly clay fill to depths of 0.2 m to .3 m in all boreholes except Bores 108, 123 and 124 where the fill was from 2.0 m to 2.2 m. Inclusions of sand, silt and gravel were encountered within the fill;

- Natural Clay: - typically stiff to very stiff silty clay in all boreholes to depths of 1.0 m and 5.5 m;
- Weathered Bedrock: - very low strength siltstone at a depth of 2.7 m to 3.0 m in Bores 201, 202 and 207;
- Low and Medium Strength Siltstone - low and medium strength, slightly weathered, slightly fractured siltstone at depths of 2.40 m to 5.6 m;
- High and Very High Strength Siltstone - high strength, slightly weathered to fresh, slightly fractured siltstone was encountered below depths of 4.3 m and 6.2 m. Very high strength, fresh, slightly fractured siltstone was encountered below a depth of 11.0 m in Bore 202.

No free groundwater was observed during augering and the use of water as a drilling fluid prevented groundwater observations during coring. Backfilling of the boreholes at the completion of drilling precluded long term monitoring of the groundwater levels. It is noted, however, that groundwater levels are affected by preceding climatic conditions and soil/rock permeability and can therefore fluctuate with time.

6. Laboratory Testing

6.1 Mechanical Testing

Selected samples from the boreholes were tested in the laboratory for measurement of plasticity and linear shrinkage. The detailed results are given in Appendix E and summarised in Table 2.

Table 2: Results of Laboratory Testing – Geotechnical

Bore	Material	Depth (m)	FMC (%)	OMC (%)	MDD (t/m^3)	CBR (%)	W_L (%)	W_P (%)	PI (%)	I_{ss} (%/ Δpf)	ECN
103	Silty Clay	0.9 – 1.0	31.3	-	-	-	79	35	44	-	-
117	Silty Clay Fill	0.6 – 0.9	-	-	-	-	-	-	-	3.2	-
124	Silty Clay Fill	0.3 – 1.0	16.4	14.5	1.85	7	-	-	-	-	2
201	Silty Clay	0.5 – 0.75					36	16	20		
201	Silty Clay	1.0 - .45					33	16	17		
202	Silty Clay	1.0 – 1.45					36	16	20		

Notes: FMC = Field Moisture Content; OMC = Standard Optimum Moisture Content; MDD = Maximum Dry Density; CBR = California bearing ratio; W_L = Liquid Limit; W_P = Plastic Limit; PI = Plasticity Index; I_{SS} = Shrink Swell Index; ECN = Emerson Crumb number

The results of the laboratory testing indicate the following:

- The Atterberg Limit test results indicates that the silty clay samples were of medium high plasticity.
- The shrink-swell result indicates the silty clays are typically highly reactive and therefore susceptible to shrink and swell movements due to changes in soil moisture content.
- The field moisture content was almost 2 % wet of the standard optimum moistures content.
- A CBR value of 7 % was obtained for the silty clay fill sample tested.
- The field moisture content was 16.4 % and 31.3 % for the silty clay fill and natural silty clay samples, respectively. The field moisture content of the natural silty clays sample was approximately 4 % dry of the plastic limit.
- The Emerson Crumb Number was 2 indicating the silty clay fill sample was highly dispersive.

6.2 Chemical Testing

Selected samples collected from the boreholes were also tested in the laboratory for determination of aggressivity to concrete and steel, sodicity, textural classification and salinity.

A result summary table (Appendix E) presents the results of laboratory tests, assessments of aggressivity to concrete and steel, sodicity class, textural classification, calculated salinity electrical conductivity (ECe) and salinity class inferred from ECe values using the method of Richards (1954). The detailed laboratory test reports and chain of custody information are also provided in Appendix E.

The total test sample numbers and the range of test results obtained are summarised in Table 3.

Table 3: Results of Laboratory Testing - Chemical

Parameter		Units	Number of Tests	Range of Results
pH		pH units	22	5.6 – 9.3
Chlorides		(mg/kg)	2	<10 – 400
Sulphates		(mg/kg)	2	41 – 310
Aggressivity [AS 2159]	to Concrete	-		non-aggressive to mildly aggressive
	to Steel	-		non-aggressive
Exchangeable Sodium (Na)		(meq/100g)	3	0.13 – 1.6
CEC (cation exchange capacity)		(meq/100g)	3	6.2 - 27
Sodicity [Na/CEC]		(ESP%)	3	<1 - 22
Sodicity Class		[after DLWC]	3	Non - Sodic to Highly Sodic
EC1:5 [Lab.]		(µS/cm)	12	97 - 630
ECe [M x EC1:5] [1]		(dS/m)	12	<2 – 5.7
Salinity Class [after Richards]		-	12	Non-Saline to Moderately Saline

Notes: 1 M is soil textural factor

6.2.1 Aggressivity

Test results showing the aggressivity assessed by pH, sulphate concentrations and chloride concentration criteria (of AS 2159) at the borehole locations, together with the aggressivity class ranges indicated in Australian Standard AS 2159 are given in Appendix E. The absence of free groundwater in the boreholes and the inferred very low permeability of the sampled clay-rich soils indicate that soils at all boreholes are in Condition “B” as defined by AS 2159.

The results show that the samples tested indicate the ground conditions are non-aggressive to concrete and steel with reference to AS2159. The pH profiles with depth are shown in Figure 2 (following page).

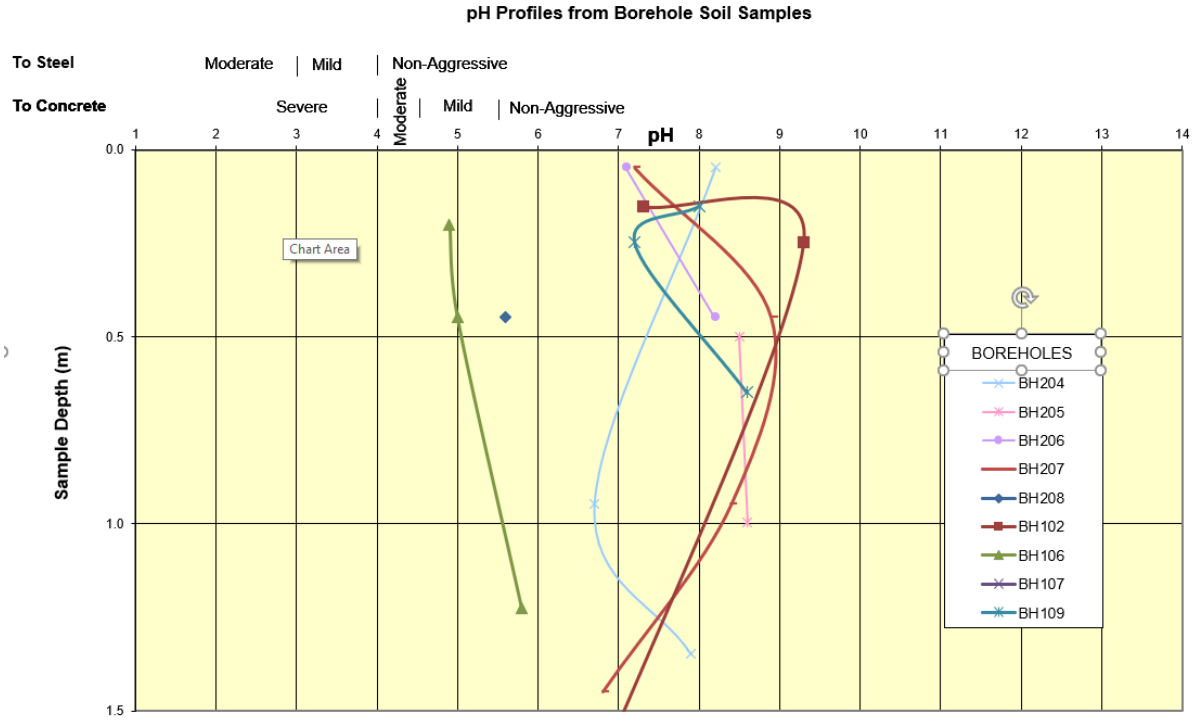

Figure 2: Vertical pH Profiles

6.2.2 Salinity

Figure 3 (following page) shows the salinity classifications based on the electrical conductivity (ECe) at borehole locations, together with the salinity classifications of Richards (1954). Test results are provided in Appendix E.

The results indicate that the samples tested were varied ranging from non-saline to moderately saline.

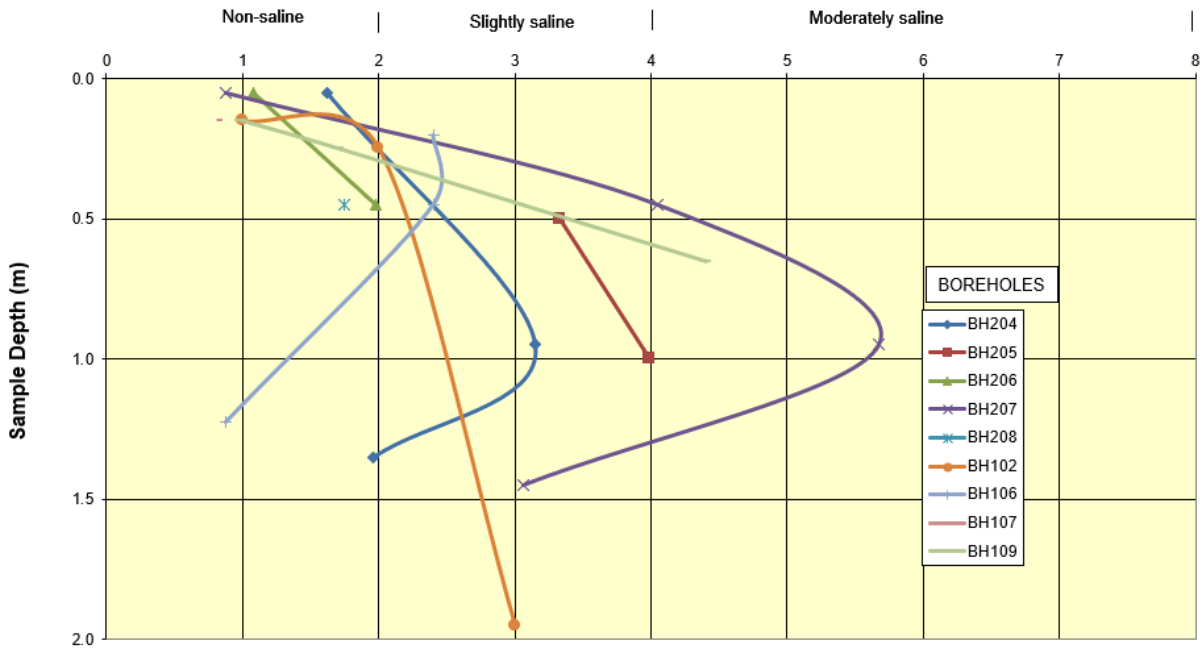

Figure 3: Vertical Salinity Profiles and Salinity Classes

6.2.3 Sodicity

The sodicity test results (refer Appendix E) indicates non - sodic to highly sodic soils, indicating a high potential for erosion of soils left exposed.

6.2.4 Rock Samples

Point Load Strength Index (Is_{50}) testing was carried out on selected rock core specimens. The results of the tests are shown on the borehole logs at the appropriate depths. Figure 4 (following page) shows the range of Is_{50} results at the various depths (shown as Reduced Levels relative to AHD).

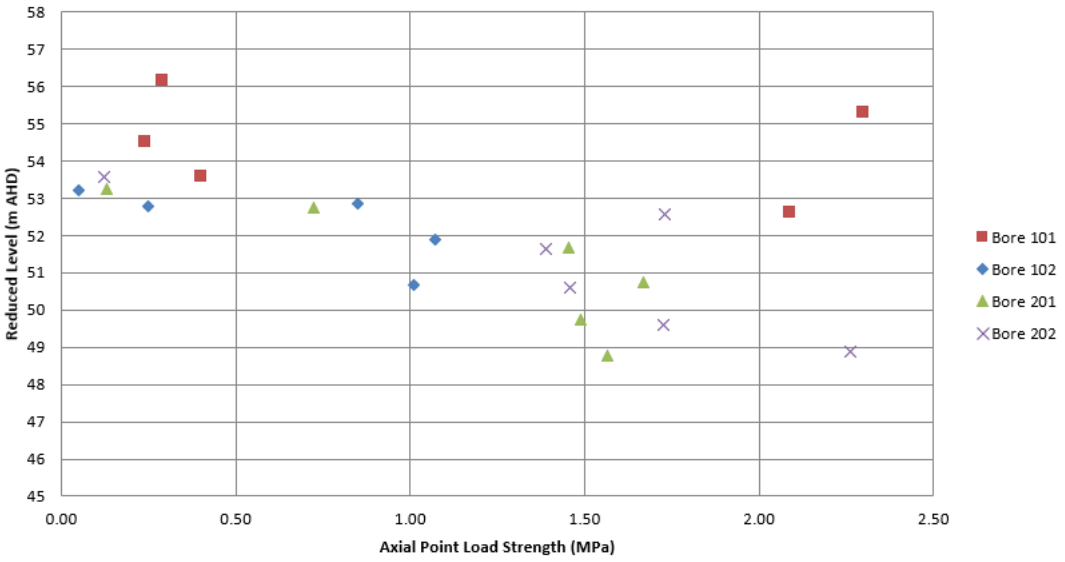

Figure 4: Results of Axial Point Load Tests

7. Proposed Development

The proposed development seeks to upgrade John Palmer Public School. The upgrade consists of the following alterations and additions:

- Construction of a new three storey building facing The Ponds Boulevarde which will accommodate 29 Permanent Learning Spaces and 1 new staff room;
- Construction of a one storey new library building;
- Relocation of service access to staff car park off The Ponds Boulevarde, including alterations to the existing car park to accommodate service vehicle;
- One-storey extension to and refurbishment of existing School Hall building. The School Hall extension will accommodate ancillary spaces for Out of Hours School Care;
- Building Block D will be re-purposed from an existing library to special program spaces and administration;
- Refurbishment of Building F to provide 1 new support unit;
- Minor additions and internal refurbishments to Building A;
- Removal of all 20 existing demountable classroom buildings once alterations and additions have been completed; and
- Ancillary works to support the alterations and additions including landscaping and service provision.

8. Comments

8.1 Geotechnical Model

The site geology in each area of site can be described as follows;

- Proposed Buildings in southern side of the site – fill to depths of 0.2 m to 0.9 m overly stiff to very stiff natural clays, with weathered bedrock at depths of 0.8 m to 2.0 m. The bedrock comprises siltstone, sandstone and laminite (interbedded siltstone and sandstone) and typically increases in strength from very low to high strength with increasing depth. The fill material on-site appears uncontrolled. The clays on site are typically reactive and of high plasticity.
- Extensions to Block B and Library – fill to depths of 0.1 m to 0.2 m overlying stiff to very stiff natural clays with very low strength siltstone at depths of 0.5 m to 1.0 m on the southern side of the building and 2.0 m depth on the northern side of the building.
- New Building in north-west corner of the site - The site is underlain generally by up to about 1.3 m of topsoil or fill overlying stiff to very stiff natural clays with siltstone bedrock at depths of between 2.0 m and 3.0 m. The fill material on-site appears to be uncontrolled. The clays on site are typically reactive and of medium plasticity.

The laboratory test results of this investigation, together with previous laboratory testing of soil samples on-site, indicate CBR values in the range of 5 % to 8 % and shrink-swell values in the range of 1.7% to 3.0%.

Groundwater was not encountered during the investigation. Notwithstanding this, water seepage is expected to flow over the soil/rock interface and along bedding planes and joints within the rock. Groundwater levels are expected to fluctuate with variations in climatic conditions.

Results from previous laboratory testing of soil samples obtained from the site, indicate CBR values in the range of 5 % to 7 %.

Groundwater seepage is expected at the soil/rock interface and along bedding planes and joints within the rock. Groundwater levels could change with variations in climatic conditions. Based on the local topography, groundwater is anticipated to flow downslope to the west towards Second Ponds Creek.

8.2 Preliminary Site Classification

The results of field work indicate that the site is underlain by topsoil or fill (up to about 1.3 m depth) overlying residual clay soils then weathered rock. Furthermore, the development area is located in close proximity to trees. Where mature trees are located near the proposed building footprint, a “P” classification would be assigned to the site in accordance with the abnormal moisture provisions and uncontrolled fill greater than 0.4 m thick of AS2870 – 2011 “Residential Slabs and Footings”. Class P sites require design from engineering principles.

The laboratory testing indicates that the clays at the site are of high plasticity and therefore likely to be susceptible to shrink-swell movements in response to seasonal variations in soil moisture content. Based on the soil depth, and the results of laboratory testing, it is considered that the natural soil profile would generally be consistent with a Class “H2” site as per AS 2870 with the exception of the extension

to the south of Block B which would be consistent with a Class “M” given the shallower rock profile. AS2870 indicates that characteristic surface movements (y_s) of up 75 mm are expected for a Class “H2” site and 40 mm for a Class “M” site.

For high-level footings, the effect of adjacent trees needs to be considered in the design. Using the methods of AS2870, an additional 10 mm to 15 mm of movement should be allowed for within the design of the footings where trees up to 15 m in height are within 5 m of the proposed structure.

8.3 Site Preparation and Earthworks

8.3.1 Excavation Conditions

Excavation to depths of up to 1.0 m is generally expected to encounter topsoil, fill and natural soils. Excavation within the soils and any very low strength rock should be achievable using conventional earthmoving equipment such as hydraulic excavators.

Vibration generated during earthworks operations would generally be at a level that would not adversely affect the neighbouring structures.

The proposed excavations on site are expected to be shallow in which case the guidelines as given in SafeWork would be appropriate. If batters are required for shallow excavations, a batter of 1H:1V could be adopted for natural clay or clay filling.

All excavated materials disposed of off-site will need to be classified in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014). This includes topsoil, fill and natural materials that may be removed from the site.

8.3.2 Site Preparation

For planning purposes, the following site preparation measures are recommended for subgrade preparation and any site platform filling placement for the development:

- remove any deleterious, soft, wet or highly compressible material or material rich in organics or root matter (such as topsoils). Based on DP’s investigation this will typically range from 0.1 m to 0.2 m depth. Topsoil materials should be separately stockpiled for use in landscaping or removed off site. Fill materials could potentially be reused on-site subject to geotechnical inspection and approval;
- roll the exposed surface with at least six passes of a minimum 12 tonne deadweight smooth drum roller, with a final test roll pass accompanied by careful visual inspection to ensure that any deleterious materials such as soft, wet or highly compressible soil and any organics are identified and removed;
- place approved filling, where required, in layers not exceeding 250 mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% Standard within 2% of optimum moisture content (OMC). New fill to form pavement subgrades will be required to be compacted to a

minimum dry density ratio of 100% relative to Standard compaction. New filling should be free of oversize particles (>75mm) and deleterious material;

- moisture conditioning of clay soils may be required if soils are saturated or dry. The soils on-site were observed to be 2% wet of optimum moisture content and would generally require drying. Moisture conditioning of saturated soils would involve drying in 'sunny and windy' weather, blending with other drier materials or lime stabilisation. Where the soil is dry, it is expected that this will involve either tyning or excavation with the addition of water to increase the moisture content;
- promptly cover any exposed clay at subgrade level with a minimum 150 mm of select granular fill (minimum CBR 15%) to reduce potential wetting and drying and trafficability problems; and
- new filling required to achieve design levels for support of any on-ground slabs and/or structural loads will need to be carried out under Level 1 testing conditions as defined in AS 3798–2007 "Guidelines on Earthworks for Commercial and Residential Developments". Level 2 testing is recommended for filling materials beneath pavements, recreational and landscaping areas.

The above procedures will require geotechnical inspection and testing services during construction.

8.4 Excavation Support

8.4.1 Batter Slopes

Excavation for the proposed building is likely to be predominantly within fill and clays.

A maximum batter slope of 3H:1V is recommended for permanent slopes in the clays to allow for maintenance, provided that the slopes are protected against surface erosion and local slumping. The batter slopes recommended above are appropriate provided there are no surcharge loads from structures near the top of the batters.

All battered slopes will need to have appropriate drainage measures installed to limit the potential for slopes to be adversely affected by water runoff.

8.5 Foundations

All structural loads will need to be supported on a uniform founding layer being either natural clays/controlled engineering fill, or siltstone bedrock.

Shallow footings (e.g. pad or strip footings) founded on controlled fill or stiff natural clays prepared in accordance with Section 8.3.2 could be designed for an allowable bearing pressure of 150 kPa. The design of shallow footings should also take account of shrink-swell movements associated with the site classification outlined in Section 8.2.

If higher structural loads are proposed, or resultant settlements are beyond tolerable limits, then a deep-footing system, probably founding on weathered rock, would be required. Bored piles are considered a feasible pile type although temporary or permanent liners (i.e. casing) may be required to manage issues associated with possible water seepage. Alternatively, piles may be constructed using continuous flight auger (CFA) piles.

Recommended design parameters for footings founding on rock are presented in Table 4.

Table 4: Maximum Foundation Design Parameters

Unit	Working Stress Design Values		Limit State Design Values		Elastic Modulus (MPa)
	Allowable End Bearing Pressure (kPa)	Shaft Adhesion (kPa)	Ultimate End Bearing Pressure (kPa)	Shaft Adhesion (kPa)	
Very Low to Low Strength Bedrock	1000	100	3000	300	100
Low and Medium Strength Bedrock	2000	200	18,000	450	800
High Strength Siltstone	3500	350	30,000	600	1200

Note: Higher design parameters could potentially be assigned with additional investigation

A geotechnical strength reduction factor (ϕ_g) should be applied to the ultimate values provided in Table 4 if the limit-state design process is undertaken to design the piles. Australian Standard AS 2159:2009 "Piling – Design and Installation" (2009) provides information on how to determine an appropriate value of ϕ_g which is based on a risk assessment. The pile designer will need to confirm a ϕ_g value when the piling contractor is selected, however it is suggested that a preliminary value of 0.5 be adopted at this stage.

The total (long-term) settlement of a piled footing designed using the allowable parameters provided in this report should be less than about 1% of the pile diameter upon application of the design dead load. Serviceability analysis should be undertaken if the ultimate bearing pressures (incorporating a suitable reduction factor) are used to proportion the piles.

Over the designated 'socket length', the sidewalls of bored piles should be clean and free of clay 'smear'. Also, the sidewalls should meet the minimum roughness category of "R2" (defined as grooves of 1 to 4 mm depth and width greater than 2 mm, at a spacing of 50 mm to 200 mm) in Pells et.al (1998). A 'grooving' or 'roughening' tool may be required to achieve this criterion.

The foundation parameters provided in this report assume all footings are free of water and loose debris immediately prior to pouring concrete. All foundations should be constructed below the zone of influence of any existing or proposed service trenches. The zone of influence can be conservatively defined by a plane extending upwards at 45° from the base of the service trench. Review of the structural drawings from the adjacent shopping centre suggests that the proposed structure would generally be outside the zone of influence of the shopping centre basement structure.

All footings in one structure should be founded on the same strata to achieve uniform founding conditions and limit the potential for differential movement between different parts of the structure.

It is recommended that all footing excavations be inspected by an experienced geotechnical engineer or engineering geologist prior to the placement of concrete and steel to confirm the design bearing pressure.

8.6 Floor Slabs

Where buildings are to be designed with a suspended floor slab, site preparation measures will be minimal. If slabs are to be cast on ground (but designed as suspended slabs), then checks should be made to ensure that concrete is not poured onto softened or wet ground that could lead to deformation of the slab. Furthermore, in areas where clay is present, to reduce the potential for swelling of soils beneath the slab, the top 100 mm of the ground surface should be scarified and loosed prior to forming up for the slab. Alternatively, void formers could be used if the risk of potential swell beneath the slab is unacceptable.

Where site preparation is undertaken in accordance with Section 8.3.2, on-grade slabs could be constructed in place of suspended slabs. Based on the results of the subsurface investigations, subgrade conditions are expected to be formed over natural clay and/or clay filling.

Floor slabs should be cast independently of pads or pile and beam footings and incorporate control joints to allow for differential movements. Edge protection, such as deepened stiffening edge beams in conjunction with surface paving should also be included to minimise the effects of reactivity movements due to the moderate reactivity of the site clays.

8.7 Seismic Design

In accordance with Part 4 of the Structural design actions Standard, AS1170.4 – 2007, the site is assessed to have a Site Sub-Soil Class of "C_e". This is in accordance with the definitions presented in Section 4.2 – Class Definitions of the Standard.

8.8 Pavements

Based on the moderately reactive clays on-site, concrete pavements and ground slabs should be articulated to allow for differential movement together with a drainage system to limit the potential for shrink-swell movements that could potentially damage pavements.

The CBR values obtained in previous investigations is in the range of 5 % to 7 %. Based on experience in the area and allowing for variability of results, it is suggested that the design of pavements be based on a design CBR value of 3 %. If imported material is used to level the site and form subgrade levels, the design CBR value will depend on the type and depth of imported material. Pavements should be placed on a subgrade prepared in accordance with the recommendations provided in Section 8.3.2.

The design CBR value given above depends on the provision of adequate surface and subsoil drainage to maintain the subgrade as close to OMC as possible. Subsoil drainage should be installed to not less than 500 mm depth below subgrade level adjacent to the pavement. Preparation of subgrade surfaces

should be such that adequate cross-falls for the surface drainage purposes are achievable across the final pavement.

8.9 Site Maintenance and Drainage

Surface and subsurface drainage for the building should be incorporated into the design.

Care should be taken to avoid external influences on the soil moisture-regime to prevent erosion and softening of the exposed soils. Detailing of surface and subsurface drainage should be aimed at avoiding substantial wetting of the soils beneath building and pavement areas. Surface water should be directed away from building or hardstand areas and the upper section of services trenches should be backfilled with compacted clay soil to avoid the trench acting as an inlet drain.

Site trafficability during dry weather should pose no problems, however inclement weather may cause clayey soils to soften and become unsuitable for construction traffic until the site conditions dry. In areas where high levels of construction traffic are expected, a temporary hardstand comprising crushed rock could be constructed to aid in trafficability during wet weather.

A copy of the CSIRO Building Technology File BTF 18 entitled, 'Foundation Maintenance and Footing Performance, A Homeowners Guide', which further describes appropriate site maintenance requirements set out within Appendix B of AS2870 is included in Appendix F.

8.10 Salinity

8.10.1 Impact of the Saline Soils on the Proposed Development

The presence of slightly to moderately saline soils and the sodic to highly sodic soils are naturally occurring features of the local landscape and are not considered significant impediments for future development of the site, provided appropriate remediation or management techniques are employed.

Salinity and aggressivity affects the durability of concrete and steel by causing premature breakdown of concrete and corrosion of steel. This has impacts on the longevity of structures in contact with these materials. As a result, management will be required.

Sodic soils have low permeability due to infilling of interstices with fine clay particles during the weathering process, restricting infiltration of surface water and potentially creating perched water tables, seepage in cut faces or ponding of water in flat open areas. In addition, sodic soils tend to erode when exposed. Management of sodic soils would therefore be required to prevent these adverse effects.

8.10.2 Salinity Management Plan

The current salinity investigation indicates that materials within the site range from non-saline to moderately saline with near-surface soils (within 0.5 m of the existing ground surface) generally non-saline. Testing of other parameters associated with salinity indicates that the materials are non – aggressive to concrete and steel. In addition, shallow soils ranged from non-sodic to highly sodic.

The amount of information regarding the distribution of salinity across the site is limited. Therefore, the management strategies assume the most conservative approach of moderately saline soils or sodic soils being present across the site.

The following management strategies are confined to the management of those factors with a potential to impact on the development:

A. Management should focus on capping of the upper surface of the sodic soils, both exposed by excavation and placed as filling, with a more permeable material to prevent ponding, to reduce capillary rise, to act as a drainage layer and to reduce the potential for erosion.

B. With respect to any required imported filling, which is expected to be only in small quantities, testing should be undertaken prior to importation, to determine the salinity characteristics of the material, which should not be greater than non - aggressive and, where possible, but should not be greater than “moderately saline” in classification.

C. Sodic soils can also be managed by maintaining vegetation where possible and planting new salt tolerant species. The addition of organic matter, gypsum and lime can also be considered where appropriate. After gypsum addition, reduction of sodicity levels may require some time for sufficient infiltration and leaching of sodium into the subsoils, however capping of exposed sodic material should remain the primary management method. Topsoil added at the completion of construction is, in effect, also adding organic matter which may help infiltration and leaching of sodium.

D. Avoiding water collecting in low lying areas, in depressions, or behind fill. This can lead to water logging of the soils, evaporative concentration of salts, and eventual breakdown in soil structure resulting in accelerated erosion.

E. Any pavements should be designed to be well drained of surface water. There should not be excessive concentrations of runoff or ponding that would lead to waterlogging of the pavement or additional recharge to the groundwater through any more permeable zones in the underlying filling material.

F. Surface drains should generally be provided along the top of batter slopes to reduce the potential for concentrated flows of water down slopes possibly causing scour.

G. Salt tolerant grasses and trees should be considered for landscaping in the drainage reserve, to reduce soil erosion and to maintain the existing evapo – transpiration and groundwater levels. Reference should be made to an experienced landscape planner or agronomist.

The following additional strategies are recommended for completion of service installation and for building construction. These strategies should be complementary to standard good building practices recommended within the Building Code of Australia, including cover to reinforcement within concrete and correct installation of a brick damp course, so that it cannot be bridged to allow moisture to move into brick work and up the wall.

H. Soils are classified as non-aggressive to concrete. Concrete piles, cast-in place, exposed to mildly aggressive soils should have a minimum strength of 25 MPa and a minimum cover to reinforcement of 65 mm (as per AS2159) to limit the corrosive effects of the surrounding soils (in accordance with AS2159).

I. With regard to concrete structures exposed to moderately saline soils with salinities in the range of 4 dS/m to 8 dS/m that are classified as non-aggressive to concrete (AS3600 – A2), slabs and foundations should have a minimum strength of 25 MPa, a minimum cover to reinforcement of 45 mm from unprotected ground and should be allowed to cure for a minimum of three days (as per AS3600) to limit the corrosive effects of the surrounding soils.

J. Wet cast concrete pipes and currently manufactured spun concrete pipes are understood to have estimated compressive strengths of 50 MPa and 60 – 70 MPa, respectively, in excess of the requirements for mass concrete in H to J above. Reference to the maximum and minimum test results of Table 3 (Section 6.2 of this report) and to Tables E1 and 3.1 of AS 4058 – 2007 "Precast concrete pipes" indicates that the site falls within the AS 4058 Clay/Stagnant (low sulphate) soil type (chlorides <=20,000 ppm, pH>=4.5 and sulphates <=1,000 ppm) and (in the absence of tidal water flow) falls within the AS 4058 Normal durability environment. Under these conditions, AS 4058-compliant reinforced concrete pipes of general purpose Portland cement, with a minimum cover to reinforcement of 10 mm, are expected to have a design life in excess of 100 years. Any concrete pipes installed within the site should employ AS 4058-compliant steel reinforced pipes of general purpose Portland cement, with minimum cover to reinforcement of 10 mm, or should be fibre reinforced.

9. References

- AS 1170 (2007) *Structural Design Actions, Part 4: Earthquake Actions in Australia*, Standards Australia
- AS 1726 (2017) *Geotechnical Site Investigations*, Standards Australia
- AS 2159 (2009) *Piling Design and Installation,* Standards Australia
- AS 2870 (2011) *Residential Slabs and Footings*, Standards Australia
- AS 3798 (2007) *Guidelines on Earthworks for Commercial and Residential Developments*, Standards Australia
- AS 4678 (2002) *Earth-retaining Structures*, Standards Australia
- Clark and Jones (1991) Penrith *1: 100 000 Geological Sheet* 9030 1st edition, Geological Survey of New South Wales, Sydney
- Department of Infrastructure, Planning and Natural Resources, *Salinity Potential in Western Sydney* (2002), NSW Government.
- NSW Department of Minerals and Energy, Penrith Geological Series Sheet No 9030 (1991), NSW Government.
- Pells, Mostyn and Walker (1998) *Foundations on Sandstone and Shale in the Sydney Region*, Australian Geomechanics Society
- Richards, L. A. (ed.) (1954), Diagnosis and Improvement of Saline and Alkaline Soils, USDA Handbook No. 60, Washington D.C.
- Walker and Pells (1998) *The Construction of Bored Piles Socketed into Shale and Sandstone*, Australian Geomechanics Society

10. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 85 The Ponds Boulevard, The Ponds in accordance with DP's proposal dated 2 June 2021 and was commissioned by Mr Matthew Wood of the Department of Education – Schools Infrastructure. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of NSW Department of Education for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

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

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

DATA FOR DESCRIPTION AND CLASSIFICATION OF SOILS – Page 1

Major Divisions				Description		Field Identification			
				Group Symbol*	Typical Name	Grading		Nature of Fines	Dry Strength
COARSE-GRAINED SOILS	More than 65% by dry mass, (excluding that larger than 63 mm) is greater than 0.075 mm	GRAVEL	More than 50% of coarse grains are greater than 2.36 mm	GW	Well graded gravels and gravel-sand mixtures, little or no fines.	Good	Wide range in grain size	'Clean' materials (not enough fines to bind grains)	None
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.	Poor	Predominantly one size or gap graded		
		GRAVELLY SOILS		GM	Silty gravels, gravel-sand-silt mixtures.	Good to Fair	'Dirty' materials with excess of fines	Fines are non-plastic	None to medium
				GC	Clay gravels, gravel-sand-clay mixtures.			Fines are plastic	Medium to high
		SAND	More than 50% of coarse grains are less than 2.36 mm	SW	Well graded sands and gravelly sands, little or no fines.	Good	Wide range in grain size	'Clean' materials (not enough fines to bind grains)	None
				SP	Poorly graded sands and gravelly sands, little or no fines.	Poor	Predominantly one size or gap graded		
		SANDY SOILS		SM	Silty sand, sand-silt mixtures.	Good to Fair	'Dirty' materials with excess of fines	Fines are non-plastic	None to medium
				SC	Clayey sands, sand-clay mixtures.			Fines are plastic	Medium to high
* For coarse grained soils where the fines content is between 5% and 12%, the soil shall be given a dual classification eg GP-GM.						Dry Strength		Dilatancy	Toughness
FINE-GRAINED SOILS	More than 35% by dry mass, (excluding that larger than 63 mm) is less than 0.075 mm	Liquid Limit less than 35%		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	None to low		Slow to rapid	Low
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high		None to slow	Medium
				OL	Organic silts and organic silty clays of low plasticity	Low to medium		Slow	Low
		35% <LL< 50%		CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high		None to slow	Medium
		Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.	Low to medium		None to slow	Low to medium
				CH	Inorganic clays of high plasticity, fat clays.	High to very high		None	High
				OH	Organic clays of medium to high plasticity.	Medium to high		None to very slow	Low to medium
				Pt	Peat muck and other highly organic soils.	Readily identified by colour, odour, spongy feel and generally fibrous texture			

ORDER OF DESCRIPTION

In the soil description the terms should be given in the following order:

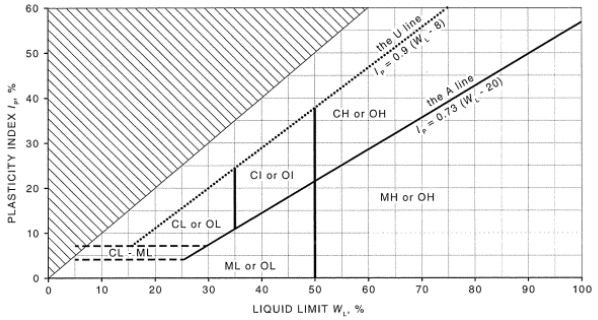
- SOIL NAME & UNIFIED CLASSIFICATION SYMBOL.
- Plasticity, behavioural or particle characteristics of the primary soil component
- Colour
- Secondary soil components' name(s), estimated proportion(s), plasticity, behavioural or particle characteristics, colour and where practical, its plasticity
- Moisture Condition (disturbed or undisturbed state)
- Consistency of fine-grained soils (undisturbed state only)
- Relative density of coarse-grained soils (determined by in situ tests)
- Structure of soil (in undisturbed state)
  - Zoning
  - Defects
  - Cementing
- Origin of soil
- Additional observations

EXAMPLES:

Silty SAND SM: fine to coarse grained, light brown, 15% non-plastic fines, with gravel, 20% angular particles, moist, apparently dense in place, alluvial.

SILT ML: low plasticity, brown, trace fine sand, w > PL, firm, estuarine.

PLASTICITY CHART (after AS 1726:2017)



The classification system excludes the boulder and cobble fractions of the soil and classifies only the materials less than 63 mm in size.

PARTICLE SIZES

Boulders	> 200 mm
Cobbles	63 mm to 200 mm
Gravel	2.36 mm to 63 mm
Sand	0.075 mm to 2.36 mm
Silt and Clay	< 0.075 mm

SAND			SILT
COARSE	MEDIUM	FINE	SILT
2.36-0.6 mm	0.6-0.2 mm	0.2-0.075 mm	0.075-0.002 mm

DATA FOR DESCRIPTION AND CLASSIFICATION OF SOILS – Page 2

GRAVEL

Density	Field Test
LOOSE	By inspection of voids and particle packing.
DENSE	

SAND

Density	Field Test	DPT Blows per 300 mm[1]		SPT N Blows	CPT q_c MPa	Relative Density %	Estimated Friction Angle
		Dry[2]	Wet[3]				
VERY LOOSE	Easily penetrated with 13 mm reinforcing rod pushed by hand.	< 1	0	0 – 4	0 – 2	0 – 15	25 - 30
LOOSE	Easily penetrated with 13 mm reinforcing rod pushed by hand. Can be excavated with a spade; 50 mm wooden peg can be easily driven.	1 - 3	< 1	4 – 10	2 – 5	15 – 35	27 - 32
MEDIUM DENSE	Penetrated 300 mm with 13 mm reinforcing rod driven by 2 kg hammer – hard shovelling.	3 - 8	1 - 6	10 – 30	5 – 15	35 – 65	30 - 35
DENSE	Penetrated 300 mm with 13 mm reinforcing rod driven with 2 kg hammer, requires pick for excavation; 50 mm wooden peg hard to drive	8 – 15	6 - 10	30 – 50	15 – 25	65 – 85	35 - 40
VERY DENSE	Penetrated only 25 – 50 mm with 13 mm reinforcing rod driven by 2 kg hammer.	> 15	> 10	> 50	> 25	85 – 100	38 - 43

[1]Valid for depths up to approx 1m bgl; [2]At a mc of approx. 3%-5%; [3]At a mc of approx. 15%.

SILT & CLAY

Consistency	Field Test		DCP Blows per 150 mm	SPT N Blows	Undrained Shear Strength C_u Shear Vane (kPa)	Unconfined Compressive Strength q_u PP* (kPa)	CPT q_c kPa
VERY SOFT	Easily penetrated > 40 mm by thumb. Exudes between thumb and fingers when squeezed in hand	Shear Vane Preferred	< 1	< 2	< 12	< 25	0 - 180
SOFT	Easily penetrated 10 mm by thumb. Moulded by light finger pressure.		1 – 1.5	2 – 4	12 – 25	25 – 50	180 - 375
FIRM	Impression by thumb with moderate effort. Moulded by strong finger pressure		1.5 – 3	4 – 8	25 – 50	50 – 100	375 - 750
STIFF	Slight impression by thumb cannot be moulded with finger		3 – 6	8 – 16	50 – 100	100 – 200	750 - 1500
VERY STIFF	Very tough. Readily indented by thumbnail.		6 – 12	16 – 32	100 – 200	200 – 400	1500 - 3000
HARD	Brittle. Indented with difficulty by thumbnail.		> 12	> 32	> 200	> 400	> 3000
FRIABLE	Easily crumbled or broken into small pieces by hand.						

* Pocket Penetrometer (**PP**) may overestimate q_u by a factor of 1.5 to 2.0.

Note: Visual-tactile assessment is indicative only. Use in-situ testing for logging

MOISTURE OF FINE GRAINED SOILS

Moist, dry of plastic limit	w < PL	Wet, near liquid limit	w ≈ LL
Moist, near plastic limit	w ≈ PL	Wet, wet of liquid limit	w > LL
Moist, wet of plastic limit	w > PL		

DEGREE OF SATURATION OF SANDS

Condition of Sand	Criteria	Degree of Saturation (%)
Dry	Non-cohesive and free-running	0 – 25%
Moist	Feels cool, darker colour, grains tend to adhere to one another	25 – 75%
Wet	Feels cold, makes hands wet, should be close to water table	75 – 99%

FIELD IDENTIFICATION PROCEDURE FOR FINE GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus 0.4 mm sieve size particles. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):

After removing particles larger than 0.4 mm sieve size, prepare a pat of moist soil with a volume of about 8000 mm^3. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):

After removing particles larger than 0.4 mm sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same dry strength but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit):

After removing particles larger than the 0.4 mm sieve size, a specimen of soil about 12 mm cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about 3 mm in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced, and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached. After the thread crumbles, the pieces should be lumped together, and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay or low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

PROPORTION OF MINOR AND SECONDARY COMPONENTS

Term	Meaning	Approximate Proportion	
		Coarse Soils	Fine Soils
Trace	Just detectable by feel or eye. Soil properties of main component virtually unaffected.	< 5% fines < 15 % coarse fraction	< 15% sand / gravel
With	Easily detectable by feel or eye. Soil properties only slightly affected by minor components.	5% – 12% fines 15% – 30% coarse fraction	15% – 30% sand / gravel
Prefix	Easily detected by feel or eye. Soil properties significantly affected by secondary components.	> 12% fines > 30% coarse fraction	> 30% sand / gravel

PROPORTIONS OF SECONDARY COMPONENTS

5%	12%	35%

DATA FOR DESCRIPTION AND CLASSIFICATION OF ROCK

SEDIMENTARY ROCK TYPE DEFINITIONS

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel sized (greater than 2 mm) fragments.
Sandstone	More than 50% of the rock consists of sand sized (0.06 mm to 2 mm) grains.
Siltstone	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale	More than 50% of the rock consists of silt or clay sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, e.g. Clayey SANDSTONE, Sandy SHALE.

DEGREE OF WEATHERING

Term	Abbreviation	Definition
Residual soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure, material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	HW	The whole of the rock is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching or may be decreased due to deposition of weathering products in pores.
Moderately Weathered	MW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable but shows little or no change of strength from fresh rock.
Slightly Weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition of individual minerals or colour changes.

DW*

*If highly and moderately weathered rock cannot be differentiated use the term, 'Distinctly Weathered (DW)'.

ORDER OF DESCRIPTION

In the rock description the terms should be given in the following order:
ROCK NAME
Grain size and type
Colour
Fabric and texture
Inclusions and minor components
Moisture content
Durability
Strength
Weathering and/or alteration
Defects – type, orientation, spacing, roughness
Stratigraphic unit
Geological structure

STRATIFICATION

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

DEGREE OF FRACTURING

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

Term	Description
Fragmented	The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than the core diameter
Highly Fractured	Core lengths are generally less than 20 mm to 40 mm with occasional fragments
Fractured	Core lengths are mainly 30 mm to 100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths are generally 300 mm or longer with occasional sections of 100 mm to 300 mm
Unbroken	The core contains very few fractures

ROCK STRENGTH

Rock strength is classified using the unconfined compressive strength (UCS). Where adequate UCS data are not available then the classification may be based on the Point Load Strength ($I_{S(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding.

Strength Term	UCS MPa	Field Guide	Approx $I_{S(50)}$ MPa
Material less than very low strength is to be described using soil properties			
Very Low	2	Material crumbles under firm blows with sharp end of pick; can be peeled with knife. Pieces up to 30 mm thick can be broken by finger pressure.	0.1
Low	6	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long by 50 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.3
Medium	20	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	1.0
High	60	A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.	3.0
Very High	200	Hand specimen breaks with pick after more than one blow; rock rings under hammer.	10.0
Extremely High		Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.	

The approximate point load strength ($I_{S(50)}$) is based on an assumed ratio to UCS of 1:20. This ratio may vary widely and should be determined for each site and rock type.

DISCONTINUITIES / DEFECTS

The actual defect is described not the process which formed or may have formed it, e.g. 'sheared zone', not 'zone of shearing'; the latter suggests a currently active process.

Spacing*:

A measure of the spacing of discontinuities. Measure mean and range of spacings for each set where possible (do not use descriptive terms).

Thickness, openness:

Measured in millimetres normal to plane of the discontinuity.

Persistence*:

The areal extent of a discontinuity. Give trace lengths in metres.

Roughness and Shape*:

A measure of the inherent surface unevenness and waviness of the defect relative to its mean plane.

Coating or Infilling:

Clean: no visible coating or infilling.

Stained: 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 (less than 1 mm).

Patchy Veneer: if discontinuous over the plane.

Coating: a visible coating or infilling of soil or mineral substance, greater than 1 mm thick. Describe composition and thickness.

* Usually determined in field exposures

Roughness:

Very Rough
Rough
Smooth
Polished
Slickensided

Shape*:

Planar
Curved
Undulating
Stepped
Irregular

Discontinuity Spacing in Three Dimensions:

The spacing of discontinuities in exposures may be described with reference to the size and shape of rock bounded by the discontinuities.

Equidimensional	Same size in all directions
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section
Polyhedral	Irregular defects without obvious pattern

Sampling

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Symbols & Abbreviations

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Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core Drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

Asphalt

Road base

Concrete

Filling

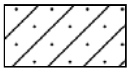
Soils

Topsoil

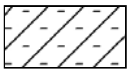
Peat

Clay

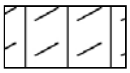
Silty clay

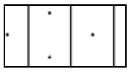

Sandy clay

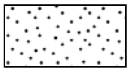
Gravelly clay

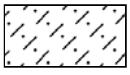

Shaly clay

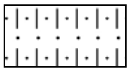
Silt


Clayey silt


Sandy silt

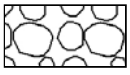

Sand

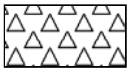

Clayey sand


Silty sand

Gravel

Sandy gravel


Cobbles, boulders

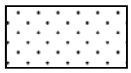

Talus

Sedimentary Rocks

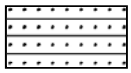
Boulder conglomerate

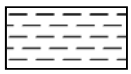
Conglomerate

Conglomeratic sandstone

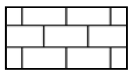

Sandstone

Siltstone


Laminite

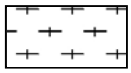

Mudstone, claystone, shale

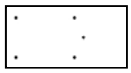
Coal


Limestone

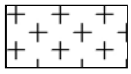
Metamorphic Rocks

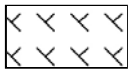
Slate, phyllite, schist

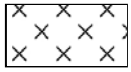

Gneiss

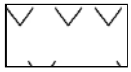

Quartzite

Igneous Rocks


Granite


Dolerite, basalt, andesite

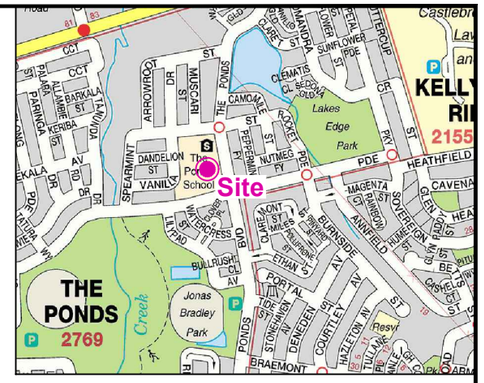

Dacite, epidote


Tuff, breccia

Porphyry

Appendix B

Drawing No 1



Location Plan

N

0 10 20 30 40 50m

SCALE 1:1250 (A3)

LEGEND:-

- Borehole locations (September 2021)
- Test pit location (September 2021)
- Borehole locations (August 2020)
- Previous test pit location (Project 41177A, February 2007)
- Previous test pit location (Project 44777, April 2007)
- Library
- Extension to Building A
- Approximate footprint of proposed buildings
- Approximate footprint of proposed car park
- Approximate footprint of proposed extensions
- Approximate site boundary

NOTE:-

1. Test locations are approximate only and are shown with reference to existing site features.
2. Image obtained from Near Map. Date of imagery 21-06-2020.

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CLIENT: Woolacotts Consulting Engineers Pty Ltd		TITLE: **Site and Test Location Plan** **John Palmer Public School Upgrade** **85 The Ponds Boulevard, The Ponds**
OFFICE: North West Sydney	DRAWN BY: JST	
SCALE: As shown	DATE: 8 October 2021	

PROJECT No:	94624.01
DRAWING No:	1
REVISION:	0

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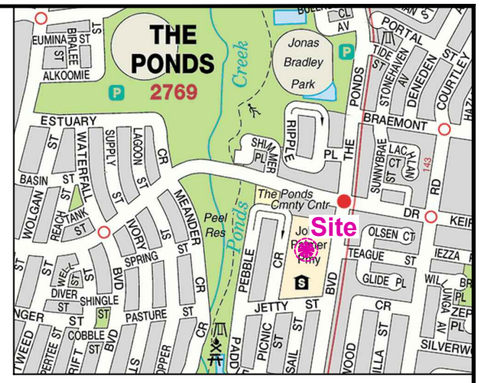

Location Plan

0 5 10 15 20

SCALE 1: 500 (A3)

LEGEND:-

- Borehole Location and Number
- Previous borehole location (Project 94624.00, September 2020)

NOTE:-

1. Test locations are approximate only and are shown with reference to existing site features.
2. Image obtained from Metromap. Date of imagery 29-08-2020.

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: NSW Department of Education		
OFFICE: North West Sydney	DRAWN BY: JST	
SCALE: As shown	DATE: 8 October 2021	

TITLE: **Site and Test Location Plan**
John Palmer Public School Upgrade
85 The Ponds Boulevard, The Ponds

PROJECT No:	94624.01
DRAWING No:	1A
REVISION:	0

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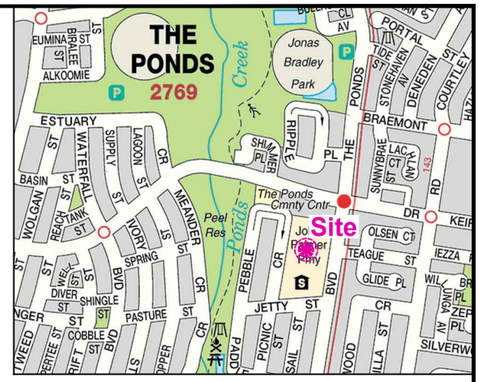

Location Plan

N

LEGEND:-

- Borehole locations (September 2021)
- Previous borehole location (July 2021)
- Previous borehole location (Project 94624.00, September 2020)

NOTE:-

1. Test locations are approximate only and are shown with reference to existing site features.
2. Image obtained from Metromap. Date of imagery 29-08-2020.

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: NSW Department of Education		
OFFICE: North West Sydney	DRAWN BY: JST	
SCALE: NTS	DATE: 8 October 2021	

TITLE: **Site and Test Location Plan**
John Palmer Public School Upgrade
85 The Ponds Boulevard, The Ponds

PROJECT No:	94624.01
DRAWING No:	1B
REVISION:	0

P:\NSW\94624.01\94624.01.D.001B.Rev0.dwg

Appendix C

Results of Previous Field Work

Appendix D

Results of Current Field Work

BOREHOLE LOG

CLIENT: NSW Department of Education
PROJECT: John Palmer Public School Upgrade
LOCATION: 85 The Ponds Boulevard, The Ponds

SURFACE LEVEL: 58.1
COORDINATE E:306488.2 **N:** 6268414
DATUM/GRID: GDA2020 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 201
PROJECT No: 94624.01
DATE: 29/06/21
SHEET: 1 of 2

GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	SOIL ORIGIN(#)	SOIL CONSIS.(*) DENSITY.(*)	SOIL MOISTURE	ROCK WEATH.	ROCK DEPTH (m)	ROCK STRENGTH	ROCK RECOVERY (%)	ROCK RQD	FRACTURE SPACING (m)	DEFECTS & REMARKS	SAMPLE REMARKS	SAMPLE TYPE	SAMPLE INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
29/06/21, No free groundwater observed	58	0.0	FILL/ Silty CLAY, with gravel; brown; with vegetation throughout (Topsoil)		FILL	NA	<PL													
		0.2	(Cl) Silty CLAY, trace gravel; pale brown grey; medium plasticity; trace vegetation; stiff		RES	ST	<PL									A E		0.2 0.3		
																A E U50		0.5 0.6 0.75		
	57	1														A E SPT		1.0 1.1 1.45	SPT	4,5,8 N=13
		1.5	(Cl) Silty CLAY; pale brown; medium plasticity; stiff to very stiff		RES	ST TO VST	<PL									A E		1.5 1.6		
	56	2.0	(Cl-CH) Silty CLAY; grey mottled orange and yellow; medium to high plasticity; hard		RES	H	<PL									SPT		2 2.5 2.95	SPT	8,18,18 N=36
	55	3.0	(CH) Silty CLAY, with gravel; grey; high plasticity; (extremely weathered siltstone)		RES	H	<PL											3		
	54	4												Unless otherwise noted all defects are bedding planes dipping at 0- 10 degrees		SPT		4.0 4.15	SPT	25,2/0 refusal
		4.2	SILTSTONE; brown grey; fine; low to medium strength, (Ashfield Shale)					MW	4.2	L-M	100	82		4.2-4.34m: J 50°-60° PL, SM, FE STN 4.46-4.5m: CS 40mm				4.2	PLT	PL(A)=0.13

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

EXPORTED 14/07/21 15:20. TEMPLATE ID: DP_103.02.00_COMBINED

PLANT: Hanjin 8D
OPERATOR: Rockwell
LOGGED: KA

METHOD: 110mm SFA to 4.0m, NMLC to 9.62m
CASING: HQ to 4.0m

REMARKS:

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: NSW Department of Education
PROJECT: John Palmer Public School Upgrade
LOCATION: 85 The Ponds Boulevard, The Ponds

SURFACE LEVEL: 58.1
COORDINATE E:306488.2 **N:** 6268414
DATUM/GRID: GDA2020 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 201
PROJECT No: 94624.01
DATE: 29/06/21
SHEET: 2 of 2

CONDITIONS ENCOUNTERED																	SAMPLE				TESTING	
				SOIL				ROCK														
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE	WEATH.	DEPTH (m)	STRENGTH VL L M H VH EH	RECOVERY (%)	RQD	FRACTURE SPACING (m) 0.01 0.05 0.10 0.50 1.00 5.00	DEFECTS & REMARKS	SAMPLE REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	53		SILTSTONE; brown grey; fine; low to medium strength, (Ashfield Shale) *(continued)*					MW		L-M										
		5.52	SILTSTONE; pale grey; fine; high strength, (Ashfield Shale)						5.52		100	82				C			PLT	PL(A)=0.72
	52	6						SW-FR		H								6	PLT	PL(A)=1.5
		6.77	SILTSTONE; grey; fine; high strength, (Ashfield shale)						6.77									6.62		
		7												4.2-9.62m: B x40 0°-5° PL, SM, CLY/CLN STN				7	PLT	PL(A)=1.7
	51																			
		8						FR		H	100	98				C		8	PLT	PL(A)=1.5
	50													8.58-8.69m: J 85°-90° PL, SM, CLN						
		9																9	PLT	PL(A)=1.6
	49													9.21-9.26m: J x3 85°-90° PL, SM, CLN						
		9.62	Borehole discontinued at 9.62m depth Limit of investigation						9.62									9.62		

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

EXPORTED 14/07/21 15:20. TEMPLATE ID: DP_103.02.00_COMBINED

PLANT: Hanjin 8D
METHOD: 110mm SFA to 4.0m, NMLC to 9.62m
REMARKS:

OPERATOR: Rockwell
CASING: HQ to 4.0m

LOGGED: KA

Refer to explanatory notes for symbol and abbreviation definitions

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BORE: BH201 PROJECT: 94624.01 July 2021

4.2m – 9.0m

BORE: BH201 PROJECT: 94624.01 July 2021

9.0m – 9.62m

BOREHOLE LOG

CLIENT: NSW Department of Education
PROJECT: John Palmer Public School Upgrade
LOCATION: 85 The Ponds Boulevard, The Ponds

SURFACE LEVEL: 59.4
COORDINATE **E:**306526.6 **N:** 6268415.3
DATUM/GRID: GDA2020 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 202
PROJECT No: 94624.01
DATE: 29/06/21
SHEET: 1 of 3

GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	SOIL ORIGIN(#)	SOIL CONSIS.(*) DENSITY.(*)	SOIL MOISTURE	ROCK WEATH.	ROCK DEPTH (m)	ROCK STRENGTH	ROCK RECOVERY (%)	ROCK RQD	ROCK FRACTURE SPACING (m)	ROCK DEFECTS & REMARKS	SAMPLE REMARKS	SAMPLE TYPE	SAMPLE INTERVAL	DEPTH (m)	TESTING TEST TYPE	TESTING RESULTS AND REMARKS
29/06/21, No free groundwater observed	59	0.0	FILL/ (CL) Silty CLAY, with gravel; brown; low plasticity; with vegetation throughout (Topsoil)		FILL	NA	<PL											0.2		
		0.2	FILL/ (CL) Silty CLAY, with gravel; brown; low plasticity; trace vegetation		FILL	ND	<PL									A E		0.25 0.5		
		0.7	(CI) Silty CLAY; grey mottled yellow; medium plasticity; very stiff		RES	VST	<PL									A E		0.55 1.0		
	58	1														A E SPT		1.05 1.45	SPT	6,13,12 N=25
		1.8	(CI-CH) Silty CLAY; pale yellow mottled orange and brown; medium to high plasticity; very stiff		RES	VST	<PL									A E		1.5 1.55		
	57	2																2		
		3.0	(CI-CH) Silty CLAY; grey mottled yellow and orange; medium to high plasticity; hard, (possibly extremely weathered siltstone)		RES	H	<PL									SPT		2.5 2.95	SPT	8,10,12 N=22
	56	3																3		
	55	4			RES	H	<PL									SPT		4.0 4.45	SPT	10,21,29 N=50

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

PLANT: Hanjin 8D
METHOD: 110mm diameter SFA to 5.5m, NMLC coring to 11.56m
REMARKS:

OPERATOR: Rockwell
CASING: HQ to 5.5m

LOGGED: KA

EXPORTED 21/07/21 16:14, TEMPLATE ID: DP_103.02.00_COMBINED

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: NSW Department of Education
PROJECT: John Palmer Public School Upgrade
LOCATION: 85 The Ponds Boulevard, The Ponds

SURFACE LEVEL: 59.4
COORDINATE **E:**306526.6 **N:** 6268415.3
DATUM/GRID: GDA2020 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 202
PROJECT No: 94624.01
DATE: 29/06/21
SHEET: 2 of 3

CONDITIONS ENCOUNTERED																				SAMPLE				TESTING	
				SOIL				ROCK																	
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE	WEATH.	DEPTH (m)	STRENGTH VL L M H VH EH	RECOVERY (%)	RQD	FRACTURE SPACING (m) 0.01 0.05 0.10 0.50 1.00 5.00	DEFECTS & REMARKS	SAMPLE REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	54	5.6	(CI-CH) Silty CLAY; grey mottled yellow and orange; medium to high plasticity; hard, (possibly extremely weathered siltstone) *(continued)*		RES	H	<PL													
			SILTSTONE; grey mottled orange brown; fine; low strength (Ashfield Shale)					SW	5.6	L				Unless otherwise noted all defects are bedding planes dipping at 0-10 degreees				5.6	PLT	0.12
	53	6.2	SILTSTONE; grey; fine; high strength (Ashfield Shale)						6.2											
	52	7									100	86		7.15-7.17m: CS 20mm		C		7	PLT	1.7
	51	8						FR		H								8	PLT	1.4
	50	9									100	77					8.45	9	PLT	1.5
																			PLT	1.7

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

EXPORTED 21/07/21 16:14, TEMPLATE ID: DP_103.02.00_COMBINED

PLANT: Hanjin 8D
OPERATOR: Rockwell
LOGGED: KA

METHOD: 110mm diameter SFA to 5.5m, NMLC coring to 11.56m
CASING: HQ to 5.5m

REMARKS:

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

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SURFACE LEVEL: 59.4
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LOCATION ID: 202
PROJECT No: 94624.01
DATE: 29/06/21
SHEET: 3 of 3

CONDITIONS ENCOUNTERED																		SAMPLE				TESTING	
				SOIL			ROCK																
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE	WEATH.	DEPTH (m)	STRENGTH VL L M H VH EH	RECOVERY (%)	RQD	FRACTURE SPACING (m) 0.01 0.05 0.10 0.50 1.00 5.00	DEFECTS & REMARKS	SAMPLE REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	49		SILTSTONE; grey; fine; high strength (Ashfield Shale) *(continued)*					FR		H	100	77				C			PLT	2.3
		11.0							11.0									11		
	48		SILTSTONE; grey; fine; very high strength (Ashfield Shale)							VH									PLT	3.8
		11.56							11.56									11.56		
			Borehole discontinued at 11.56m depth Limit of investigation																	
	12																	12		
	47																			
	13																	13		
	46																			
	14																	14		
	45																			

NOTES: (#)Soil origin is "probable" unless otherwise stated. (*)Consistency/Relative density shading is for visual reference only - no correlation between cohesive and granular materials is implied.

EXPORTED 21/07/21 16:14, TEMPLATE ID: DP_103.02.00_COMBINED

PLANT: Hanjin 8D
OPERATOR: Rockwell
LOGGED: KA

METHOD: 110mm diameter SFA to 5.5m, NMLC coring to 11.56m
CASING: HQ to 5.5m

REMARKS:

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: BH202 PROJECT: 94624.01 July 2021
5.6m – 10.0m

BORE: BH202 PROJECT: 94624.01 July 2021
10.0m – 11.56m

Appendix E

Laboratory Test Results

Appendix F

CSIRO Notes

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).

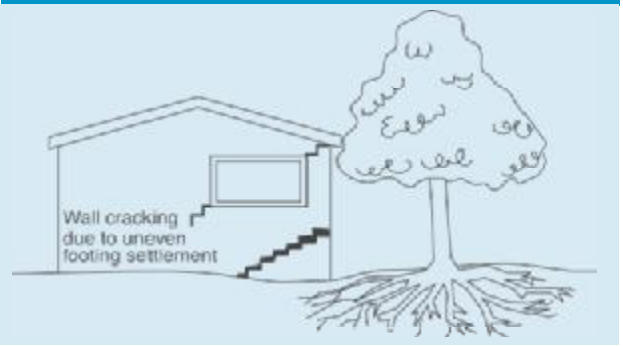
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

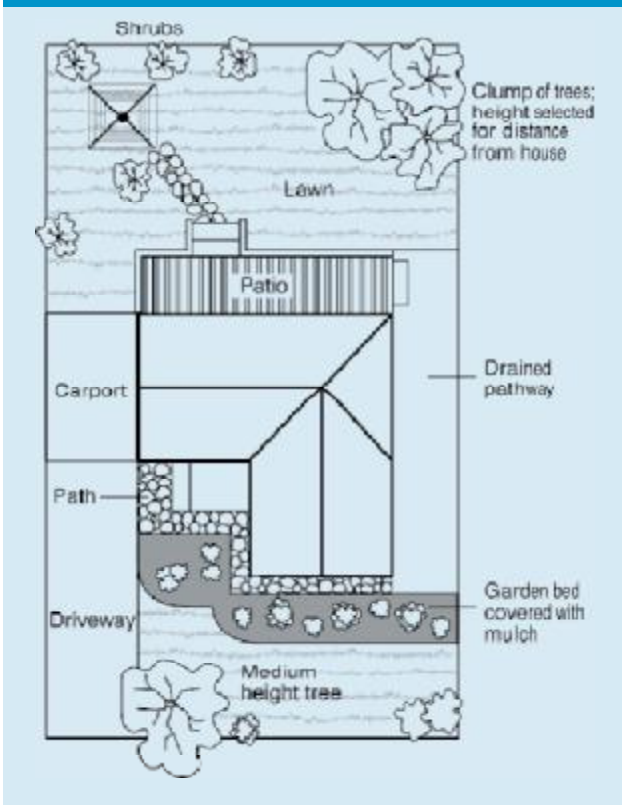
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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.

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

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

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

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