REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW BUILDINGS

at

KING GEORGE PARK, ROZELLE

Prepared For

LEICHHARDT COUNCIL

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REPORT ON GEOTECHNICAL INVESTIGATION FOR
PROPOSED NEW BUILDINGS AT
KING GEORGE PARK, ROZELLE

1. INTRODUCTION:

This report details the results of a geotechnical investigation conducted at King George Park, Rozelle, NSW. The investigation was undertaken at the request of Saltwater Studio Architects on behalf of the clients Leichhardt Council.

The site currently consists of a park/reserve with the proposed works along the southern edge. Just to the southeast of the site is an electricity substation and it is understood that several electricity and other service easements are located below ground near the proposed structures.

It is understood that the proposed works involve construction of two single storey buildings: an amenities building to be constructed at the southern side of the playing fields in the southwest corner of the site; and a storage building to be constructed at the southern end of the site. The new buildings will be on grade and founded on concrete slabs. No significant excavation is required although some minor works may be required for new building footings. It is understood that a report is required to allow structural engineering design.

The site is located within Leichhardt Council LEP (2013) as being Class 2 Acid Sulphate Soils (ASS) hazard zone. An assessment of the likelihood of these soils being disturbed is required as well as the impact of the proposed development on any groundwater table that may be present. It is assumed that due to the lack of excavation works proposed it is not likely that there will be significant disturbance to the ground.

The investigation comprised:

- Detailed geotechnical mapping of the entire site and adjacent land, with identification of all geotechnical hazards by a Senior Engineering Geologist and Geotechnical Engineer
- Drilling of four auger boreholes to refusal and four Dynamic Penetrometer tests to identify sub-surface geology and collect samples for ASS testing. This investigation was carried out using our mini drill-rig.
The following plans and diagrams were supplied for this work:

2. SITE FEATURES:

2.1. Description:
The site comprises a large sports field which is bordered to the north by Iron Cove. To the east of the park there are asphalt and grass surfaced car parks, accessed from Manning Street. The site is essentially flat, and is located on the lower northern side of the northeast trending ridge which extends into Sydney Harbour and forms the suburb of Rozelle.

2.2. Geology:
Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is located in an area of man-made fill, dredged estuarine sands and mud, demolition rubble, industrial and household waste (Mf) which may also contain sands of alluvial stream and estuarine origin of Holocene Age (<12,000 years old) (Qha). These sands comprise silty to peaty quartz sand, silt and clay with ferruginous and humic cementation in places. Common shell layers.

The geological map indicates that the rock adjacent is Hawkesbury Sandstone which is of Triassic Age. The rock unit typically comprises medium to coarse grained quartz sandstone with minor lenses of shale and laminite. This rock unit was identified in outcrops upslope of the site and in the surrounding area.

3. FIELD WORK:

3.1 Methods:
The field investigation comprised a walk over inspection of the site and limited inspection of adjacent properties on the 17th December 2014 by a Senior Engineering Geologist. It involved geological/geomorphological mapping of the site and adjacent land with examination of soil slopes, bedrock outcrops and existing structures for stability.

An additional site visit was undertaken on the 18th December 2014 which involved the drilling of four boreholes (BH1 to BH4) using a mini drill rig with solid stem, spiral flight augers and tungsten carbide bit along with four Dynamic Cone Penetrometer tests (DCP1 to DCP4), conducted in accordance with AS1289.6.3.2 1997, "Determination of the penetration resistance of a soil by 9kg dynamic cone penetrometer". Soil samples were collected and placed in sterile glass jars at several intervals to allow for laboratory testing for ASS.

Mapping information and test locations are shown on Figure: 1, in Appendix: 2 along with detailed log sheets. Explanatory notes are included in Appendix: 1.
3.2. Field Observations:
King George Park is situated at the base of a low, northeast trending ridge that forms the peninsula on which the suburb of Rozelle is formed. Throughout the surrounding area Hawkesbury Sandstone Formation outcropping can be observed. The park itself is essentially flat and is adjacent to Iron Cove to the north.

Access to the park is possible from Manning Street, where grass and asphalt surfaced carparks form the eastern end of the site. A single storey brick services building and a line of trees run along the eastern side of the park, which comprises a large sports field. Some concrete shot put circles and a shot put cage are also present in the southeast corner of the park.

To the southeast of the site and near the location of the proposed new buildings is an electrical substation which is partly cut into the ridgeline, while to the west of the substation is a heavily vegetated area which slopes moderately down to the site.

3.3. Boreholes:

Borehole 1 was drilled in the grass area at the eastern end of the proposed new amenities building (See Figure 1). This bore intersected medium dense to very dense dark brown and some orange/brown gravelly sand fill with blue metal gravel becoming black sandy gravel fill below 1.50m. The fill is underlain at 2.0m by grey/brown natural fine to medium grained wet sand containing some shell fragments. Between 3.0m and 3.90m depth the borehole intersected a layer of grey, high plasticity, wet silty clay, which contained shell fragments and had a sulphuric odour, with grey, medium grained, wet sand below 3.90m depth. This sand also contained shell fragments and had a sulphuric odour. Extremely to highly weathered, extremely to very low strength sandstone was encountered below 5.30m, with auger refusal at 5.40m on low strength sandstone.

DCP1 was undertaken adjacent to the borehole and extended to 0.52m depth where the hammer bounced, indicating refusal and the test was relocated nearby as DCP1A. This test advanced to 0.50m where the hammer bounced, indicating refusal and the test was terminated.

Groundwater was measured at 1.85m below existing ground level following the completion of drilling.

Borehole 2 was drilled in the grass area at the western end of the proposed new amenities building. This borehole intersected dark brown gravelly sand fill containing blue metal gravels until refusal on the gravels at 0.75m depth. The borehole was relocated nearby as BH2A.

Borehole 2A intersected medium dense to very dense dark brown gravelly sand fill containing blue metal gravel, with some blue metal cobbles and metal sheeting below 1.50m. At 2.20m the borehole intersected natural brown, medium grained, wet sand containing some shell fragments, becoming grey and with some clay patches below 3.0m. Extremely weathered, extremely low strength sandstone was encountered below 3.90m, becoming highly weathered and very low strength below 4.20m and with auger refusal at 4.25m on low to medium strength sandstone. DCP2 was undertaken adjacent to the borehole and extended to 0.35m depth where
the hammer bounced, indicating refusal and the test was terminated. Groundwater was measured at 1.70m below existing ground level following the completion of drilling.

Borehole 3 was drilled toward the western end of the park adjacent to the vegetated area which marks the southern boundary of the site. This bore intersected loose to medium dense dark brown gravelly sand fill containing some glass fragments, becoming sand fill below 0.40m and with a sandstone boulder between 0.65m and 0.80m depth. At 1.90m the borehole intersected orange/pale grey, extremely to highly weathered, extremely to very low strength sandstone which became grey/pale grey below 3.0m. At 3.30m depth the auger refused on low strength sandstone and the borehole was terminated. A DCP test (DCP3) was undertaken adjacent to the borehole and extended to 1.20m, indicating loose to medium dense fill soil to at least 1.20m depth. Groundwater was not measured in this borehole.

Borehole 4 was drilled to the east of the park adjacent to the southernmost carpark and Manning Street, at the potential location of the proposed storage building. This bore intersected medium to very dense brown gravelly sand fill, underlain at 1.50m by natural, brown, medium grained wet sand containing some shell fragments. Between 3.0m and 3.60m the borehole intersected a layer of grey, high plasticity, wet silty clay containing some shell fragments and with a sulphuric odour, underlain by brown/grey, medium grained, wet sand, also containing some shell fragments and with a sulphuric odour. The sand became grey below 4.50m and brown/grey below 5.40m before the borehole was discontinued at 5.50m within this sand. A DCP test (DCP4) was undertaken adjacent to the borehole and extended to 0.90m, indicating medium to very dense fill soil to at least 0.90m depth. Groundwater was measured at 1.85m below existing ground level following the completion of drilling.

### 3.4. Laboratory Test Results

Of the soil sampling carried out from the boreholes, samples from BH1, BH2A and BH4 were collected and tested at a NATA accredited laboratory (Envirolab Services) for their acid sulphate soil characteristics. Suspension Peroxide Oxidation Combined Acidity & Sulfur (sPOCAS) tests were carried out on all soil samples to provide existing acidity, potential acidity, and liming rates. The test results are summarised in Table 1 below.

<table>
<thead>
<tr>
<th></th>
<th>BH1 – 4.0m depth</th>
<th>BH2A – 4.0m depth</th>
<th>BH4 – 4.0m depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>7.7</td>
<td>9.2</td>
<td>9.1</td>
</tr>
<tr>
<td>pH\textsubscript{Ox}</td>
<td>2.3</td>
<td>7.8</td>
<td>8.3</td>
</tr>
<tr>
<td>a-Net Acidity (mole\textsubscript{H}^+/t)</td>
<td>390</td>
<td>&lt;10</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Liming rate (kg CaCO\textsubscript{3}/t)</td>
<td>30</td>
<td>&lt;0.75</td>
<td>&lt;0.75</td>
</tr>
</tbody>
</table>
4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified medium to very dense gravelly and sandy fill containing significant quantities of blue metal gravel and cobbles as well as glass and metal fragments to between 1.50m and 2.20m depth. This fill is underlain by grey and brown, fine to medium grained sand which contained shell fragments, with a slight sulphuric odour. A band of grey, high plasticity silty clay which also contained shell fragments and had a sulphuric odour was identified between 3.0m and 5.30m depth in BH1, and below 3.0m depth in BH4. These interpreted natural soil horizons are considered to represent Holocene aged estuarine sediments deposited in a mangrove environment on the foreshore. Extremely to highly weathered, extremely to very low strength sandstone bedrock was encountered at depths from 1.90m to 5.30m depth, grading to low to medium strength sandstone between 3.30m and 5.40m depth. Sandstone was not encountered in BH4 which was drilled to 5.50m. Groundwater was observed at depths between 1.70m and 1.85m depth.

It is understood that the proposed works involve construction of two single storey buildings, including an amenities building at the southern side of the sports field and a storage building at the southern side of the site. The new structures are not expected to require significant excavation except for footings to rock for the main amenities building and some other minor footing works. Due to the presence of the fill the new structures will be expected to require pier footings founded below the fill and possibly to bedrock. Alternatively if even bearing and some capacity for differential settlement are allowed for and the required footing loads are small, then a shallow footing system such as a raft slab may be used.

The site is located within Leichardt Council’s Acid Sulfate Soils Class 2 zoning. The investigation intersected acid sulfate soils in the sample taken from a depth of 4.0m in BH1, beneath the location of the proposed new amenities building. It is expected that the acid sulphate soils may be encountered from approximately 3.0m to 5.30m in this location and may also be present beneath the proposed storage building site. Therefore should piered footings be utilised through the natural soils then an acid sulphate soil management plan or treatment of excavated soils will be necessary prior to commencement of any excavation works that extend below 3.0m depth.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and boreholes drilled with a mini drill rig. This test equipment provides small isolated test points across the entire site with limited penetration into fill or rock; therefore some variation to the interpreted subsurface conditions is possible, especially between test locations.
4.2. Design & Construction Recommendations:

4.2.1. New Footings:

In accordance with the Australian Standard for Residential Slabs and Footings AS2870 – 2011 the site would be classified as a Class $P$ site due to the fill soils intersected below the site.

The results of the investigation suggest that the location of the proposed new amenities building is underlain by fill soil to between 2.0m and 2.20m, underlain by natural sand with a band of high plasticity silty clay between 3.0m and 3.90m, and extremely weathered, extremely low strength sandstone encountered between 3.90m and 5.30m. Low strength bedrock was encountered between 4.25m and 5.40m depth.

The western portion of the site, in the vicinity of BH3, is underlain by fill soil to 1.90m underlain by extremely to highly weathered, extremely to very low strength sandstone bedrock, with low strength sandstone at approximately 3.30m depth.

The proposed storage building is expected to be underlain by fill soil to 1.50m, underlain by natural sand with a band of silty clay between 3.0m and 3.60m, with bedrock not encountered to 5.50m (the limit of investigation).

It is recommended that shallow footings not be used due to the presence of fill which is of variable density and could result in differential settlement of structures unless consultation between the structural and geotechnical engineers has developed a shallow footing system that can overcome these issues. This may include a raft slab or similar founded at shallow depth.

The fill appeared to be relatively dense and may be suitable for a bearing capacity of 200kPa at 0.50m depth provided all footings are inspected and tested by a geotechnical engineer to identify loose areas. There is the possibility of differential settlement for footings founded at shallow depth within fill however this will be reduced by maintaining low, even bearing pressures across the structures.

Where higher loads footings or minimal capacity for settlement are required they will involve piers extending beyond the fill. Pier design will be dependent upon the structural engineer's requirements and may require further geotechnical testing to provide accurate design parameters. Footings founded on low strength bedrock, expected to be encountered at around 4.25m to 5.40m depth at the eastern end of the site and at around 3.30m depth at the western end, should be designed for a maximum allowable bearing capacity of 1000kPa. Higher footing pressures may be permissible following additional testing of the bedrock below footing level. All footings should be founded off bedrock of consistent strength to prevent differential settlement.

The groundwater table was intersected at depths between 1.70m and 1.85m depth. It is expected that the water table will have tidal characteristics below the site. Pier footings which extend below the water table or through sandy soils will require the use of drilled pier liners, bentonite, CFA/grout injection or similar methods. Drilling through the fill is expected to be difficult due to the presence of blue metal gravels and cobbles and occasional
metal fragments. Driven piles may be uses for the storage building and amenities building pending the requirements of the service owners.

The footing trenches and/or piers must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity. This is mandatory to allow them to be certified at the end of the project.

4.2.2 Site Exposure

The site is located less than 500m from the ocean and within a severe marine environment as per Australian Standard 3700 (2001) Masonry Structures. It is important to take into consideration the given environment during the design and construction of new brickwork, to minimise the potential for salt attack. The most suitable mortar joints for aggressive environments are ironed or weather struck joints.
5. CONCLUSION:

The investigation found that the site is underlain by sandy fill soils containing significant quantities of blue metal gravels and cobbles along with large unidentified objects to depths up to around 2.20m. Such material is likely to be contaminated and further disturbance of the ground will require environmental consultation and supervision. The fill is underlain by natural sand soils, with a layer of silty clay acid sulphate soils from approximately 3.0m to 5.30m, with weathered sandstone bedrock from around 3.90m to 5.30m at the location of the proposed new amenities building (Option 1). These depths will vary across the site due to man-made changes to the local topography.

The groundwater table was measured at depths between 1.70m and 1.85m and is expected to be tidal. The site is located in an aggressive environment for steel and concrete footings.

The use of shallow footings will require consultation between the structural and geotechnical engineers to provide a suitable footing system due to the potential for differential settlement due to variability in the fill density. Should deep pile footings be utilised for the proposed amenities building then an Acid Sulphate Management plan and Continuous Flight Auger (CFA) style piles will be required. All footings will need to be inspected, and tested, by a geotechnical engineer to identify soft areas and provide advice on suitable bearing capacity.

Ben Taylor
Geotechnical Engineer

Troy Crozier
Principal Engineering Geologist
6. REFERENCES:

Appendix 1
NOTES RELATING TO THIS REPORT

Introduction
These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the discussion and comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring 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.

Description and Classification Methods
The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties – strength or density, colour, structure, soil or rock and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Particle Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>Less than 0.002 mm</td>
</tr>
<tr>
<td>Silt</td>
<td>0.002 to 0.06 mm</td>
</tr>
<tr>
<td>Sand</td>
<td>0.06 to 2.00 mm</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.00 to 60.00 mm</td>
</tr>
</tbody>
</table>

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows:

<table>
<thead>
<tr>
<th>Classification</th>
<th>Under drained Shear Strength kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>Less than 12</td>
</tr>
<tr>
<td>Soft</td>
<td>12 - 25</td>
</tr>
<tr>
<td>Firm</td>
<td>25 - 50</td>
</tr>
<tr>
<td>Stiff</td>
<td>50 - 100</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>100 - 200</td>
</tr>
<tr>
<td>Hard</td>
<td>Greater than 200</td>
</tr>
</tbody>
</table>

None-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>SPT “N” Value (blows/300mm)</th>
<th>CPT Cone Value (qc-MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>Less than 5</td>
<td>Less than 2</td>
</tr>
<tr>
<td>Loose</td>
<td>5 - 10</td>
<td>2 - 5</td>
</tr>
<tr>
<td>Medium dense</td>
<td>10 - 30</td>
<td>5 - 15</td>
</tr>
<tr>
<td>Dense</td>
<td>30 - 50</td>
<td>15 - 25</td>
</tr>
<tr>
<td>Very Dense</td>
<td>greater than 50</td>
<td>greater than 25</td>
</tr>
</tbody>
</table>

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

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

Disturbed samples taken during sampling 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 with 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.

Details of the type and method of sampling are given in the report.

Drilling Methods
The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a back hole and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Penco) – the hole is advanced by a rotating plate of short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more that 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable that with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90-115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling of in-situ testing. This is a relatively economical means of drilling in Clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed...
Samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water 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 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tend to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength 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 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm 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 150mm of say 4,5 and 7
  \[ N = 13 \]
- In the case where the rest is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm

As 15,30/40 mm

- The results of the tests can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone and Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the core and the friction resistance on a separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises:

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0-5 MPa) is used in very soft soils in the graphs as a dotted line. The main scale (0-50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:

\[ q_c (\text{MPa}) = (0.4 \text{ to } 0.6) \text{ N (blow per 300mm)} \]

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:

\[ q_c = (12 \text{ to } 18) \text{ cu} \]

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements. Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension roads.

Two relatively similar tests are used:

- Perth sand penetrometer — a 16mm diameter flat-ended rod is driven with a 9kg hammer, dropping 600mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
• Cone penetrometer (sometimes known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein 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. 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 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, the frequency of sampling and the possibility of other than straight line variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems.

• In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
• A localized 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 of recent weather changes. They may not be the same at the time of construction as are indicated in the report.
• The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations 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.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company 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 condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the company cannot always anticipate or assume responsibility for

• Unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency.
• Changes in policy or interpretation of policy by statutory authorities.
• The action of contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advise to resolve the matter.

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, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document “Guideline for the Provision of Geotechnical Information in Tender Documents”. Published by the Institution of Engineers, Australia. Where information obtained from this investigation 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, 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. The Company 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 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.
APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

Show orientations, widths etc as appropriate.
Symbols for surface features should be drawn to reflect their true shape and extent, as far as possible.

Example of Mapping Symbols
Appendix 2
CROZIER - Geotechnical Consultants
Geotechnical Engineers & Engineering Geologist

SITE PLAN & TEST LOCATIONS

LEGEND

SCALE: 1:1000
PREPARED FOR: Leichhardt Council.

CONSTRUCTION OF TWO NEW BUILDINGS
KING GEORGE PARK
PROJECT: 2014-263  DATE: 07/05/2015

DRAWING: 2014-263
DRAWN: AW
APPROVED BY:

KING GEORGE PARK

FIGURE 1.

PROPOSED RUGBY

MANNING STREET

LEONARD STREET

PARK STREET

WYNYARD STREET

MEADOW STREET

PROPOSED RUGBY

SAND PIT

BURY

SW Consent

STREET

SLOPE ANGLE

TOP OF SLOPE

BASE OF SLOPE

ROCK OUTCROP & BURIED BOULDERS

SURFACE WATER FLOW

SCALE: 1:1000 @ A3
PREPARED FOR:
Leichhardt Council.

CONSTRUCTION OF TWO NEW BUILDINGS
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PROPOSED RUGBY

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SURFACE WATER FLOW

SCALE: 1:1000 @ A3
PREPARED FOR:
Leichhardt Council.

CONSTRUCTION OF TWO NEW BUILDINGS
KING GEORGE PARK
PROJECT: 2014-263  DATE: 07/05/2015

DRAWING: 2014-263
DRAWN: AW
APPROVED BY:

KING GEORGE PARK

FIGURE 1.
GEOLOGICAL MODEL

FIGURE 2

PREPARED FOR:
Leichhardt Council.

CONSTRUCTION OF TWO NEW BUILDINGS
KING GEORGE PARK
PROJECT: 2014-263 DATE: 07/05/2015

SCALE: 1:200 @ A3
DRAWING: 2014-263
DRAWN: AW
APPROVED BY:

CROZIER - Geotechnical Consultants
Geotechnical Engineers & Engineering Geologist

CROZIER - Geotechnical Consultants
Geotechnical Engineers & Engineering Geologist

LEGEND

- Sandstone

- Bedrock

- Fill

- Clay

- Sand

- Water Table

- Surface Water Flow

- Test Locations

- SANDSTONE

- BEDROCK

- SAND

- CLAY

- FILL

- WATER TABLE

- SURFACE WATER FLOW

- TEST LOCATIONS

- SANDSTONE

- BEDROCK

- SAND

- CLAY

- FILL

- WATER TABLE

- SURFACE WATER FLOW

- TEST LOCATIONS

- SANDSTONE

- BEDROCK

- SAND

- CLAY

- FILL

- WATER TABLE

- SURFACE WATER FLOW

- TEST LOCATIONS

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- BEDROCK

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- SAND

- CLAY

- FILL

- WATER TABLE

- SURFACE WATER FLOW

- TEST LOCATIONS

- SANDSTONE

- BEDROCK

- SAND
### Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILL - Medium dense to very dense, dark brown, some orange/brown, medium grained, moist gravelly sand fill, some blue metal</td>
</tr>
<tr>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>* 1.50m becoming sandy gravel fill, black</td>
</tr>
<tr>
<td>2.00</td>
<td>SAND - Grey/brown, fine to medium grained, wet sand, some shell fragments</td>
</tr>
</tbody>
</table>

**Sampling**

- **Type:** D
- **Depth (m):** 2.40

**In Situ Testing**

- **Results:**

---

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit

**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.85m after completion

---

**RIG:** Dingo Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT  
**CLIENT:** Leichhardt Council  
**DATE:** 18/12/2014  
**BORE No.:** 1  
**PROJECT:** Construction of Two New Buildings  
**PROJECT No.:** 2014-263  
**LOCATION:** King George Park, Rozelle  
**SURFACE LEVEL:** RL ñ 3.5m

---

Craizer Geotechnical
# TEST BORE REPORT

**CLIENT:** Leichhardt Council  
**DATE:** 18/12/2014  
**BORE No.:** 1  

**PROJECT:** Construction of Two New Buildings  
**PROJECT No.:** 2014-263  
**SHEET:** 2 of 3  

**LOCATION:** King George Park, Rozelle  
**SURFACE LEVEL:** RL ≈ 3.5m  

## Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>PRIMARY SOIL - strength/density, colour, grain size/plasticity, moisture, soil type incl. secondary constituents, other remarks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>SILTY CLAY - Grey, high plasticity, wet silty clay, sulfur odour, some shells</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.90</td>
<td>SAND - Grey, medium grained, wet sand, some shells, sulfur odour</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Sampling Data

- **Type:** D  
- **Depth (m):** 3.80  
- **POCAS:** J  
- **Depth (m):** 4.00

## In Situ Testing

- **Results:** sPOCAS  

**RIG:** Dingo Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT  

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit  

**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.85m after completion  

**REMARKS:**  

---

Crozier Geotechnical
### Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00</td>
<td>PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.30</td>
<td>SANDSTONE - Grey, extremely to highly weathered, extremely to very low strength</td>
<td>D</td>
<td>5.40</td>
</tr>
<tr>
<td>5.50</td>
<td>AUGER REFUSAL AT 5.40m on low strength sandstone</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Sampling and In Situ Testing

- **Type**
- **Depth (m)**
- **Results**

**RIG:** Dingo Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT  
**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit  
**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.85m after completion  
**REMARKS:**

---

Crozier Geotechnical
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<table>
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<tr>
<th>Depth (m)</th>
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</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILL - Medium dense to very dense, dark brown, fine grained, moist gravelly sand fill, coarse blue metal gravels</td>
</tr>
<tr>
<td>0.50</td>
<td>AUGER REFUSAL AT 0.75m on unknown fill</td>
</tr>
<tr>
<td></td>
<td>Relocated nearby as BH2A</td>
</tr>
<tr>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td></td>
</tr>
</tbody>
</table>

**Sampling**

<table>
<thead>
<tr>
<th>Type</th>
<th>Depth (m)</th>
<th>Type</th>
<th>Results</th>
</tr>
</thead>
</table>

**In Situ Testing**

- Freestanding groundwater measured at 1.85m after completion

**RIG:** Dingo Mini Drill Rig
**DRILLER:** JB
**LOGGED:** BT

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit

**GROUND WATER OBSERVATIONS:**

**REMARKS:**

---

**CLIENT:** Leichhardt Council  
**DATE:** 18/12/2014  
**BORE No.:** 2  
**PROJECT:** Construction of Two New Buildings  
**PROJECT No.:** 2014-263  
**SHEET:** 1 of 1  
**LOCATION:** King George Park, Rozelle  
**SURFACE LEVEL:** RL ≈ 3.5m
**TEST BORE REPORT**

**CLIENT:** Leichhardt Council  
**DATE:** 18/12/2014  
**BORE No.:** 2A  

**PROJECT:** Construction of Two New Buildings  
**PROJECT No.:** 2014-263  
**SURFACE LEVEL:** RL 3.5m

**LOCATION:** King George Park, Rozelle  
**SHEET:** 1 of 2

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILL - Medium dense to very dense, dark brown, fine grained, moist gravelly sand fill, coarse blue metal gravels</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>* 1.50m blue metal cobbles and metal sheeting present</td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.20</td>
<td>SAND - Brown, medium grained, wet sand, some shell fragments</td>
<td></td>
</tr>
<tr>
<td>2.50</td>
<td>J</td>
<td></td>
</tr>
</tbody>
</table>

**RIG:** Dinga Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT  

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit

**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.70m after completion

**REMARKS:**

**CHECKED:** TMC  

---

*Crozier Geotechnical*
## Test Bore Report

**CLIENT:** Leichhardt Council

**DATE:** 18/12/2014

**BORE No.:** 2A

**PROJECT:** Construction of Two New Buildings

**PROJECT No.:** 2014-263

**LOCATION:** King George Park, Rozelle

**SURFACE LEVEL:** RL Æ 3.5m

---

### Description of Strata

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<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>*3.0m becoming grey, some clay in patches</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.50</td>
<td>D</td>
<td>3.50</td>
<td></td>
</tr>
<tr>
<td>3.90</td>
<td>SANDSTONE - Pale grey, extremely weathered, extremely low strength</td>
<td>J</td>
<td>sPOCAS</td>
</tr>
<tr>
<td>4.00</td>
<td>* 4.20m becoming highly weathered, very low strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.50</td>
<td>AUGER REFUSAL AT 4.25m on low to medium strength sandstone</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**RIG:** Dingo Mini Drill Rig

**DRILLER:** JB

**LOGGED:** BT

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit

**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.70m after completion

**REMARKS:**  

---

_Crozier Geotechnical_
# Test Bore Report

**Client:** Leichhardt Council  
**Date:** 18/12/2014  
**Bores No.:** 3

**Project:** Construction of Two New Buildings  
**Project No.:** 2014-263  
**Sheet:** 1 of 2  
**Location:** King George Park, Rozelle  
**Surface Level:** RL ≈ 3.5m

<table>
<thead>
<tr>
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<tbody>
<tr>
<td></td>
<td>PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.00</td>
<td>FILL - Loose to medium dense, dark brown, fine to coarse grained, moist gravelly sand fill, glass fragments present</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.90</td>
<td>SANDSTONE - Orange/pale brown, extremely weathered, extremely low strength, medium grained</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Rig:** Dingo Mini Drill Rig  
**Driller:** JB  
**Logged:** BT  
**Method:** Spiral Flight Augers with Tungsten Carbide Bit  
**Ground Water Observations:** Borehole collapse before groundwater could be measured  
**Remarks:**

---

Craizer Geotechnical
### Description of Strata

#### PRIMARY SOIL - strength/density, colour, grainsize/plasticity, moisture, soil type incl. secondary constituents, other remarks

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>*2.50m to 3.0m extremely weathered, extremely low strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>*3.0m becoming grey/pale grey, fine to medium grained, highly weathered, very low strength, wet</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.30</td>
<td>AUGER REFUSAL AT 3.30m on low strength sandstone</td>
<td>D</td>
<td>3.30</td>
</tr>
</tbody>
</table>

### Sampling

- **Type**: D
- **Depth (m)**: 3.30

### In Situ Testing

#### Results

- **Results**: Borehole collapse before groundwater could be measured

### REMARKS

- **Checked**: TMC
- **Log**: BT

### Method

- **RIG**: Dingo Mini Drill Rig
- **DRILLER**: JB
- **LOGGED**: BT

- **METHOD**: Spiral Flight Augers with Tungsten Carbide Bit

- **GROUND WATER OBSERVATIONS**: Borehole collapse before groundwater could be measured
## TEST BORE REPORT

**CLIENT:** Leichhardt Council  
**DATE:** 18/12/2014  
**BORE No.:** 4  
**PROJECT:** Construction of Two New Buildings  
**PROJECT No.:** 2014-263  
**LOCATION:** King George Park, Rozelle  
**SURFACE LEVEL:** RL ≈ 3.5m

### Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>FILL - Loose to very dense, brown, fine to medium grained, moist gravelly sand fill</td>
</tr>
<tr>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>SAND - Brown, medium grained, wet sand, some shells</td>
</tr>
<tr>
<td>2.00</td>
<td></td>
</tr>
</tbody>
</table>

### Sampling

<table>
<thead>
<tr>
<th>Type</th>
<th>Depth (m)</th>
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</tr>
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</table>

### In Situ Testing

<table>
<thead>
<tr>
<th>Type</th>
<th>Results</th>
</tr>
</thead>
</table>

**RIG:** Dingo Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT  
**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit  
**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.85m after completion  
**REMARKS:**

---

Crozier Geotechnical
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description of Strata</th>
<th>Sampling</th>
<th>In Situ Testing</th>
<th>Type</th>
<th>Depth (m)</th>
<th>Type</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>PRIMARY SOIL - strength/density, colour, grain size/plasticity, moisture, soil type incl. secondary constituents, other remarks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>SILTY CLAY - Grey, high plasticity, wet silty clay, sulfur odour, some shell fragments</td>
<td></td>
<td></td>
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<tr>
<td>3.50</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.60</td>
<td>SAND - Brown/grey, medium grained, wet sand, some shells, sulfur odour</td>
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<td></td>
<td></td>
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<tr>
<td>4.00</td>
<td></td>
<td></td>
<td></td>
<td>D</td>
<td>4.00</td>
<td></td>
<td>sPOCAS</td>
</tr>
<tr>
<td>4.50</td>
<td>*4.50m becoming grey</td>
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<td></td>
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</tr>
</tbody>
</table>

RIG: Dingo Mini Drill Rig DRILLER: JB LOGGED: BT

METHOD: Spiral Flight Augers with Tungsten Carbide Bit

GROUND WATER OBSERVATIONS: Freestanding groundwater measured at 1.85m after completion

REMARKS: CHECKED: TMC
## Description of Strata

<table>
<thead>
<tr>
<th>Depth (m)</th>
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<th>In Situ Testing</th>
</tr>
</thead>
<tbody>
<tr>
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<td>D 5.40</td>
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</tr>
<tr>
<td></td>
<td>*5.40m becoming brown/grey</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.50</td>
<td>BOREHOLE TERMINATED AT 5.50m in sand</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Crozier Geotechnical

**RIG:** Dingo Mini Drill Rig  
**DRILLER:** JB  
**LOGGED:** BT

**METHOD:** Spiral Flight Augers with Tungsten Carbide Bit

**GROUND WATER OBSERVATIONS:** Freestanding groundwater measured at 1.85m after completion

**REMARKS:**
## Test Location

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>1</th>
<th>1A</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
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<tbody>
<tr>
<td>0.00 - 0.15</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>2</td>
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<tr>
<td>0.15 - 0.30</td>
<td>10</td>
<td>6</td>
<td>10</td>
<td>9</td>
<td>5</td>
</tr>
<tr>
<td>0.30 - 0.45</td>
<td>8</td>
<td>8</td>
<td>11(\text{B})</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>0.45 - 0.60</td>
<td>8(\text{B})</td>
<td>8(\text{B})</td>
<td>Bouncing at 0.35m</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>0.60 - 0.75</td>
<td>Bouncing at 0.52m</td>
<td>Bouncing at 0.50m</td>
<td>3</td>
<td>15</td>
<td></td>
</tr>
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<td>0.75 - 0.90</td>
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<td></td>
</tr>
<tr>
<td>1.20 - 1.35</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1.35 - 1.50</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1.50 - 1.65</td>
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<tr>
<td>1.65 - 1.80</td>
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<td></td>
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<tr>
<td>1.80 - 1.95</td>
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<td></td>
</tr>
<tr>
<td>1.95 - 2.10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.10 - 2.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.25 - 2.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.40 - 2.55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.55 - 2.70</td>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>2.70 - 2.85</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>2.85 - 3.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

**REMARKS:**
- \(\text{B}\) Test hammer bouncing upon refusal on solid object
- -- No test undertaken at this level due to prior excavation of soils
Appendix 3
CERTIFICATE OF ANALYSIS 121314

Client: Crozier Geotechnical Consultants
Unit 12/42-46 Wattle Rd
Brookvale
NSW 2100

Attention: Tony Crozier

Sample log in details:
Your Reference: 2014-263 Rozelle
No. of samples: 3 Soils
Date samples received / completed instructions received 19/12/2014 / 19/12/2014

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:
Date results requested by: / Issue Date: 24/12/14 / 24/12/14
Date of Preliminary Report: Not Issued
NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. Tests not covered by NATA are denoted with *.

Results Approved By:

[Signature]
Jacinta Hunt
Laboratory Manager
<table>
<thead>
<tr>
<th>sPOCAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Our Reference:</td>
</tr>
<tr>
<td>Type of sample</td>
</tr>
<tr>
<td>Date prepared</td>
</tr>
<tr>
<td>Date analysed</td>
</tr>
<tr>
<td>pH</td>
</tr>
<tr>
<td>TAA pH 6.5</td>
</tr>
<tr>
<td>s-TAA pH 6.5</td>
</tr>
<tr>
<td>pH</td>
</tr>
<tr>
<td>TPA pH 6.5</td>
</tr>
<tr>
<td>s-TPA pH 6.5</td>
</tr>
<tr>
<td>TSA pH 6.5</td>
</tr>
<tr>
<td>s-TSA pH 6.5</td>
</tr>
<tr>
<td>ANCE</td>
</tr>
<tr>
<td>a-ANCE</td>
</tr>
<tr>
<td>s-ANCE</td>
</tr>
<tr>
<td>SkCl</td>
</tr>
<tr>
<td>Sp</td>
</tr>
<tr>
<td>SPOS</td>
</tr>
<tr>
<td>a-SPOS</td>
</tr>
<tr>
<td>CaCl</td>
</tr>
<tr>
<td>CaP</td>
</tr>
<tr>
<td>CaA</td>
</tr>
<tr>
<td>MgKCl</td>
</tr>
<tr>
<td>MgP</td>
</tr>
<tr>
<td>MgA</td>
</tr>
<tr>
<td>SNAS</td>
</tr>
<tr>
<td>Fineness Factor</td>
</tr>
<tr>
<td>a-Net Acidity</td>
</tr>
<tr>
<td>Liming rate</td>
</tr>
<tr>
<td>a-Net Acidity without ANCE</td>
</tr>
<tr>
<td>Liming rate without ANCE</td>
</tr>
<tr>
<td>Method ID</td>
</tr>
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<td>-----------</td>
</tr>
<tr>
<td>Client Reference: 2014-263 Rozelle</td>
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### QUALITY CONTROL

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<th>Duplicate</th>
<th>Duplicate results</th>
<th>Spike Sm#</th>
<th>Spike %</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Sm#</td>
<td>Sm#</td>
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#### sPOCAS

**Date prepared:** 22/12/2014 || 22/12/2014
**Date analysed:** 22/12/2014 || 22/12/2014

<table>
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<tr>
<th>pH</th>
<th>Inorg-064</th>
<th>[NT]</th>
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<td>9.2</td>
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<td>TAA pH 6.5</td>
<td>5 Inorg-064</td>
<td>&lt;5</td>
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<tr>
<td>s-TAA pH 6.5</td>
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</tr>
<tr>
<td>pH ox</td>
<td>Inorg-064</td>
<td>[NT]</td>
</tr>
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<td>TPA pH 6.5</td>
<td>5 Inorg-064</td>
<td>&lt;5</td>
</tr>
<tr>
<td>s-TPA pH 6.5</td>
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<td>&lt;0.01</td>
</tr>
<tr>
<td>ANCE</td>
<td>% CaCO3</td>
<td>0.05 Inorg-064</td>
</tr>
<tr>
<td>a-ANCE</td>
<td>moles</td>
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</tr>
<tr>
<td>s-ANCE</td>
<td>% w/w</td>
<td>0.05 Inorg-064</td>
</tr>
<tr>
<td>SKCI</td>
<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>SP</td>
<td>% w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>SPOS</td>
<td>% w/w</td>
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<tr>
<td>a-SPOS</td>
<td>moles</td>
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</tr>
<tr>
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<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>CaP</td>
<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>CaA</td>
<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>MgKCI</td>
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<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>MgP</td>
<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>MgA</td>
<td>% w/w</td>
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</tr>
<tr>
<td>SHCI</td>
<td>% w/w</td>
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<tr>
<td>SNAS</td>
<td>% w/w</td>
<td>0.005 Inorg-064</td>
</tr>
<tr>
<td>a-SNAS</td>
<td>moles</td>
<td>5 Inorg-064</td>
</tr>
<tr>
<td>s-SNAS</td>
<td>% w/w</td>
<td>0.01 Inorg-064</td>
</tr>
<tr>
<td>Fineness Factor</td>
<td>1.5 Inorg-064</td>
<td>&lt;1.5</td>
</tr>
<tr>
<td>a-Net Acidity</td>
<td>moles</td>
<td>10 Inorg-064</td>
</tr>
<tr>
<td>Liming rate</td>
<td>kg CaCO3</td>
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</tbody>
</table>

**Notes:**
- RPD: Recovery Percentage Difference
- [NT]: Not Tested
- [NR]: Not Reported
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<th>QUALITY CONTROL</th>
<th>UNITS</th>
<th>PQL</th>
<th>METHOD</th>
<th>Blank</th>
<th>Duplicate Sm#</th>
<th>Duplicate results</th>
<th>Spike Sm#</th>
<th>Spike % Recovery</th>
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<tr>
<td>sPOCAS</td>
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<td>Inorg-064</td>
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<td>121314-1</td>
<td>&lt;10</td>
<td></td>
<td>&lt;10</td>
</tr>
<tr>
<td>Liming rate without ANCE</td>
<td>kg CaCO3 / t</td>
<td>0.75</td>
<td>Inorg-064</td>
<td>&lt;0.75</td>
<td>121314-1</td>
<td>&lt;0.75</td>
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<td>&lt;0.75</td>
</tr>
</tbody>
</table>
Report Comments:

Asbestos ID was analysed by Approved Identifier: Not applicable for this job
Asbestos ID was authorised by Approved Signatory: Not applicable for this job

INS: Insufficient sample for this test  PQL: Practical Quantitation Limit  NT: Not tested
NA: Test not required  RPD: Relative Percent Difference  NA: Test not required
<: Less than  >: Greater than  LCS: Laboratory Control Sample
**Quality Control Definitions**

**Blank:** This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

**Duplicate:** This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

**Matrix Spike:** A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

**LCS (Laboratory Control Sample):** This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

**Surrogate Spike:** Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

**Laboratory Acceptance Criteria**

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and specified phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.
Appendix 4
Foundation Maintenance and Footing Performance: A Homeowner’s Guide

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 geological 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 planning of the site for construction. Building Technology File 19 (BT F 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 binding layers.

However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clay 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

<table>
<thead>
<tr>
<th>Class</th>
<th>Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Most sand and rock sites with little or no ground movement from moisture changes</td>
</tr>
<tr>
<td>S</td>
<td>Slightly reactive clay sites with only slight ground movement from moisture changes</td>
</tr>
<tr>
<td>M</td>
<td>Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes</td>
</tr>
<tr>
<td>H</td>
<td>Highly reactive clay sites, which can experience high ground movement from moisture changes</td>
</tr>
<tr>
<td>E</td>
<td>Extremely reactive sites, which can experience extreme ground movement from moisture changes</td>
</tr>
<tr>
<td>A to P</td>
<td>Filled sites</td>
</tr>
<tr>
<td>P</td>
<td>Sites which include soft soils, such as soft clay or silt or loose sands, landfills, mine subsidence, collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise</td>
</tr>
</tbody>
</table>
Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause soil settlement in two ways:
- Roots that grow under footings may increase in cross-sectional area, 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 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 the area water-ponded. It can also occur wherever there is a source of water near footings in clay soil. This leads to a decrease in strength of the soil, which may cause 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 cause 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 brick (usually not necessary in line with the vertical beds or pendants).
- 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 permitting inside the building footings 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 area of the building, the internal footings will rise if the spread of moisture is greatly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun’s effect is strongest. This has the effect of lowering the 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.

Drowning 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—domming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensities 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 point 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 wall/Alhinkl 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 easily 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 swelling than masonry buildings because of their flexibility. Also, the damping effects tend to be less because of the light 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 fail away this can double the span which a wall must bear. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by an opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage. Although general symptoms of settling are a definite risk to such structures as edges crack and settle, it is not usual for framing members to fail. This is because of the flexibility of the framing system and the ability of the structure to absorb the stress.

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 waterlogged even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, internal 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. Incomet falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

<table>
<thead>
<tr>
<th>Description of typical damage and required repair</th>
<th>Approximate crack width limit (see Note 3)</th>
<th>Damage category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hairline cracks</td>
<td>&lt;0.1 mm</td>
<td>0</td>
</tr>
<tr>
<td>Fine cracks which do not need repair</td>
<td>&lt;1 mm</td>
<td>1</td>
</tr>
<tr>
<td>Cracks noticeable but easily filled. Doors and windows stick slightly</td>
<td>&lt;5 mm</td>
<td>2</td>
</tr>
<tr>
<td>Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weather tightness often impaired</td>
<td>515 mm (or a number of cracks 3 mm or more in one group)</td>
<td>3</td>
</tr>
<tr>
<td>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distorted. Walls lean or bulge noticeably some loss of bearing in beams. Service pipes disrupted</td>
<td>1525 mm but also depend on number of cracks</td>
<td>4</td>
</tr>
</tbody>
</table>

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 slabs, 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 change 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 BT F 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
Water that is transmitted into masonry masonry 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 of threat of upheaval of footings, if the offending roots are subsoil 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-Alnwick effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearing 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 Lever FAIB, MIAMA, Partner, Construction Diagnosis.