Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

New Science and Technology Building
6A Waropara Road, Medowie

Prepared for
Medowie Christian School

Project 81808.03
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Integrated Practical Solutions

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Report on Geotechnical Investigation
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1. Introduction

This report presents the results of a geotechnical investigation undertaken for a new science and technology building to be constructed in Medowie Christian School located at 6A Waropara Road, Medowie. The investigation was commissioned by Medowie Christian School Ltd and was undertaken in consultation with SHAC, architecture for this project. The investigation was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal NCL180353 dated 14 June 2018.

It is understood that the proposed design consists of a 3-storey building encompassing a range of spaces for flexible teaching and learning, as well as providing new Science and Technology laboratories. The façade of the building houses the double height ISTEM laboratory and includes a large glass curtain wall. A second storey linking bridge is proposed for direct connection to the existing Administration Building, allowing accessibility to the proposed building via the existing Administration lift. It is understood that significant earthwork consisting of cut and fills with the construction of retaining walls (height up to 4 m) is anticipated during site preparation.

The aim of the investigation was to assess the subsurface soil and groundwater conditions and provide comments on the following:

- Subsurface conditions;
- Site classification to AS2870 with regard to the reactivity of the soil;
- Site factor to AS 1170.4 with regard to earthquake effects;
- Footing design options and parameters (shallow footings and piled footings);
- Estimated settlements including differential settlement;
- Retaining wall design parameters including temporary and long term batter slope requirements;
- Depth to groundwater (if encountered) and its impact during construction;
- Site preparation requirements including excavatability and suitability of material for reuse on site;
- Comment on soil aggressiveness (pH, EC, sulfates and chlorides); and
- Comment on the site locality in relation to mine subsidence districts as designated by the Subsidence Advisory NSW (SA NSW) and any restrictions that may apply to the proposed development described by the SA NSW guidelines.

The investigation comprised the drilling of two boreholes supplemented by Dynamic Penetrometer Test (DPT), laboratory testing and preparation of this report. The details are presented in this report together with comments on the items listed above.

2. Site Description

The site is located within the north-western corner of the Medowie Christian School, off Waropara Road, Medowie. The site of the proposed new building was occupied by an existing Block S school building comprising a combination of rendered brick and steel sheeting at the time of the investigation.

The existing building is bounded by the Administration building to the south-west, an existing carpark to the north-west and a combination of grassed and concrete footpath and driveway areas in the remaining directions.

The site of the proposed new building is shown below in Figure 1.

Figure 1: Aerial image of Medowie Christian School showing site location of proposed new building.

Based on LiDAR contour information the ground surface level at the school site ranges from approximately RL 19.5 AHD in the west to 16.5 AHD in the east.

The existing ground level beneath the footprint of the existing building appears to have been levelled by cut and fill earthworks with batters observed on the western and eastern sides of the existing building and an approximately 1 m to 1.5 m high retaining wall observed on the southern side of the building. The cut batter on the western side of the building was observed to be up to about 1.5 m in height sloping at approximately 30º towards the existing building and the fill batter on the eastern side of the building was observed to slope at approximately 15º to 20º away from the existing buildings. The existing ground surface in the areas surrounding the existing building was observed to slope down towards the north-east.

Features of the site are shown in Figures 2 to 4 below.

Figure 2: Looking south-west towards existing Block S with Bore 101 in the background.

Figure 3: Looking south-east towards Block S from existing carpark.

Figure 4: Looking north-west toward Block S.

Reference to the 1:100,000 Newcastle Coalfields Regional Geology Map indicates that the site is underlain by the bedrock of the Tomago Coal Measures. The investigation encountered filling overlying residual clay soils.

3. Field Work Methods

Field work was undertaken on 3 July 2018 and comprised the following:

- Drilling of two boreholes (designated Bores 101 and 102) taken to depths of 5.95 m and 9.0 m below existing ground surface; and
- Dynamic cone penetrometer (DCP) testing was undertaken at three locations (103, 104 and 105) in addition to each borehole location.

The bores were drilled using a four-wheel drive mounted site investigation drilling rig equipped with 100 mm diameter solid flight augers. Standard penetration tests (SPTs) were performed at selected depths.

Test locations were set out by a geotechnical engineer from DP based on the proposed development, site features and presence of in-ground services. The engineer also logged the subsurface profile at each test location and collected samples for laboratory testing and identification purposes.

At the completion of drilling, the boreholes were backfilled with the cuttings from the drilling process.

The approximate test locations of bores and DCPs are shown on Drawing 1, in Appendix C.

4. Field Work Results

The subsurface conditions encountered at the bores are presented in the Borehole Logs in Appendix A. These should be read in conjunction with the preceding accompanying notes which explain the descriptive terms and classification methods used in the logs.

A summary of the subsurface conditions encountered at the test locations is provided below:

FILLING	Generally comprising silty clay, clay and sand filling to depths of 0.1 m and 1.5 m in Bores 101 and 102, respectively.
CLAY	Initially stiff becoming very stiff to hard residual clay at depths of 2.0 m and 4.3 m in Bores 101 and 102, respectively.

Groundwater was observed in Bore 102 at 0.9 m depth within the filling, whilst augering. It is possible the groundwater observed in Bore 102 was perched within the filling overlying the low permeability natural clay soils. No free groundwater was observed in the Bore 101 whilst augering. It should be noted that groundwater levels are affected by factors such as climatic conditions and soil permeability and will therefore vary with time.

5. Laboratory Testing

Laboratory testing was carried out on cohesive materials from the bores within the proposed building area and comprised the following:

- Two shrink-swell tests; and
- Two soil aggresivity tests.

Detailed results of laboratory testing are attached in Appendix B and summarised in Tables 1 and 2 below.

Table 1: Summary of Geotechnical Laboratory Testing

Bore	Depth (m)	Description	FMC (%)	LL (%)	PL (%)	PI (%)	Iss (% per ΔpF)
101	0.5 – 0.68	Yellow brown CLAY	32.5	-	-	-	3.8
102	1.7 -1.9	Grey CLAY	30.1	85	21	64	-

The results of the soil aggressivity testing are summarised in below:

Table 2: Summary of Soil Aggressiveness Test Results

Bore	Depth (m)	Description	Laboratory Results			
			pH	EC (µS/cm)	Soluble Sulphate SO_4 (mg/kg)	Soluble Chloride Cl (mg/kg)
101	**1 – 1.45**	**Grey mottled red brown CLAY**	**4.8**	**330**	**65**	**410**
102	**2.5 – 2.95**	**Orange brown CLAY**	**5.2**	**280**	**120**	**330**

6. Proposed Development

The design for the proposed development consists of a 3-storey building encompassing a range of spaces for flexible teaching and learning, as well as providing new Science and Technology laboratories. A second storey linking bridge is proposed for direct connection to the existing Administration Building, allowing accessibility to the proposed building via the existing Administration lift. Significant earthwork is anticipated during site preparation including retaining wall up to a height of 4 m may be required. Maximum working column loads are anticipated to be in the range of 200 kN to 1500 kN compression with tension load requirements for several piles up to 300kN.

7. Comments

7.1 Site Classification

Site classification to AS 2870 is not strictly applicable to this site due to it being a school development rather than a residential development. However, the principles of footing design and site maintenance presented therein should be taken into account for structures such as that proposed for the site.

Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with seasonal variation in moisture. The site classification is based on procedures presented in AS 2870-2011 (Ref 1), the typical soil profiles revealed in the bores, and the results of laboratory testing.

Owing to the presence of the existing buildings and possible abnormal moisture conditions beneath the building footprint and existing filling to depths greater than 0.4 m, the site classification for the site is Class P in accordance with AS2870 – 2011.

As a guide for footing design, the range of characteristic surface movements (y_s) is estimated to be approximately 45 mm to 55 mm for footings founded in the natural very stiff clay under normal seasonal moisture fluctuations without the influence of filling and abnormal moisture conditions beneath the footprint of the existing building and on the understanding that new, additional filling is not proposed.

Design, construction and maintenance should take into account the need to achieve and preserve an equilibrium soil moisture regime beneath and around buildings. Such measures include paved areas around structures to fall away from the building, flexible plumbing connections and service trenches to be backfilled with compacted clay. These and other measures are described in AS 2870-2011 and the CSIRO-BTF 18 publication in Appendix A.

Site classification, as above, has been based on the information obtained from the bores and on the results of laboratory testing, and have involved some interpolation between data points. In the event that conditions encountered during construction are different to those presented in this report, it is recommended that advice be sought from this office.

7.2 Footings

7.2.1 Shallow Foundation

It is considered that shallow pad or strip footings would be suitable for the support of structural loads associated with the proposed development. Shallow footings founded at least 0.5 m depth below the finished surface level and within the natural stiff or better clay, as encountered below about 0.1 m depth in Bore 101 and 1.5 m depth in Bore 102, should be proportioned for a maximum allowable bearing capacity of 150 kPa.

Estimated total settlements of up to about 20 mm (independent of seasonal reactive ground movements) are anticipated for the footings with a maximum width of 2.5 m and proportioned for the recommended allowable bearing capacity above.

Differential settlements would depend on the loading applied between adjacent footings. It should be noted that an increases in footing dimensions or applied pressure will result in non-linear increases in settlement.

Footings should not be founded in existing or proposed filling unless it has been placed and compacted under Level 1 earthworks as defined in AS 3798-2007 (Ref 2).

Footing excavations should be inspected by geotechnical engineers to confirm design parameters.

7.2.2 Deep Foundation (Piles)

As maximum working column loads are anticipated to be in the range of 200 kN to 1500 kN compression with tension load requirements for several piles up to 300kN, pile foundations may be required to support some structural loads where a shallow footing is not practical. Suitable pile types could include:

- Continuous flight auger (CFA) piles or bored piles; or
- Steel screw (helical) piles.

The design of the piles should be based on the parameters presented in Table 3 below and in accordance with AS 2159-2009 (Ref 3).

Table 3: Pile Design Parameters

Soil Description	Ultimate Strength ($R_{d,ug}$)*		Serviceability/Max Allowable End - Bearing (kPa)	Elastic Modulus (MPa)
	Design End - Bearing (kPa)	Design Shaft Adhesion (kPa)		
Very stiff to Hard Clay	1500	60	500	35

Notes to Table 3:

Ultimate end bearing occurs at large displacements (> 5% of footing diameter) as opposed to shaft adhesion which occurs at very less displacement (1% of footing diameter).

Serviceability / Max allowable end bearing to cause settlement of < 1% of minimum footing dimension or pile diameter.

AS 2159 – 2009 requires that the contribution of the shaft from finished surface to 1.5 times pile diameter or 1 m (whichever is greater) shall be ignored.

Piles should be founded at least four pile diameters below finished surface levels.

Shaft adhesion should not be considered in screw pile capacity assessment.

A geotechnical reduction factor (ϕ_g) of 0.4 is recommended for the pile design if no static or high-strain dynamic testing of installed piles will be undertaken during pile installation. The ϕ_g value could be increased if static or high-strain dynamic testing is carried out on a proportion of the piles (the higher the proportion of piles tested, the higher ϕ_g becomes). The value of testing will depend on cost-benefit analysis that compares the cost of testing to the potential savings in pile installation.

For CFA or bored piles in tension, the shaft adhesion parameters should be reduced to 75% of the values in Table 3.

For vertical loading, it is suggested that piles should be spaced at 2.5 pile diameters or greater such that the overall capacity of the pile group can be equivalent to the sum of the individual piles (i.e. group efficiency factor of unity).

Piled foundations proportioned on the basis of the above parameters would be expected to experience total settlements of less than about 1% to 2% of the pile diameter under the applied working (Serviceability) load with differential settlements between adjacent columns expected to be less than about half of this value.

If any water collects in the base of the pile holes, this should be removed, and the excavation base checked for potential softening and over-drilled as necessary, prior to pouring of concrete. Suitable founding strata should be confirmed by a geotechnical engineer during construction.

If steel screw piles are selected, the piling contractor will require a clear indication of the depth of pile installation due to limitations in confirming pile capacity during installation of the piles. They are solely end bearing piles with design parameters generally similar to that of bored or other non-displacement pile types. Notwithstanding the above, the settlement and bearing capacity of steel screw piles is not only dependent on the modulus of the bearing stratum, but also upon the structural strength of the helix. It is noted that some contractors rely on in-house correlations between torque and pile capacity, although the experience of DP is that such relationships are often invalid for layered soil profiles. Static load testing is the only reliable way of confirming steel screw pile capacity. Generally, it is recommended that the ratio of the pile helix outstand to the helix plate thickness be less than 10, otherwise considerable elastic flexing or plastic deformation of the helix plate will occur and conventional pile settlement predictions could be exceeded.

It should be noted that the tension capacity of steel screw piles is highly variable. This is largely due to installation technique and the consequential degree of soil disturbance caused by the helix auger as it penetrates the soil. If, for example, the helix is installed too fast, or slow, the soil is sheared (around the perimeter of the helix) and uplift capacity of the pile can be as low as the weight of the “cylinder” of soil on the pile, the pile itself, plus some small value of disturbed (or residual) interface friction for the soil. Conversely, high tension or uplift capacities are achieved when screw piles are installed at the rate of one-times the pitch (typically 150 mm) per revolution of the helix auger. At this installation rate, soil disturbance is minimised and the full passive resistance can be mobilised on the top face / side of the helix. In summary the capacity of steel screw piles is operator dependant and the rate of penetration is very important to the tension capacity of the pile.

7.3 Site Drainage

During the investigation it was noted that the ground surface was damp adjacent to the footprint of the proposed building and groundwater was observed to be perched within the filling encountered in Bore 102. Based on our previous work for the administration building construction at the Medowie School, it was understood that seepage regularly occurs within the area immediately upslope of, and within, the proposed building footprint.

To minimise the effects of fluctuations of moisture content within the reactive soils present at the site, it is recommended that a cut-off drain is constructed upslope of the proposed building to intersect surface and near surface water flows. The cut-off drain should be located at a distance of 3 m from the building and a minimum long gradient of 2% should be maintained in the area between the cut-off drain and the building. The drain should be constructed to at least 0.6 m depth and include dual ag-line encapsulated in geofabric and surrounded by free draining gravel, with the upper 0.3 m of the trench excavation backfilled with low permeability clay soils, such as present at the site. The drains should convey the collected water into the formal stormwater collection system downslope of the proposed building development. Consideration should also be given to damp proofing the ground floor slab.

7.4 Excavation and Reuse of Excavated Material

Excavation of the clay soils encountered at the site is anticipated to be readily achieved with the use of conventional earthmoving equipment, such as 5 tonne or larger excavators.

The residual clay soils are considered suitable for the re-use as engineered filling, provided they are placed and compacted with control of layer thickness, moisture content and compaction. Due consideration should be given to the effect on reactive soil movements should clay material won on site be used beneath the foundations or floor slabs for the proposed structures as it may lead to a more severe site classification.

7.5 Retaining Walls

7.5.1 Temporary Excavation

It is understood that excavation of up to 4 m may be required for the construction of the building and retaining walls will be constructed along the excavation interface. The clay encountered in the bores is generally of stiff or better consistency and would be expected to stand unsupported in the short term for excavation height less than 1.5 m. However, there would be the possibility of localised dry friable lumps dislodging. This may be exacerbated by prolonged exposure and adverse weather. The risk could be reduced by ensuring a short exposure period, and undertaking the construction in sections, if feasible.

For the excavation height is greater than 1.5 m, the stiff or stronger clay should be battered no steeper than 1H:1V. Alternatively, the excavation can be benched with vertical cut not exceeding 1.5 m height at each bench level and at an effective slope no steeper than 1H:1V.

The above recommendation is applicable for cut batters with no seepage flow through the slope face. Geotechnical advice should be sought if considerable seepage is observed from the cut batter and stability of the slope should be assessed before any work is carried out at the toe of the cut.

7.5.2 Design Parameters

For permanent retaining walls, where the wall will be free to deflect, design may be based on “active” (K_a) earth pressure coefficients, assuming a triangular earth pressure distribution. This would comprise any non-propped or laterally unrestrained walls (e.g. cantilever type walls). Cantilever walls should not be used to support any adjacent building foundations or underground services unless it is designed for the additional surcharge loading. Walls which are not free to move, should be designed for an at rest earth pressure coefficient (K_o) in additional to any surcharge from the footings if support of adjacent footings is required.

The suggested long term (permanent) design soil parameters are shown in Table 4 below. Any additional surcharge loads, including those imposed by proposed footings or inclined slopes, during or after construction, should be accounted for in design.

Table 4: Geotechnical Parameters for Retaining Structures

Parameter	Symbol	Clay
Bulk Density	γ	18 kN / m^3
Effective Cohesion	$c^{'}$	0 kPa
Angle of Friction	$\phi^{'}$	25°
Active Earth Pressure Coefficient	K_a	0.4
At Rest Earth Pressure Coefficient	K_o	0.6

Backfill placed behind the wall should be free-draining (20 mm single size gravel or coarser) and connected to the wall drainage system. A slotted drainage pipe should be placed at the base of the backfill which should all be encapsulated in a geotextile fabric. Alternatively, the retaining wall should be designed for full hydrostatic pressure. Retaining walls greater than 2 m high should have additional slotted piles at 2 m vertical intervals.

A clay lining, a dish drain or impermeable surface should be formed at the top of the wall backfill to prevent stormwater overland flow surcharging the retaining wall.

The very stiff or better clay would be a suitable bearing stratum for retaining wall footings which should be proportioned for a maximum allowable bearing pressure of 150 kPa in clay.

7.6 Soil Aggressivity

The results of the laboratory testing on the soil collected from Bore 101 and 102 were compared against exposure classification limits provided in AS2159-2009 (Ref 3). The results of the testing indicated that the soil is mildly aggressive to buried concrete based on soil pH and non-aggressive to buried steel piles.

7.7 Earthquake Site Factor

Using the results of test bores as well as the procedures described in AS1170.4 – 2007 (Ref 4), a hazard factor of 0.10 and a site sub-soil Class C_e – shallow soil site should be used for structural design of the building.

7.8 Mine Subsidence

Reference to the planning portal on the SA NSW website indicates that the site does not lie within a mine subsidence district and therefore is not restricted by any development restrictions imposed by SA NSW.

8. References

1. Australian Standard AS2870 - 2011, "Residential Slabs and Footings".
2. Australian Standard AS3798-2007, "Guidelines on Earthworks for Commercial & Residential Developments", Standards Australia, March 2007.
3. Australian Standard AS2159-2009, "Piling - Design and Installation".
4. Australian Standard AS1170.4-2007, "Structural Design Actions, Part 4: Earthquake Actions in Australia".

9. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 6A Waropara Road, Medowie in accordance with DP's proposal NCL180353 dated 14 June 2018 and acceptance received from Garry Couper dated 21 June 2018. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Medowie Christian School 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.

This report must be read in conjunction with all of the attached 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.

The scope for work for this investigation / report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the (geotechnical / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About this Report

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.

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.

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.

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Symbols & Abbreviations

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

BOREHOLE LOG

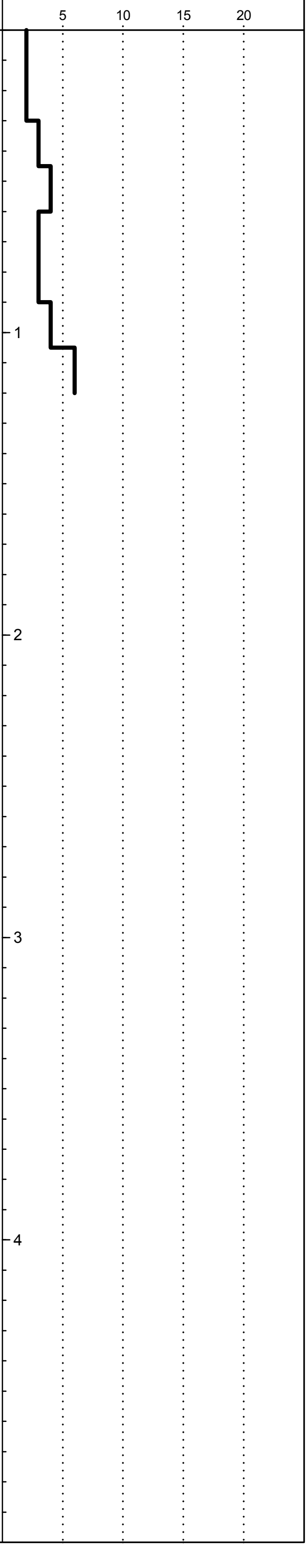
CLIENT: Medowie Christian School
PROJECT: New Science and Technology Building
LOCATION: 6A Waropara Road, Medowie

SURFACE LEVEL: --
EASTING: 392441
NORTHING: 6377693
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 81808.03
DATE: 3/7/2018
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water
	0.1	FILLING - Grey silty clay filling, with some fine grained sand and subrounded to subangular gravel up to 10mm in size and trace to some rootlets, M>Wp		D	0.05			
		CLAY - (Stiff to very stiff), yellow brown clay, with some silt and trace roots, M>Wp		D	0.3			
				U_{50}	0.5 0.6		pp = 250	
				D	0.8			
1				S	1.0 1.3 1.45		2,4,3 N = 7 pp = 250-300	
2	2.0	CLAY - Very stiff to hard, grey clay with some silt and trace fine to medium grained sand, M>Wp						
				S	2.5 2.7 2.95		pp = 400 6,8,11 N = 19	
3								
4				S	4.0 4.4 4.45		5,8,12 N = 20 pp = 350	
	4.3	SANDY CLAY - Very stiff, grey sandy clay, with fine to medium grained sand, M<Wp (possible rock like structure)						
	5.0							

Dynamic Penetrometer Test
(blows per 150mm)



RIG: MD20q **DRILLER:** Sytech Drilling **LOGGED:** West **CASING:** Nil
TYPE OF BORING: Solid flight auger
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Hand held GPS ± 5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Medowie Christian School
PROJECT: New Science and Technology Building
LOCATION: 6A Waropara Road, Medowie

SURFACE LEVEL: --
EASTING: 392441
NORTHING: 6377693
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 81808.03
DATE: 3/7/2018
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 150mm) 5 10 15 20
		SILTY CLAY - COMPLETELY WEATHERED SILTSTONE - Hard, brown silty clay / completely weathered siltstone							
				S	5.5 5.8 5.95		4,12,13 N = 25 pp = 550		
	6								6
	7			S	7.0 7.3 7.45		3,17,14 N = 31 pp = 350		7
	7.2	CARBONACEOUS CLAY - Stiff to very stiff, carbonaceous clay with completely weathered coal / point leases up to 200mm thick, M>Wp							
	8								8
	8.2	CLAY - (Very stiff) brown clay, with some silt, carbonaceous in parts, M>Wp							
	9 9.0	Bore discontinued at 9.0m, limit of investigation							9

RIG: MD20q **DRILLER:** Sytech Drilling **LOGGED:** West **CASING:** Nil
TYPE OF BORING: Solid flight auger
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Hand held GPS ± 5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

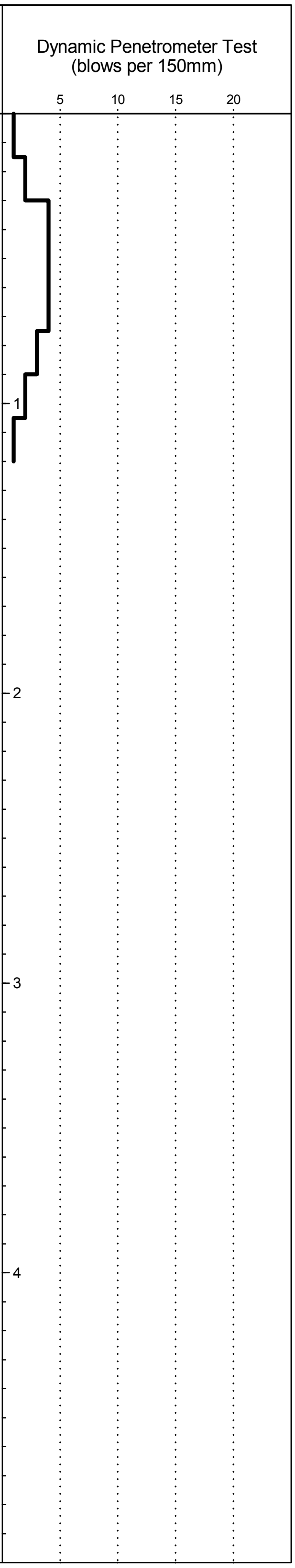
BOREHOLE LOG

CLIENT: Medowie Christian School
PROJECT: New Science and Technology Building
LOCATION: 6A Waropara Road, Medowie

SURFACE LEVEL: --
EASTING: 392490
NORTHING: 6377667
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 81808.03
DATE: 3/7/2018
SHEET 1 OF 2

RL	Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water
		FILLING - Generally comprising grey brown clay filling, with some fine to medium grained sand and subrounded to subangular gravel up to 10mm in size, M>Wp					
	0.4	FILLING - Generally comprising yellow brown clay filling, with some fine to medium grained sand and trace subrounded to subangular gravel up to 10mm in size, M>Wp	U_{50} D	0.7 0.8 0.9			
	0.9	FILLING - Generally comprising grey fine to medium grained sand with some silt, saturated	D	1.2			▼ 03-07-18
	1.5	CLAY - Stiff, grey clay with some silt and trace fine grained sand, M>Wp	U_{50}	1.7 1.8 1.9		pp = 200	
			S	2.5 2.7 2.95		pp = 250 2,4,8 N = 12	
			S	4.0 4.3 4.45		5,14,25 N = 39 pp = 400-500	
	4.3	SANDY CLAY - Very stiff, grey sandy clay with fine to medium grained sand, M>Wp					



RIG: MD20q **DRILLER:** Sytech Drilling **LOGGED:** West **CASING:** Nil
TYPE OF BORING: Solid flight auger
WATER OBSERVATIONS: Free groundwater observed at 0.9m, whilst augering (possible perched water table in filling)
REMARKS: Hand held GPS ± 5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Medowie Christian School
PROJECT: New Science and Technology Building
LOCATION: 6A Waropara Road, Medowie

SURFACE LEVEL: --
EASTING: 392490
NORTHING: 6377667
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 81808.03
DATE: 3/7/2018
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Dynamic Penetrometer Test (blows per 150mm) 5 10 15 20
				Type	Depth	Sample	Results & Comments		
		SANDY CLAY - Very stiff, grey sandy clay with fine to medium grained sand, M>Wp *(continued)*	▨						
			▨	S	5.5 5.8		5,6,12 N = 18 pp = 200-250		
	5.95	Bore discontinued at 5.95m, limit of investigation			5.95				
	6								6
	7								7
	8								8
	9								9

RIG: MD20q **DRILLER:** Sytech Drilling **LOGGED:** West **CASING:** Nil

TYPE OF BORING: Solid flight auger

WATER OBSERVATIONS: Free groundwater observed at 0.9m, whilst augering (possible perched water table in filling)

REMARKS: Hand held GPS ± 5m

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics | Environment | Groundwater

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Results of Dynamic Penetrometer Tests

Client Medowie Christine School
Project New Science and Technology Building
Location 6A Waropara Road, Medowie

Project No. 81808.03
Date 03/07/18
Page No. 1 of 1

Test Location	101	102	103	104	105					
RL of Test (AHD)										
Depth (m)	**Penetration Resistance** Blows/150 mm									
0 - 0.15	2	1	1	3	0					
0.15 - 0.30	2	2	2	4	1					
0.30 - 0.45	3	4	2	5	2					
0.45 - 0.60	4	4	3	5	4					
0.60 - 0.75	3	4	3	7	4					
0.75 - 0.90	3	3	3	11	4					
0.90 - 1.05	4	2	3	11	5					
1.05 - 1.20	6	1	4	13	10					
1.20 - 1.35										
1.35 - 1.50										
1.50 - 1.65										
1.65 - 1.80										
1.80 - 1.95										
1.95 - 2.10										
2.10 - 2.25										
2.25 - 2.40										
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										
3.00 - 3.15										
3.15 - 3.30										
3.30 - 3.45										
3.45 - 3.60										

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Sand Penetrometer ☐

Tested By DJW
Checked By SS

Remarks Ref = Refusal, 24/110 indicates 25 blows for 110 mm penetration

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO
PUBLISHING
BTF 18-2011
replaces
Information
Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870-2011, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume, particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.

In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

Notes

1. Where controlled fill has been used, the site may be classified A to E according to the type of fill used.
2. Filled sites. Class P is used for sites which include soft fills, such as clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soil subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise.
3. Where deep-seated moisture changes exist on sites at depths of 3 m or greater, further classification is needed for Classes M to E (M-D, H1-D, H2-D and E-D).

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

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

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

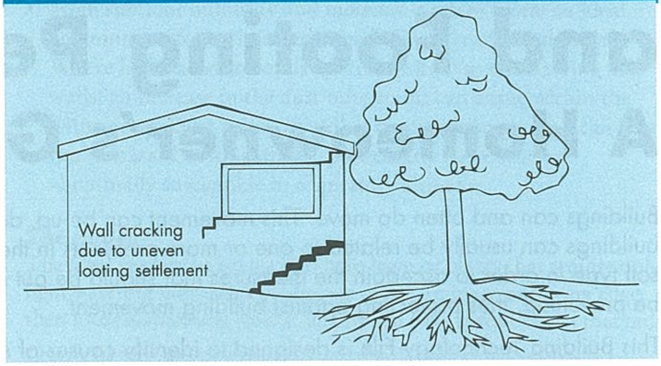
Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends 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.

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.

Trees can cause shrinkage and damage



Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures

Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation causes a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures

Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem. Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870-2011.

AS 2870-2011 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing

Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage

In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

Protection of the building perimeter

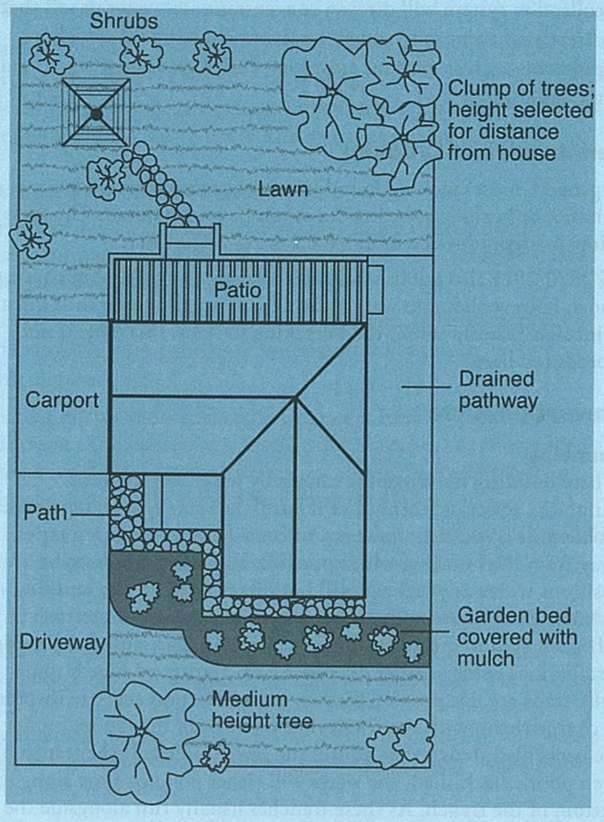
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving should

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly.	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired.	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted.	15–25 mm but also depends on number of cracks	4

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

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Appendix B

Laboratory Test Results

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Newcastle Laboratory
15 Callistemon Close Warabrook Newcastle NSW 2310
Phone: (02) 4960 9600
Fax: (02) 4960 9601
Email: Peter.Gorseski@douglaspartners.com.au

Report Number:	**81808.03-1**
Issue Number:	1
Date Issued:	23/07/2018
Client:	Medowie Christian School 6B Waropara Road, Medowie NSW 2318
Project Number:	81808.03
Project Name:	New Science and Technology Building
Project Location:	6A Waropara Road, Medowie
Work Request:	2223
Sample Number:	18-2223A
Date Sampled:	03/07/2018
Sampling Method:	Sampled by Engineering Department
Sample Location:	**101 (0.5 - 0.68m)**
Material:	CLAY

NATA WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing

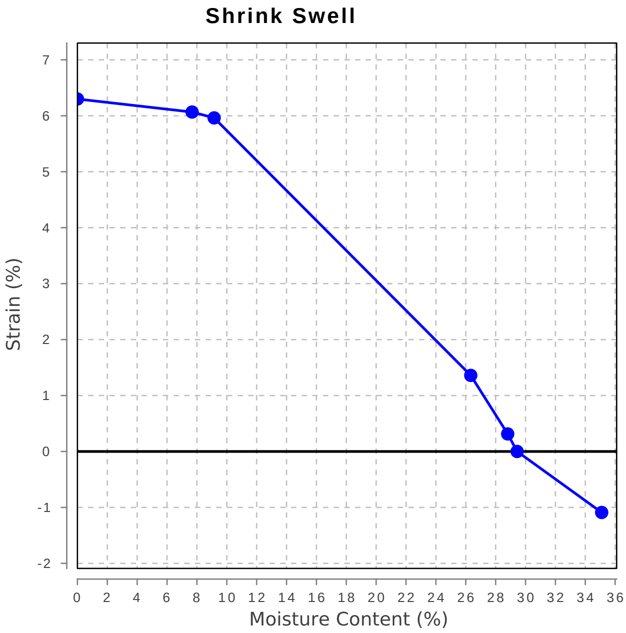
Approved Signatory: Peter Gorseski
Earthworks Manager

NATA Accredited Laboratory Number: 828

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	**3.8**
Visual Description	CLAY
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	**6.3**
Estimated % by volume of significant inert inclusions	0
Cracking	Uncracked
Crumbling	No
Moisture Content (%)	29.4

Swell Test	
Initial Pocket Penetrometer (kPa)	290
Final Pocket Penetrometer (kPa)	210
Initial Moisture Content (%)	32.5
Final Moisture Content (%)	35.1
Swell (%)	**1.1**
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	



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Approved Signatory: Peter Gorseski
Earthworks Manager

NATA Accredited Laboratory Number: 828

Report Number: **81808.03-1**
Issue Number: 1
Date Issued: 23/07/2018
Client: Medowie Christian School
6B Waropara Road, Medowie NSW 2318
Project Number: 81808.03
Project Name: New Science and Technology Building
Project Location: 6A Waropara Road, Medowie
Work Request: 2223
Sample Number: 18-2223B
Date Sampled: 03/07/2018
Sampling Method: Sampled by Engineering Department
Sample Location: **102 (1.7 - 1.9m)**
Material: CLAY

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	30.1

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	85		
Plastic Limit (%)	21		
Plasticity Index (%)	**64**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	17.5		
Cracking Crumbling Curling	Curling		

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CERTIFICATE OF ANALYSIS 196222

Client Details

Client	Douglas Partners Newcastle
Attention	Sasi Sasiharan
Address	Box 324 Hunter Region Mail Centre, Newcastle, NSW, 2310

Sample Details

Your Reference	**81808.03, Medowie**
Number of Samples	2 Soil
Date samples received	13/07/2018
Date completed instructions received	13/07/2018

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.

Report Details

Date results requested by	20/07/2018
Date of Issue	17/07/2018

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Soil Aggressivity			
Our Reference		196222-1	196222-2
Your Reference	UNITS	Bore 101	Bore 102
Depth		1.0-1.45	2.5-2.95
Date Sampled		10/05/2018	30/05/2018
Type of sample		Soil	Soil
pH 1:5 soil:water	pH Units	4.8	5.2
Electrical Conductivity 1:5 soil:water	µS/cm	330	280
Chloride, Cl 1:5 soil:water	mg/kg	410	330
Sulphate, SO4 1:5 soil:water	mg/kg	65	120

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	104	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	96	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	103	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

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.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

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 (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated 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.

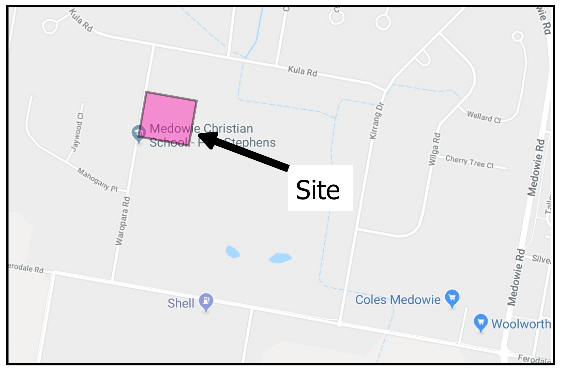
When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Appendix C

Drawing 1 – Test Location Plan

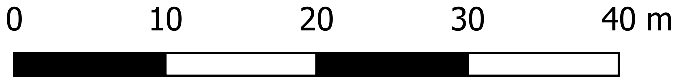


Locality Plan

Legend

⊕ Approximate DCP Location

⊕ Approximate Bore Location



Drawing adapted from Near Map Image, dated 16 February 2018

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Medowie Christian School
OFFICE: Newcastle
DRAWN BY: DJW
SCALE: 1:500 @ A3
DATE: 19.07.2018

TITLE: **Test Location Plan**
New Science and Technology Building
6A Waropara Road, Medowie

N

PROJECT No: 81808.03
DRAWING No: 1
REVISION: 0