Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Investigation

Proposed New Building
75-81 Chelmsford Road, Metford

Prepared for
Paynter Dixon Constructions Pty Ltd

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Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
15 Callistemon Close
Warabrook NSW 2304
PO Box 324
Hunter Region Mail Centre NSW 2310
Phone (02) 4960 9600

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Report on Geotechnical Investigation
Proposed New Building
75-81 Chelmsford Road, Metford

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed new building at Maitland Christian School, located at 75-81 Chelmsford Road, Metford. The investigation was commissioned in an email dated 10 January 2023 from Paynter Dixon Constructions Pty Ltd and was undertaken with reference to Douglas Partners' proposal 102070.02.P.001.Rev0 dated 21 December 2022.

The proposed development will include construction of two new, adjoining two storey buildings. The buildings will replace two, single storey buildings, and will be constructed in two stages. Stage 1 will include replacing the northern-most building, referred to as “Block B”, together with an extension of the adjacent existing car park. Stage 2 will include replacing the southern building, referred to a “Block C”, and connection with the new Block B building.

The purpose of the investigation was to provide the following for the Stage 1 works:

- Subsurface conditions at test locations;
- Depth to groundwater, if encountered during drilling;
- Site classification in accordance with AS2870;
- Geotechnical design parameters for high level and pile footings;
- Earthquake classification in accordance with AS1170.4;
- Design subgrade CBR and pavement thickness design for proposed car parking; and
- Recommended site / subgrade preparation measures.

The investigation included the drilling of five (5) boreholes and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments and recommendations on the items listed above. Although the investigation was targeted to the Stage 1 works, the investigation included positioning of boreholes to inform the Stage 2 works also.

Douglas Partners Pty Ltd (DP) was also engaged to undertake a HAZMAT assessment which was provided in a separate report (102070.02.R.001.Rev0).

2. Site Description and Regional Geology

Maitland Christian School is located on the southern side of Chelmsford Drive in Metford and comprises five main building blocks. The school site covers approximately 2 ha in total.

The area of the proposed development is located within the north-western corner of the site, and is currently occupied by two, single storey brick buildings separated by a large COLA. The northern building is referred to as Block B, and the southern building as Block C.

A small car park is located between Block B and Chelmsford Drive, which is sealed.

The site and relevant buildings are indicated in Figure 1, below. Photographs of the site taken during the field work are shown in Figure 2 to Figure 5.

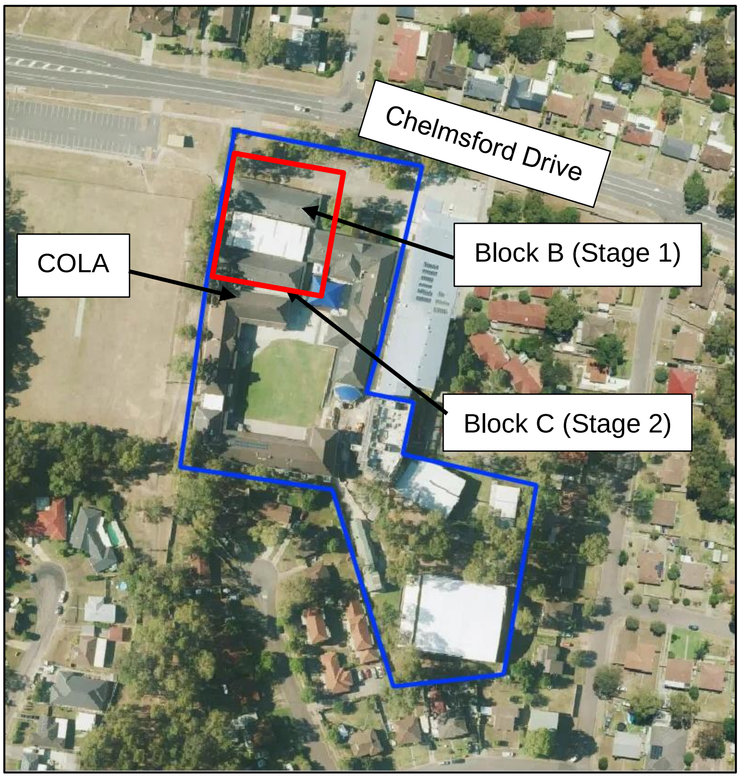

Figure 1: Site location (red line) and the lot boundary (blue line) of Maitland Christian College. Aerial image from Metromap.com.

Figure 2: Existing car park on the north side of Block B, looking north-west

Figure 3: Car park and Block B frontage, looking east. Location of new parking spaces

Figure 4: Rear of Block C, looking north-west. COLA beyond.

Figure 5: COLA, looking east. Block B visible on the left of the view, and Block C on the right.

Reference to the NSW Seamless Geology dataset indicates that the site is underlain by rocks of the Tomago Coal Measures, typically comprising sandstone (sporadically interbedded with laminated to carbonaceous shale), mudstone, siltstone, coal (with sporadic interbeds of carbonaceous shale) and claystone.

3. Previous Investigation

DP has previously carried out two other geotechnical investigations for developments elsewhere in the school, some 80 m to the south-east of the current project, the locations of which are shown in Figure 6.

Figure 6: Site Location – Current project area shown in red, previous investigation shown by yellow dashed line

The 2022 investigation was carried out to inform construction of a new three-storey building comprising drama facilities, a gymnasium and classrooms (DP, 2022). The investigation included drilling of three bores, including coring of the bedrock. The bores encountered a thin layer of fill, underlain by clay soil to depths ranging from 3.7 m to 5.1 m, underlain by rock.

The 2020 investigation was caried out at the adjoining Arise Christian College to inform a proposed single storey building (DP, 2020). The investigation included drilling of five bores to a depth of 2.5 m. The bores encountered residual clays to depths of around 1 m underlain by extremely weathered rock (with hard soil like properties).

4. Field Work

4.1 Methods

The field work was carried out on 23 to 25 January 2023 and comprised the following:

- The drilling of five boreholes (Bores 1 to 5) using a purpose-built, track mounted geotechnical drilling rig. The bores were drilled to depths ranging from 4.3 m to 9.7 m. The bores were drilled using a combination of solid flight auger (TC bit) and rotary methods in the soil and weathered rock profile and NMLC coring of the underlying bedrock, at Bores 1, 2 and 4. Bores 3 and 5 were terminated in the weathered rock;

- In situ testing, consisting of pocket penetrometer tests at selected depths within thin wall samples of cohesive soil strata;
- Photographs of the recovered core from Bores 1, 2 and 4 were taken upon completion of drilling and are presented in the core photoplates in Appendix A;
- Point load testing on recovered rock samples, the results of which are presented on the attached borehole logs;
- Collection of undisturbed soil samples using a 50 mm diameter steel tube for the purpose of assessing shrink-swell soil reactivity;
- The subsurface soil, rock and groundwater conditions were logged by DP personnel, who also recovered representative samples for identification purposes and lab testing;
- Upon completion of drilling, the bores were backfilled using cuttings. The surface was reinstated with concrete or coldmix asphalt, where required; and
- The locations and levels of the bores were obtained using a differential GPS, which is typically accurate to ±0.1 m depending on satellite coverage.

The test locations are shown on the Test Location Plan, Drawing 1, in Appendix C.

4.2 Field Work Results

The subsurface conditions encountered in the boreholes are presented in detail in the attached borehole logs in Appendix A. These should be read in conjunction with the accompanying notes which explain the descriptive terms and classification methods used in the reports. Photographs of the rock core from Bores 1, 2 and 4 are also presented in Appendix A.

A summary of typical conditions and depths to bedrock or refusal if encountered is shown in Table 1 and Table 2.

Table 1: Summary of Geological Units (Bore 1 to 5)

From (m)	To (m)	Unit	Stratum	Description
Surface (0.0)	0.2 / 0.3	1a	Topsoil	Generally comprising silty clay or silty sand with trace organics encountered at Bores 1 to 3.
Surface (0.0)	0.13	1b	Softfall and Concrete	Encountered in Bores 4 and 5
0.13 / 0.3	2.0 / 5.5	2a	Silty Clay	Pale grey, high plasticity silty clay, typically in a very stiff or hard condition. Encountered in all bores Stiff layer encountered in Bore 2 from 0.3m to 2.0m
2.0 / 5.5	3.4 / 5.61	2b	Silty Clay	Pale grey, low plasticity silty clay. Comprising extremely weathered rock with hard soil-like properties, encountered in all bores. Bores 3 and 5 were terminated in this layer
3.4 / 5.61	6.45 / 9.7(LOI)	3	Sandstone	Typically low to medium strength, with some clay seams/ very low strength bands, together with high or very high strength bands. Encountered at Bores 1, 2 and 4 Some high and very high strength bands

Notes to Table 1:
LOI – limit of investigation

Table 2: Summary of Subsurface Conditions

Unit	Subsurface Description	Depth Range Encountered (m)				
		Bore 1	Bore 2	Bore 3	Bore 4	Bore 5
1a	Topsoil	0.0 – 0.2	0.0 – 0.3	0.0 – 0.2	NE	NE
1b	Concrete	NE	NE	NE	0.0 – 0.13	0.0 – 0.13
2a	Silty Clay – very stiff to hard	0.2 – 2.0	0.3 – 5.5	0.2 – 2.0	0.13 – 3.4	0.13 – 4.5
2b	Silty Clay / Silty Sand: Extremely weathered rock	2.0 – 3.4	5.5 – 6.1	2.0 – 4.3 (LOI)	3.4 – 5.2	4.5 – 6.0 (LOI)
3	Sandstone: Low to medium strength.	3.4 – 6.5 (LOI)	6.1 – 9.0 (LOI)	-	5.2 – 9.7 (LOI)	-

Notes to Table 2:
LOI – Limit of investigation
NE – Not encountered

No free groundwater was observed in the bores while they remained open. Groundwater observations were obscured by drilling fluid from 3.4 m, 5.7 m and 5.2 m depth in Bores 1, 2 and 4, respectively.

It should be noted that groundwater levels are affected by factors such as climactic conditions and soil permeability and will therefore vary with time.

5. Laboratory Testing

Geotechnical laboratory testing comprised the following:

- Two shrink-swell index tests; and
- One California bearing ratio (CBR) test.

The detailed results are presented in Appendix B and are summarised in Table 3 and Table 4, below.

Table 3: Results of Shrink-Swell Laboratory Testing

Bore	Depth (m)	Description	Initial Pocket Penetrometer (kPa)	Final Pocket Penetrometer (kPa)	FMC (%)	I_{ss} (% per ΔpF)
1	1.00 – 1.38	Silty Clay	550	150	22.8	5.0
3	1.00 – 1.39	Silty Clay	590	120	25.5	6.5

Notes to Table 3:
FMC – Field Moisture Content
Iss – Shrink/Swell Index

Table 4: Results of Geotechnical Laboratory Testing

Bore	Depth (m)	Description	FMC (%)	SOMC (%)	SMDD (t/m^3)	CBR (%)	Swell (%)
2	0.3 – 0.8	Silty Clay	15.5	19.5	1.62	2.5	1.5

Notes to Table 4:
FMC - Field Moisture Content
SOMC - Standard Optimum Moisture Content
SMDD - Standard Maximum Dry Density
CBR - California Bearing Ratio (4 day soaked)

6. Proposed Development

It is understood that Stage 1 of the proposed development will include the demolition of the existing Block B building (indicated in Figure 1) followed by the construction of a two-storey building with a rooftop playground and a lift structure.

The existing car park just to the north of the Block B will also be expanded as part of the Stage 1 works, with new parking spaces provided abutting the new building on the northern side.

Stage 2 works will include demolition of the existing Block C building and construction of a similar new building, which will be connected to the new Block B building.

Proposed loads of the new buildings have not been provided at the time of writing.

It is understood that the COLA is to be retained.

The concept plan for the proposed new building is shown in Figure 7 below.

Figure 7: Concept plan provided by Paytner Dixon. First floor, Stage 1 shown by pink shading.

7. Comments

7.1 Site Classification

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 the procedures presented in the residential slabs and footings code (AS 2870, 2011), laboratory testing and the typical soil profiles revealed in the boreholes. It is noted that site classification to AS 2870:2011 is not strictly applicable to this site as it is not a residential development. However, the principles of footing design and site maintenance presented therein should be taken into account for buildings such as that proposed for the site.

The results from shrink-swell testing of samples taken from the site returned an I_{ss} value of 5.0% and 6.5% per Δ pF for the residual clay soils. Previous laboratory testing completed by DP (DP, 2020) (DP, 2022) returned values of 2.5 % and 5.0 % per ΔpF.

The site classification of the site is Class P in accordance with AS2870-2011 due to the existing buildings that will need to be demolished prior to construction of the new building, and the subsequent likelihood of abnormal moisture conditions. As a guide however, the range of characteristic surface movements (y_s) is estimated to be commensurate with a Class E-D classification (Extremely Reactive - Deep), in the range of of 75 mm to 135 mm, for footings founded in the natural clay material. This estimate relates to normal seasonal moisture fluctuations without the influence of trees and abnormal moisture conditions.

Site classification, as above, is based on the information obtained from the test bores and on the results of limited laboratory testing and has involved some interpolation between data points.

Articulation joints should be provided within masonry walls in accordance with CCAA (2008) in order to reduce the effects of differential movement. This classification is dependent on proper site maintenance, which should be carried out in accordance with the attached CSIRO Sheet BTF 18 and Appendix B of AS 2870:2011.

7.2 Shallow Footings

It is likely that the loads of the main building will need to be supported on piles. However, any shallow pad footings could be designed for the following allowable bearing pressures:

- Unit 2a - Stiff or stronger clay: 100 kPa;
- Unit 2b - Extremely weathered material: 400 kPa.

It is noted that a significant reduction in shear strength occurred within the shrink-swell samples during the soaking phase.

Footings should not be supported on uncontrolled fill.

Alternatively, footings may be founded in engineered fill placed and compacted under Level 1 earthworks inspection and testing requirements in accordance with the procedures outlined in AS 3798-2007. Footings founded in engineered fill should be proportioned for a maximum allowable bearing pressure of 100 kPa.

7.3 Piles

Conventional bores piles socketed into the bedrock are considered to be suitable for the site. Screw piles could possibly be considered but it is anticipated that installation below around 2 m would not be possible at this site, owing the hard clay / extremely weathered rock.

Bored Piles

The rock mass was classified with respect to Pells et al (Pells, Mostyn, & Walker, 1998) which categorises the bedrock based on defect spacing, unconfined compressive strength (UCS), allowable seams and lithology (sandstone or shale). It is noted that sandstone and shale classified using this system may have an intact rock strength that would satisfy one particular class of rock but defect spacing may not be satisfied and therefore a lower classification would apply.

Based on the results of the investigation and the methods presented in Pells et al (1998), the bedrock encountered at this site has been classified as follows:

Table 5: Classification of Bedrock with reference to Pells, Mostyn, & Walker (1998)

Unit	Description	Class
3	Sandstone: Low to medium strength with some locally higher strength layers clay seams or with very low strength claystone bands up to 400mm	Class IV Sandstone

Table 6 provides the design parameters for rock socketed bored piled for the units presented in Table 5 above. Table 6 presents both ultimate design parameters and serviceability (allowable) parameters for end bearing and shaft adhesion. It should be noted that Pells, Mostyn & Walker (1998) states that “allowable side shear and allowable end bearing stresses are not additive” and therefore pile design using allowable parameters can be conservative.

A geotechnical reduction factor of 0.52 is suggested for preliminary design of piles based on current data and is subject to inspections to be completed during piling operations by a qualified geotechnical engineer. The factor could be increased depending on the redundancy of the footings and type of testing completed during piling operations. A value of 0.40 should be used for pile design if no pile load testing is to be carried out at the time of installation.

Table 6: Pile Design Parameters in Rock

Unit	Description	Depth Encountered (m)			Ultimate Pressures [1,2] ($R_{d,ug}$)		Serviceability/Max Allowable (Based on Working Loads)[4]		Elastic Modulus[7] (E_{field}) (MPa)
		Bore 1	Bore 2	Bore 4	End Bearing (kPa)	Shaft Adhesion[3,5] (kPa)	End Bearing (kPa)	Shaft Adhesion[3,5] (kPa)	
Unit 2b	Silty Clay (extremely weathered material)	2.0 – 3.4	5.5 – 6.1	3.4 – 5.2	1,500	75	500	25	50-100
Unit 3	Sandstone (IV)	3.4 – 6.5 (LOI)	6.1 – 9.0 (LOI)	5.2 – 9.7 (LOI)	8,000	350	2,500	150	200-600

Notes to Table 6:
1 Ultimate Values occur at large settlements (> 5% of minimum footing diameter)
2 Design geotechnical strength ($R_{d,g}$) should be based on a strength reduction factor of $\phi_g = 0.52$
3 Shaft adhesion values based on a shaft roughness of R2 or better
4 Serviceability / Max Allowable end bearing to cause settlement of < 1% of minimum footing dimension or pile diameter
5 (AS 2159, 2009) requires that the contribution of the shaft from ground surface to 1.5 times pile diameter or 1 m (whichever is greater) shall be ignored
6. Inferred by drilling slow progress / refusal
7 A range of values has been given for vertical Young's Modulus (Ev) based on typical published correlations.
8 Additional deeper investigation is required if these parameters are to be adopted in design to confirm the absence of weaker layers below the depth of investigation
NE - not encountered

While the piling code (AS 2159, 2009) requires that the serviceability state should be determined with reference to settlement, experience with rock foundations in the sedimentary rock is that settlement of single piles constructed for the serviceable/allowable pressures given in Table 6 would be expected to be less than 1% of pile diameter but settlement would be greater than 5% of pile diameter for ultimate loads.

For piles in tension, the shaft adhesion parameters should be reduced to 75% of the values presented in Table 6.

It should be noted that the parameters given in Table 6 are for clean sockets and bases only. Specific cleaning buckets and grooving tools should be used in pile construction.

Screw Piles

Screw piles are unlikely to be able to be installed below around 2 m due to the hard clay / extremely weathered rock encountered in each of the bores (Unit 2b).

Notwithstanding the above, where steel screw piles may be designed for a maximum allowable bearing pressure of 700 kPa where founded on weathered bedrock. Shaft adhesion should be ignored. Based on the anticipated loads, it is expected that a pile groups will be required.

Due to the strength of the bedrock, it is considered that the steel screw piles may not be able to penetrate into the rock and therefore the tension capacity would be governed by the strength of the overlying clay.

Care should be taken to not 'over-rotate' the piles and disturb the foundation stratum. The steel needs to last at least as long as the design life of the structure. Consideration should be given to providing additional corrosion protection to the steel pile sections to be located above the water table, to the pile helix and to any sections of the pile likely to be subjected to abrasive conditions during installation or aggressive soil/water conditions in service.

7.4 General

Consideration should be given in design to allowing for the potential of differential settlement if footings are to be supported on materials with a large difference in stiffness (i.e. footings should be all in soil or all in rock).

It is recommended that piles are founded a minimum of 2.5 m below existing surface levels, ie below the depth of the expected shrink swell movements.

Geotechnical monitoring and inspection of drill cuttings or pad footing excavations should be undertaken during construction to confirm design parameters.

7.5 Earthquake Classification

The Site sub-soil class is assessed to be Class C_e – "shallow soil site", with reference to Table 4.1 of AS1170.4.

7.6 Car Park Pavements

7.6.1 Adopted Design Subgrade CBR

The subgrade conditions at the site are anticipated to include silty clay. The results of laboratory testing on one sample of the silty clay indicated CBR of 2.5%. Swell of 1.5% was also recording during testing, indicating moderately expansive clays.

Due to the relatively low CBR vales and the expansive nature of the clay subgrade, it is recommended that a select subgrade layer is included in the pavement thickness design. Select subgrade should be non-expansive and have a CBR of greater than 15%.

Where a select subgrade of 150 mm in included, the pavement should be designed based on effective subgrade CBR of 3.5%.

7.6.2 Assumed Design Traffic Loading

Details regarding the expected traffic loading at each site are not known. In the absence of detailed information, an indicative traffic loading of 1 x 10^4 Equivalent Standard Axles (ESA) has been adopted, which is considered appropriate for car parks.

It is anticipated that the majority of traffic will be light vehicles / cars, with the occasional heavy vehicle, such as minibuses.

The above traffic loadings are indicative only, and should be reviewed as more detailed information on traffic loading becomes available. In particular, the likely number and types of trucks should be confirmed to assess the suitability of the suggested pavement thickness.

7.6.3 Flexible Pavement Thickness Design

Based on the procedures presented in Austroads (2017), the recommended pavement thickness design for the traffic loadings above is as presented in Table 7 below.

Table 7: Flexible Pavement Thickness Design: CBR$\geq$2.5%

Pavement Layer	**Minimum Layer Thickness (mm)**
Wearing Course	2-coat spray seal or AC(1)
Basecourse(2)	200
Subbase(3)	-
Select	150*
Total Minimum Pavement Thickness	350

Notes to Table 7:

(1) Where asphalt is to be used as a wearing course a 7mm - 10 mm prime seal should be placed over the basecourse. 30 mm of AC10 is generally recommended for the above traffic loadings.

(2) If AC is used as a wearing course, the thickness of the base layer may be decreased by the thickness of the AC to maintain the overall total minimum pavement thickness.

(3) The above table combines the base and subbase layer into one basecourse layer.

*Additional select material may be required, subject to geotechnical inspection.

The pavement thickness presented above is dependent on the provision and maintenance of adequate surface and subsurface drainage.

7.6.4 Material Quality and Compaction Requirements

Recommended pavement material quality and compaction requirements are presented in Table 8 below.

Table 8: Material Quality and Compaction Requirements – Flexible Pavements

Pavement Layer	Material Quality	Compaction
Basecourse	Basecourse quality gravel in accordance with TfNSW 3051 or MCC (2014) (e.g. DGB20 or similar)	Compact to at least 98% dry density ratio modified (AS 1289.5.2.1)
Subbase	Subbase quality gravel in accordance with TfNSW 3051 or MCC (2014) (e.g. DGS40 or similar)	Compact to at least 95% dry density ratio modified (AS 1289.5.2.1)
Select Material	Approved granular select material, CBR ≥ 15%	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1)
Subgrade	Silty Clay CBR ≥ 2.5%	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1)

Geotechnical inspections and testing should be performed during construction in accordance with the earthworks guideline (AS 3798, 2007).

7.7 Earthworks

The following procedure is recommended for preparation of the pavement subgrade:

- Excavate to design subgrade level;
- Remove any additional topsoil or deleterious materials, such as existing uncontrolled fill;
- Shape the subgrade to ensure continuous fall towards draining pits;
- Test roll the surface in order to determine any soft zones and assess moisture condition. Moisture contents should be in the range -4% (dry) to -1% (dry) OMC, for pavements where OMC is the optimum moisture content at standard compaction;
- Compact the tyned natural subgrade to 100% Standard. The compacted clay subgrade should be left exposed for a minimum of time prior to placement of pavement layers, to minimise the occurrence of desiccation cracking and/or softening due to weather exposure; and
- If raising of the subgrade level is required, all deleterious material should be removed, and approved granular fill placed in layers not exceeding 300 mm loose thickness (150 mm compacted thickness in the case of select pavement layers) and compacted in accordance with Table 8. Fill placement must be subject to Level 2 inspection and testing, as defined in AS 3798 (2007).

Geotechnical inspections and testing should be performed during construction in accordance with (AS 3798, 2007).

8. References

AS 1170.4. (2007). *Structural Design Actions, Part 4: Earthquake Actions in Australia.* Reconfirmed 2018. Incorporating Amendments 1 & 2: Standards Australia.

AS 2159. (2009). *Piling - Design and Installation.* Standards Australia.

AS 2870. (2011). *Residential Slabs and Footings.* Standards Australia.

AS 3798. (2007). *Guidelines on Earthworks for Commercial and Residential Developments.* Standards Australia.

DP. (2020). *Report of Geotechnical Investigation and Preliminary Waste Classification, Proposed Special Learning Centre, Arise Christian College, Metford.* 102070.00.R.001.Rev0: Douglas Partners Pty Ltd.

DP. (2022). *Report on Geotechnical Investigation, Proposed New Building, 75-81 Chelmsford Drive, Metford.* 102070.01.R.001.Rev0: Douglas Partners Pty Ltd.

Pells, P. J., Mostyn, G., & Walker, B. F. (1998). Foundations on Sandstone and Shale in the Sydney Region. *Australian Geomechanics, No 33 Part 3*, 17-29.

9. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at Maitland Christian School with reference to DP's email proposal dated 21 December 2022 and acceptance received from Paytner Dixon Pty Ltd. The work was carried out under Paynter Dixon's Consultancy Services Agreement (CSA AA13789) with agreed departures dated 10 January 2023. This report is provided for the exclusive use of Paynter Dixon for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

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

This report must be read in conjunction with all of the attached 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 of 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 fill 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 fill may contain contaminants and hazardous building materials.

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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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 Methods

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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 generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. 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	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

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

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

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
Friable	Fr	-

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	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

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;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

Dry (D)	Non-cohesive and free-running.
Moist (M)	Soil feels cool, darkened in colour. Soil tends to stick together. Sand forms weak ball but breaks easily.
Wet (W)	Soil feels cool, darkened in colour. Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- ‘Moist, dry of plastic limit’ or ‘w <PL’ (i.e. hard and friable or powdery).
- ‘Moist, near plastic limit’ or ‘w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- ‘Moist, wet of plastic limit’ or ‘w >PL’ (i.e. soils usually weakened and free water forms on the hands when handling).
- ‘Wet’ or ‘w ≈LL’ (i.e. near the liquid limit).
- ‘Wet’ or ‘w >LL’ (i.e. wet of the liquid limit).

Symbols & Abbreviations

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Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

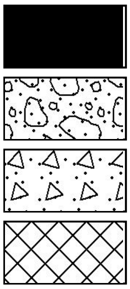
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General



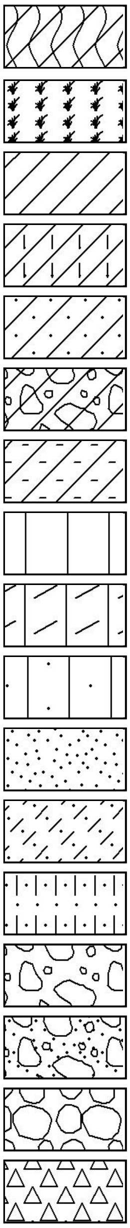
Asphalt

Road base

Concrete

Filling

Soils



Topsoil

Peat

Clay

Silty clay

Sandy clay

Gravelly clay

Shaly clay

Silt

Clayey silt

Sandy silt

Sand

Clayey sand

Silty sand

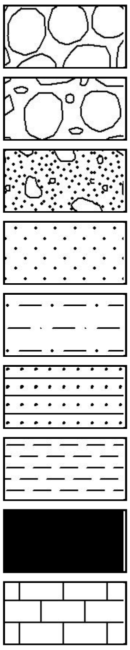
Gravel

Sandy gravel

Cobbles, boulders

Talus

Sedimentary Rocks



Boulder conglomerate

Conglomerate

Conglomeratic sandstone

Sandstone

Siltstone

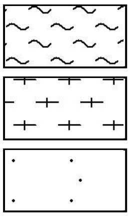
Laminite

Mudstone, claystone, shale

Coal

Limestone

Metamorphic Rocks

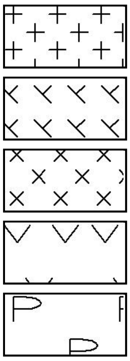


Slate, phyllite, schist

Gneiss

Quartzite

Igneous Rocks



Granite

Dolerite, basalt, andesite

Dacite, epidote

Tuff, breccia

Porphyry

BUILDING | TECHNOLOGY | RESOURCES

FOUNDATION MAINTENANCE AND FOOTING PERFORMANCE

Understanding and preventing soil-related building movement

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

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 home owner 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.

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. Table 1 below is a reproduction of Table 2.1 from Australian Standard AS 2870-2011, Residential slabs and footings.

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 may be the province of the builder and should be taken into consideration as part of the preparation of the site for construction.

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, from AS 2870). 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.

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.

TABLE 1. 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

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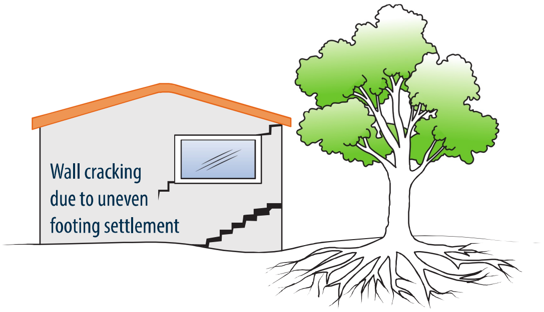

FIGURE 1 Trees can cause shrinkage and damage.

- 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 through absorption. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Shrinkage usually begins on the side of the building 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.

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

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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. Table 2 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 AND 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 may 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 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.

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.

TABLE 2. CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS.

Description of typical damage and required repair	Approximate crack width limit	Damage category
Hairline cracks	<0.1 mm	0 – Negligible
Fine cracks which do not need repair	<1 mm	1 – Very Slight
Cracks noticeable but easily filled. Doors and windows stick slightly.	<5 mm	2 – Slight
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 – Moderate
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 – Severe

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Warning: Although this Building Technology Resource 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, and mould.
- 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.

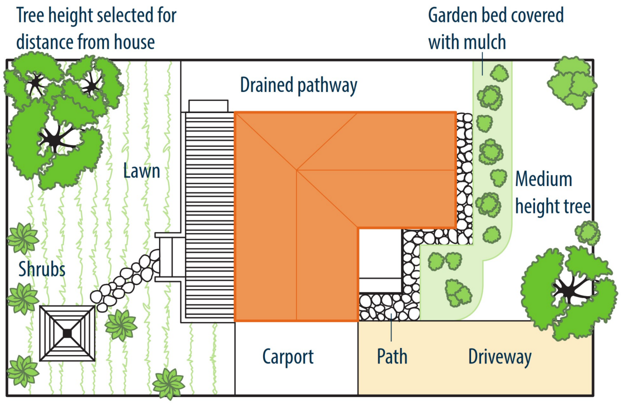

FIGURE 2 Gardens for a reactive site.

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

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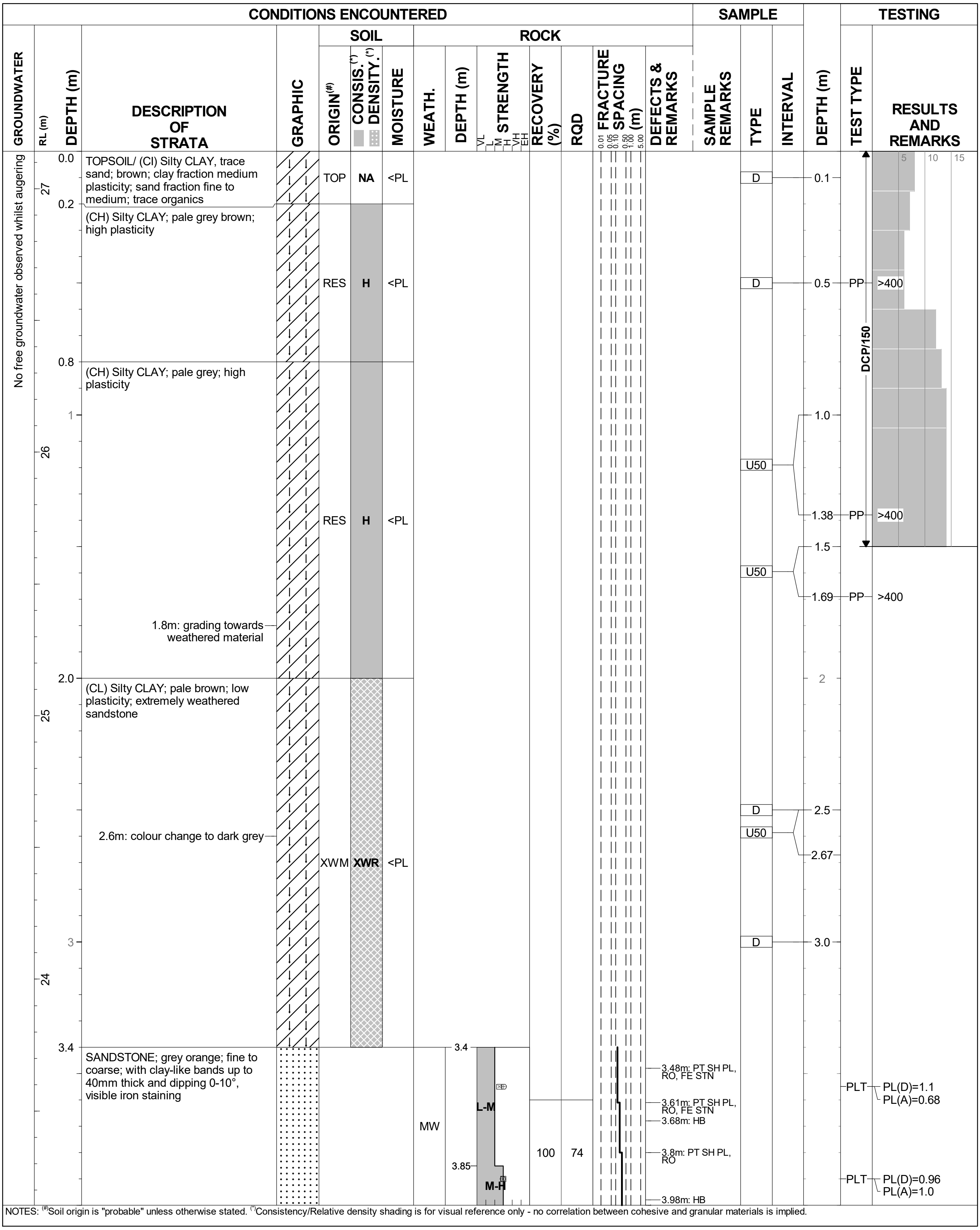
BOREHOLE LOG

CLIENT:	Paynter Dixon Constructions Pty Ltd	**SURFACE LEVEL:**	27.1 AHD	**LOCATION ID:**	1
PROJECT:	Proposed New Building	**COORDINATE**	**E:**369614.9 **N:** 6373797.5	**PROJECT No:**	102070.02
LOCATION:	75-81 Chelmsford Road, Metford	**DATUM/GRID:**	MGA94 Zone 56	**DATE:**	25/01/23
		DIP/AZIMUTH:	90°/---	**SHEET:**	1 of 2



GROUNDWATER: No free groundwater observed whilst augering

DEPTH (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE
0.0	TOPSOIL/ (CI) Silty CLAY, trace sand; brown; clay fraction medium plasticity; sand fraction fine to medium; trace organics	TOP	**NA**	<PL
0.2	(CH) Silty CLAY; pale grey brown; high plasticity	RES	**H**	<PL
0.8	(CH) Silty CLAY; pale grey; high plasticity 1.8m: grading towards weathered material	RES	**H**	<PL
2.0	(CL) Silty CLAY; pale brown; low plasticity; extremely weathered sandstone 2.6m: colour change to dark grey	XWM	**XWR**	<PL
3.4	SANDSTONE; grey orange; fine to coarse; with clay-like bands up to 40mm thick and dipping 0-10°, visible iron staining			

WEATH.	DEPTH (m)	STRENGTH	RECOVERY (%)	RQD	DEFECTS & REMARKS
MW	3.4	**L-M**	100	74	3.48m: PT SH PL, RO, FE STN 3.61m: PT SH PL, RO, FE STN 3.68m: HB 3.8m: PT SH PL, RO
	3.85	**M-H**			3.98m: HB

TYPE	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
D	0.1		
D	0.5	PP	>400
U50	1.0–1.38	PP	>400
U50	1.5–1.69	PP	>400
D	2.5		
U50	2.5–2.67		
D	3.0		
		PLT	PL(D)=1.1 PL(A)=0.68
		PLT	PL(D)=0.96 PL(A)=1.0

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

PLANT: Multidrill 4.0T

OPERATOR: Traccess (Scott Kennedy)

LOGGED: Reiher-Smith

METHOD: Solid flight auger with TC bit to 3.4m, NMLC coring to 6.45m

CASING: HQ to 3.6m

REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

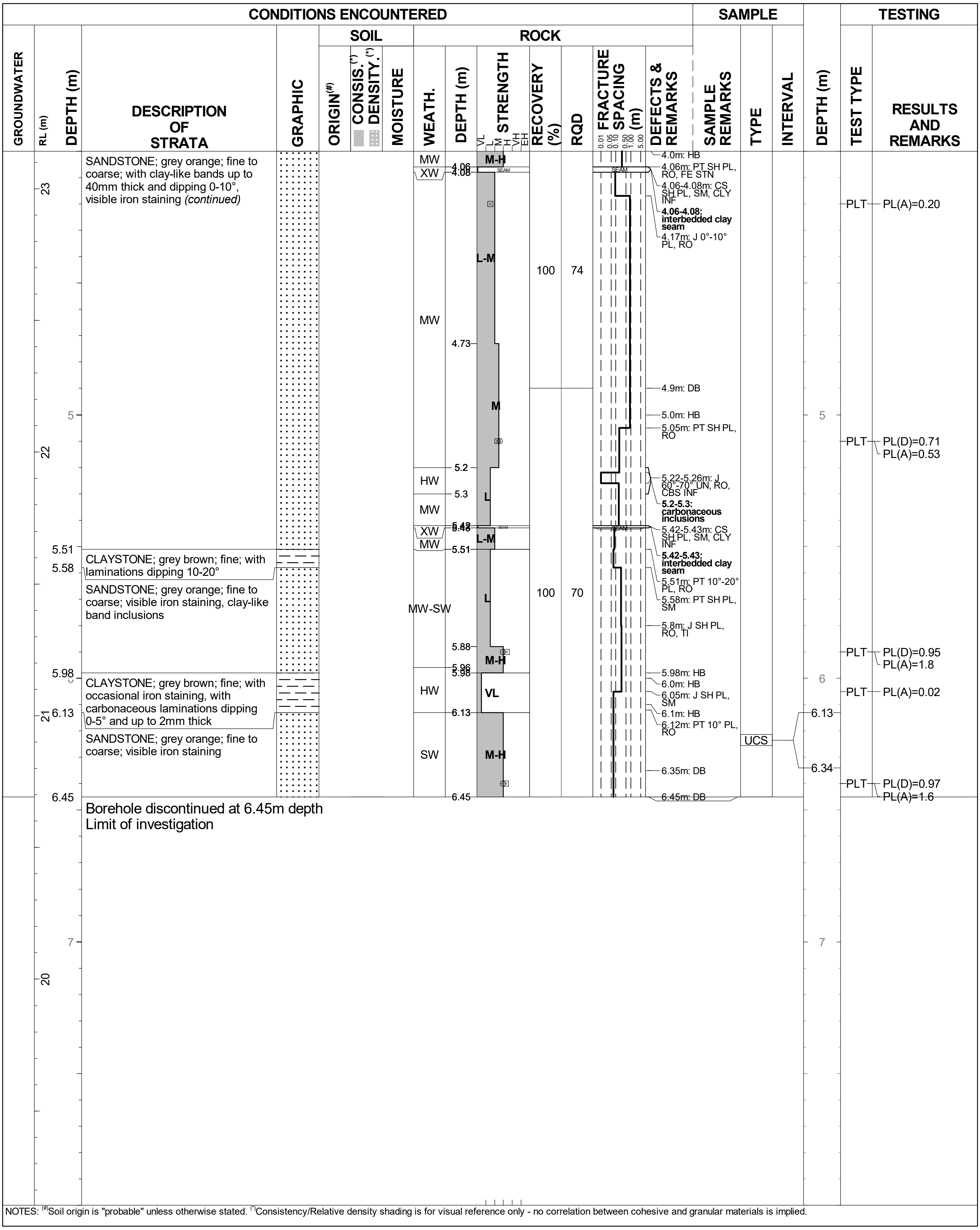
Refer to explanatory notes for symbol and abbreviation definitions

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.1 AHD
COORDINATE E:369614.9 **N:** 6373797.5
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 1
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 2 of 2



DEPTH (m)	DESCRIPTION OF STRATA
	SANDSTONE; grey orange; fine to coarse; with clay-like bands up to 40mm thick and dipping 0-10°, visible iron staining *(continued)*
5.51	CLAYSTONE; grey brown; fine; with laminations dipping 10-20°
5.58	SANDSTONE; grey orange; fine to coarse; visible iron staining, clay-like band inclusions
5.98	CLAYSTONE; grey brown; fine; with occasional iron staining, with carbonaceous laminations dipping 0-5° and up to 2mm thick
6.13	SANDSTONE; grey orange; fine to coarse; visible iron staining
6.45	Borehole discontinued at 6.45m depth Limit of investigation

WEATH.	DEPTH (m)	STRENGTH
MW		M-H
XW	4.06 4.08	
MW		L-M
	4.73	M
	5.2	
HW	5.3	L
MW	5.42 5.43	
XW		L-M
MW	5.51	
MW-SW		L
	5.88	M-H
	5.96 5.98	
HW		VL
SW	6.13	M-H
	6.45	

RECOVERY (%)	RQD
100	74
100	70

DEFECTS & REMARKS

- 4.0m: HB
- 4.06m: PT SH PL, RO, FE STN
- 4.06-4.08m: CS SH PL, SM, CLY INF
- **4.06-4.08: interbedded clay seam**
- 4.17m: J 0°-10° PL, RO
- 4.9m: DB
- 5.0m: HB
- 5.05m: PT SH PL, RO
- 5.22-5.26m: J 60°-70° UN, RO, CBS INF
- **5.2-5.3: carbonaceous inclusions**
- 5.42-5.43m: CS SH PL, SM, CLY INF
- **5.42-5.43: interbedded clay seam**
- 5.51m: PT 10°-20° PL, RO
- 5.58m: PT SH PL, SM
- 5.8m: J SH PL, RO, TI
- 5.98m: HB
- 6.0m: HB
- 6.05m: J SH PL, SM
- 6.1m: HB
- 6.12m: PT 10° PL, RO
- 6.35m: DB
- 6.45m: DB

TYPE	DEPTH (m)
UCS	6.13 6.34

TEST TYPE	RESULTS AND REMARKS
PLT	PL(A)=0.20
PLT	PL(D)=0.71 PL(A)=0.53
PLT	PL(D)=0.95 PL(A)=1.8
PLT	PL(A)=0.02
PLT	PL(D)=0.97 PL(A)=1.6

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

PLANT: Multidrill 4.0T
METHOD: Solid flight auger with TC bit to 3.4m, NMLC coring to 6.45m
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING: HQ to 3.6m

LOGGED: Reiher-Smith

Bore 1 – 3.4 m to 6.45 m

Douglas Partners Geotechnics / Environment / Groundwater	**Core Photoplates** **Proposed New Building** **75-81 Chelmsford Road, Metford**	PROJECT:	102070.02
		PLATE No:	1
		REV:	0
	CLIENT: Maitland City Council	DATE:	24-Mar-23

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.0 AHD
COORDINATE E:369646.5 **N:** 6373790.1
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 2
PROJECT No: 102070.02
DATE: 23/01/23
SHEET: 1 of 3

CONDITIONS ENCOUNTERED

GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE
No free groundwater observed	27	0.0	TOPSOIL/ (CI) Silty Sandy CLAY; brown; clay fraction medium plasticity; sand fraction fine; organics, rootlets	TOP	NA	<PL
		0.3	(CH) Sandy Silty CLAY; pale grey mottled orange brown; clay fraction high plasticity; sand fraction fine	RES	(ST)	<PL
		0.8	(CH) Silty CLAY, trace sand; pale grey pale red; clay fraction high plasticity; sand fraction fine	RES	ST	=PL
	26	1	1.0-1.45m: trace rootlets 1-2mm thick			
	25	2			VST TO H	
	24	3				

ROCK: WEATH. | DEPTH (m) | STRENGTH (VL L M H VH EH) | RECOVERY (%) | RQD | FRACTURE SPACING (m) (0.01 0.05 0.10 0.50 1.00 5.00) | DEFECTS & REMARKS

SAMPLE / TESTING

TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
D		0.1		
D		0.5		
D		1.0	PP	370-450
SPT	1.0-1.45	1.45	SPT	4,5,9 N=14
		2.5	PP	>600
SPT	2.5-2.95	2.95	SPT	14,20,24 N=44
			PP	>600

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

PLANT: Multidrill 4.0T
METHOD: Solid flight auger to 5.7m, NMLC to 9.0m
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING: HQ to 5.5m

LOGGED: Sloan

Refer to explanatory notes for symbol and abbreviation definitions

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.0 AHD
COORDINATE **E:**369646.5 **N:** 6373790.1
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 2
PROJECT No: 102070.02
DATE: 23/01/23
SHEET: 2 of 3

DEPTH (m)	RL (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	WEATH.	DEPTH (m)	STRENGTH	RECOVERY (%)	RQD	FRACTURE SPACING (m)	DEFECTS & REMARKS	SAMPLE TYPE	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
4.0–5.5	23, 22	(CH) Silty CLAY, trace sand; pale grey pale red; clay fraction high plasticity; sand fraction fine *(continued)*	RES	H								SPT	4.0–4.3	SPT	12,20/150
5.5		(CI-CH) Sandy CLAY, with silt; pale grey; clay fraction medium to high plasticity; sand fraction fine; iron staining, extremely weathered sandstone, soil like properties, friable	XWM	XWR							5.7-5.85m: SOIL		5.5	PP	>600
												SPT	5.5–5.65	SPT	19/150
5.85		SANDSTONE; pale grey; iron staining, carbonaceous inclusions			HW	5.85	VL-L	100	70		5.85m: PT 10° UN, RO			PLT	PL(A)=0.05
					XW	6.0–6.06	SEAM			SEAM	6.0-6.06m: CS				
6	21				HW	6.06	VL-L							PLT	PL(D)=0.05 PL(A)=0.08
					XW	6.2–6.25	SEAM			SEAM	6.2-6.25m: CS SH UN, RO				
					MW	6.25	L				6.53m: PT			PLT	PL(A)=0.13
					XW	6.57–6.65	SEAM			SEAM	6.57m: J 30°-50° UN, RO, CLY 6.57-6.65m: CS 6.65m: J SH-40° UN, RO, CLY				
					MW-SW	6.65	L							PLT	PL(D)=0.05 PL(A)=0.21
7	20					7.0	M				6.98m: PT 20° UN, RO, CBS 7.01m: J 50° CBS 7.06m: PT 10°-20° UN, RO, COAL 7.07m: J 30° UN, RO, COAL 7.19m: PT SH UN, RO			PLT	PL(D)=0.38 PL(A)=0.64
											7.34m: PT SH UN, RO, CBS 7.45m: PT SH-30° UN, RO, CBS				
		7.53m: colour change to yellow brown						100	83		7.84m: PT SH PL, RO			PLT	PL(D)=0.16 PL(A)=0.34

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

PLANT: Multidrill 4.0T
METHOD: Solid flight auger to 5.7m, NMLC to 9.0m
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING: HQ to 5.5m

LOGGED: Sloan

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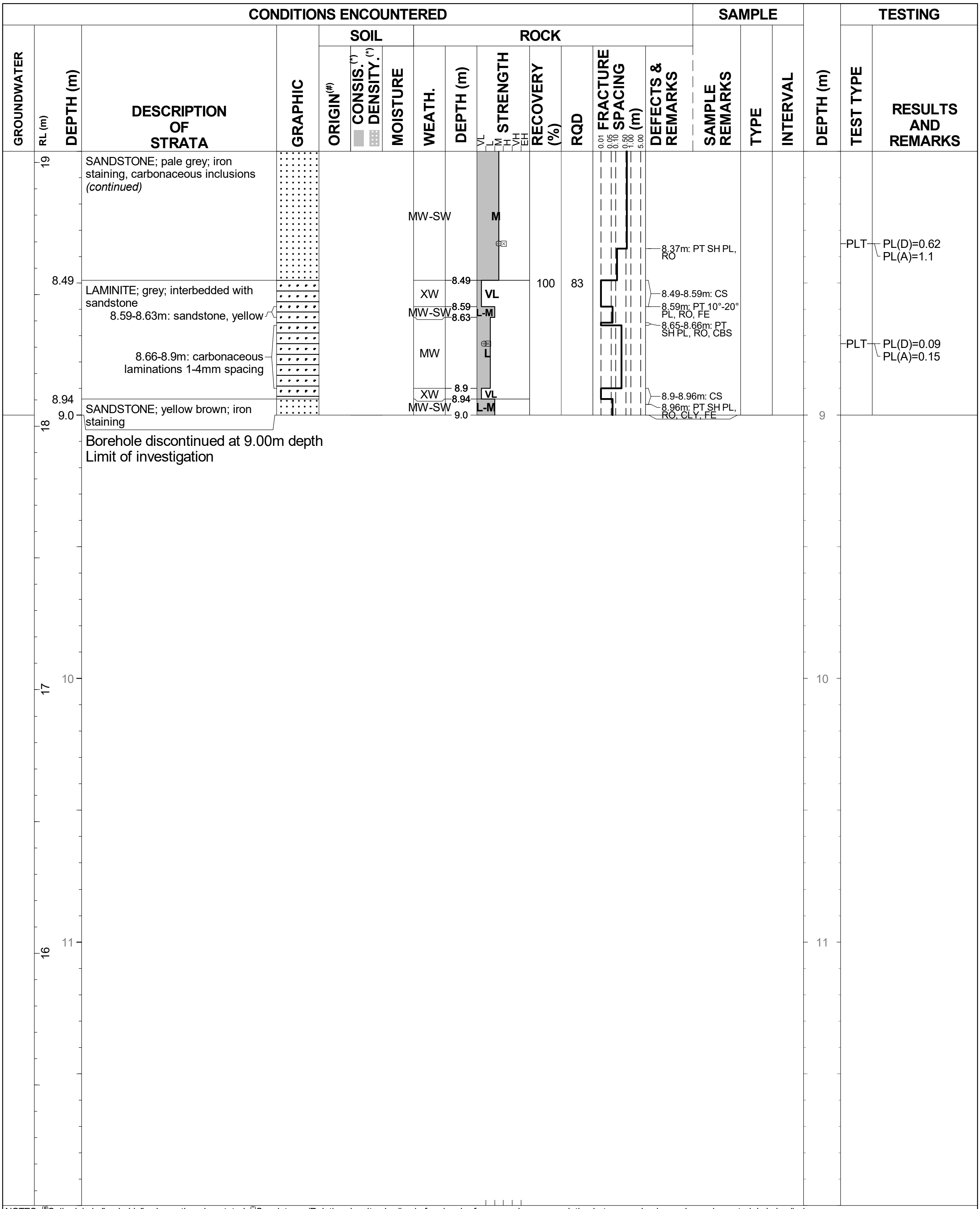
Refer to explanatory notes for symbol and abbreviation definitions

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.0 AHD
COORDINATE E:369646.5 **N:** 6373790.1
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 2
PROJECT No: 102070.02
DATE: 23/01/23
SHEET: 3 of 3



RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	WEATH.	DEPTH (m)	STRENGTH	RECOVERY (%)	RQD	DEFECTS & REMARKS	TEST TYPE	RESULTS AND REMARKS
19		SANDSTONE; pale grey; iron staining, carbonaceous inclusions *(continued)*	MW-SW		M	100	83	8.37m: PT SH PL, RO	PLT	PL(D)=0.62 PL(A)=1.1
	8.49	LAMINITE; grey; interbedded with sandstone	XW	8.49	VL			8.49-8.59m: CS		
		8.59-8.63m: sandstone, yellow	MW-SW	8.59 8.63	L-M			8.59m: PT 10°-20° PL, RO, FE		
		8.66-8.9m: carbonaceous laminations 1-4mm spacing	MW		L			8.65-8.66m: PT SH PL, RO, CBS	PLT	PL(D)=0.09 PL(A)=0.15
			XW	8.9	VL			8.9-8.96m: CS		
	8.94	SANDSTONE; yellow brown; iron staining	MW-SW	8.94	L-M			8.96m: PT SH PL, RO, CLY, FE		
18	9.0	Borehole discontinued at 9.00m depth Limit of investigation		9.0						

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

PLANT: Multidrill 4.0T
OPERATOR: Traccess (Scott Kennedy)
LOGGED: Sloan
METHOD: Solid flight auger to 5.7m, NMLC to 9.0m
CASING: HQ to 5.5m
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

Refer to explanatory notes for symbol and abbreviation definitions

Bore 2 – 5.7 m to 9.0 m

Douglas Partners Geotechnics / Environment / Groundwater	**Core Photoplates** **Proposed New Building** **75-81 Chelmsford Road, Metford**	PROJECT:	102070.02
		PLATE No:	2
		REV:	0
	CLIENT: Maitland City Council	DATE:	24-Mar-23

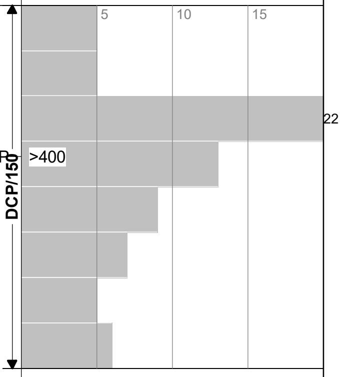
BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.3 AHD
COORDINATE **E:**369653.5 **N:** 6373780.8
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 3
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 1 of 1

CONDITIONS ENCOUNTERED								SAMPLE				TESTING AND REMARKS	
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN$^{(\#)}$	CONSIS.$^{(*)}$ ▨ DENSITY.$^{(*)}$ ▦	MOISTURE	REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
No free groundwater observed		0.0	TOPSOIL/ (SM) Silty SAND, trace gravel; dark brown; sand fraction fine to coarse; gravel fraction sub-angular to sub-rounded; trace organics		TOP	NA	M		D		0.1		
	27	0.2	(CH) Silty CLAY, trace gravel, trace sand; grey brown; clay fraction high plasticity; gravel fraction sub-angular to sub-rounded; sand fraction fine to medium		RES	H	<PL		D		0.5	PP DCP/150	>400
		1							U50		1.0		
	26	1.1	(CH) Silty CLAY; pale grey mottled orange; high plasticity		RES	H	<PL =PL				1.39	PP	>400
	25	2.0	(CL) Silty CLAY, trace sand; pale brown; clay fraction low plasticity; sand fraction fine to medium; extremely weathered sandstone		XWM	XWR	<PL		D		2		
											2.5		
			2.6m: colour change to dark brown grey						U50		2.7	PP	>400
	24	3									3		
		3.5	(SM) Silty SAND; dark brown; fine to coarse; extremely weathered sandstone		XWM	XWR	D						
		4	4.0m: trace orange sand grains						D		4.0		
	23	4.3	Borehole discontinued at 4.30m depth Auger refusal on sandstone bedrock										



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

PLANT: Multidrill 4.0T
METHOD: Solid flight auger with TC bit to 4.3m (auger refusal)
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING:

LOGGED: Reiher-Smith

Refer to explanatory notes for symbol and abbreviation definitions

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.7 AHD
COORDINATE E:369640.3 **N:** 6373766.0
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 4
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 1 of 3

CONDITIONS ENCOUNTERED

GROUNDWATER	DEPTH (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE
No free groundwater observed whilst augering	0.0	Softfill Material		NA	NA
	0.02	FILL/ CONCRETE; grey brown; with aggregates sub-rounded to sub-angular up to 15mm in size, visible steel reinforcement	FILL	NA	NA
	0.13	(CH) Silty CLAY, trace sand; dark brown mottled orange; clay fraction high plasticity; sand fraction fine	RES	ST TO VST	=PL
	0.4	(CI) Silty CLAY; pale grey; medium plasticity 1.5m: colour change to grey brown 2.5m: with red staining	RES	(VST TO H)	<PL
	3.4	(CL) Silty CLAY; grey brown; low plasticity; extremely weathered sandstone	XWM	XWR	

RL (m): 27, 26, 25, 24

ROCK: WEATH. | DEPTH (m) | STRENGTH (VL L M H VH EH) | RECOVERY (%) | RQD | FRACTURE SPACING (m) (0.01 0.05 0.10 0.50 1.00 5.00) | DEFECTS & REMARKS

SAMPLE / **TESTING**

SAMPLE REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
	D		0.3	PP	150-200
	D		0.5		
	D		1.0		
	D		2.0		
	D		3.0		

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

PLANT: Multidrill 4.0T
OPERATOR: Traccess (Scott Kennedy)
LOGGED: Reiher-Smith

METHOD: Diatube coring from 0.0m to 0.13, solid flight auger with TC bit to 5.2m, NMLC coring to 9.67m
CASING: HQ to 5.2m

REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

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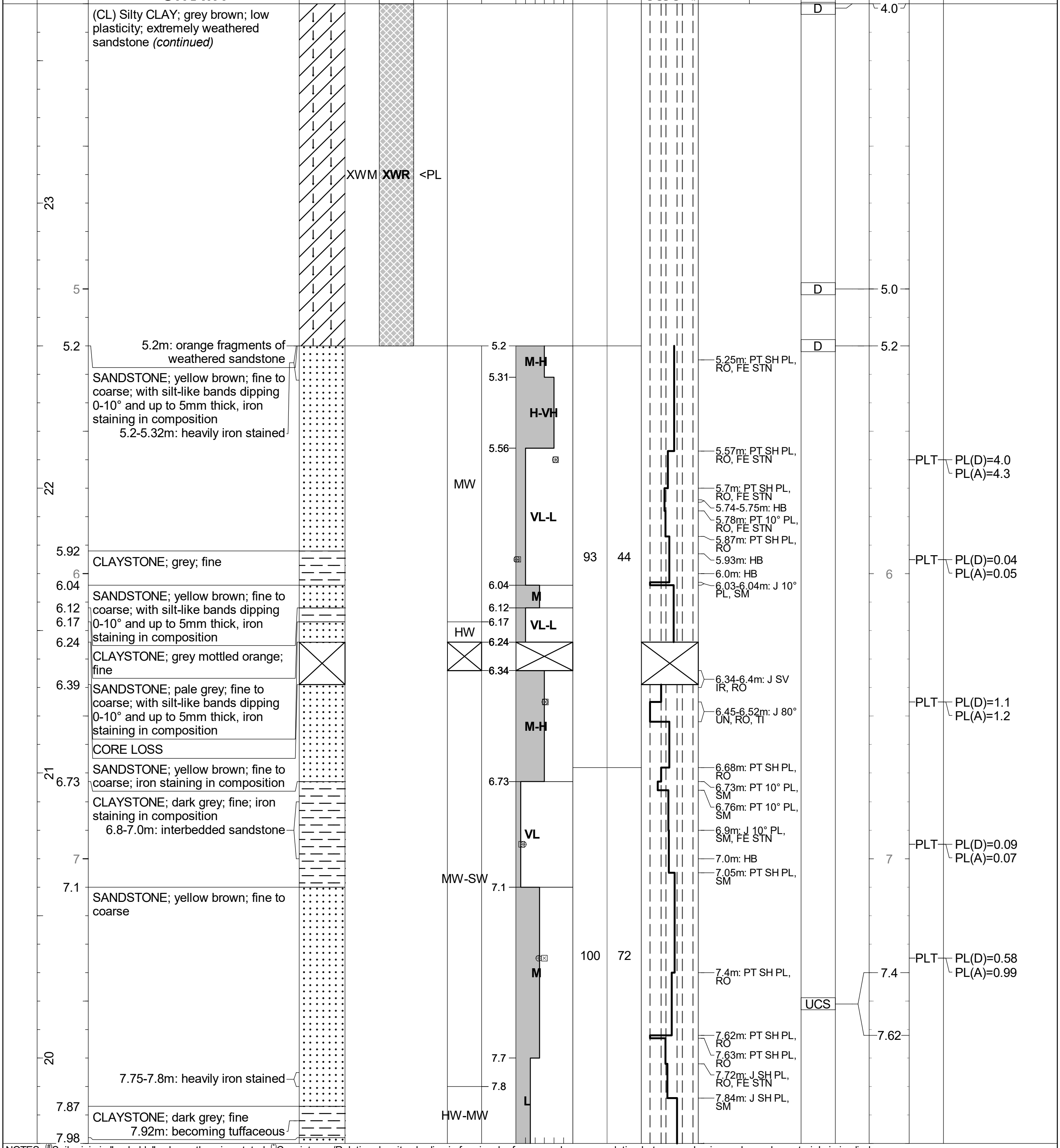
Refer to explanatory notes for symbol and abbreviation definitions

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.7 AHD
COORDINATE E:369640.3 **N:** 6373766.0
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 4
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 2 of 3



RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE	WEATH.	DEPTH (m)	STRENGTH	RECOVERY (%)	RQD	DEFECTS & REMARKS	TYPE	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
23		(CL) Silty CLAY; grey brown; low plasticity; extremely weathered sandstone *(continued)*	XWM	XWR	<PL							D D	4.0 5.0		
	5.2	5.2m: orange fragments of weathered sandstone SANDSTONE; yellow brown; fine to coarse; with silt-like bands dipping 0-10° and up to 5mm thick, iron staining in composition 5.2-5.32m: heavily iron stained				MW	5.2	M-H	93	44	5.25m: PT SH PL, RO, FE STN	D	5.2		
							5.31	H-VH							
22							5.56	VL-L			5.57m: PT SH PL, RO, FE STN 5.7m: PT SH PL, RO, FE STN 5.74-5.75m: HB 5.78m: PT 10° PL, RO, FE STN 5.87m: PT SH PL, RO			PLT	PL(D)=4.0 PL(A)=4.3
	5.92	CLAYSTONE; grey; fine									5.93m: HB 6.0m: HB 6.03-6.04m: J 10° PL, SM		6	PLT	PL(D)=0.04 PL(A)=0.05
	6.04	SANDSTONE; yellow brown; fine to coarse; with silt-like bands dipping 0-10° and up to 5mm thick, iron staining in composition					6.04	M							
	6.12	CLAYSTONE; grey mottled orange; fine					6.12								
	6.17	SANDSTONE; pale grey; fine to coarse; with silt-like bands dipping 0-10° and up to 5mm thick, iron staining in composition				HW	6.17	VL-L							
	6.24	CORE LOSS					6.24								
	6.39	SANDSTONE; yellow brown; fine to coarse; iron staining in composition				MW-SW	6.34	M-H			6.34-6.4m: J SV IR, RO 6.45-6.52m: J 80° UN, RO, TI 6.68m: PT SH PL, RO			PLT	PL(D)=1.1 PL(A)=1.2
21	6.73	CLAYSTONE; dark grey; fine; iron staining in composition 6.8-7.0m: interbedded sandstone					6.73	VL	100	72	6.73m: PT 10° PL, SM 6.76m: PT 10° PL, SM 6.9m: J 10° PL, SM, FE STN 7.0m: HB 7.05m: PT SH PL, SM		7	PLT	PL(D)=0.09 PL(A)=0.07
	7.1	SANDSTONE; yellow brown; fine to coarse					7.1	M			7.4m: PT SH PL, RO	UCS	7.4 7.62	PLT	PL(D)=0.58 PL(A)=0.99
20		7.75-7.8m: heavily iron stained					7.7				7.62m: PT SH PL, RO 7.63m: PT SH PL, RO 7.72m: J SH PL, RO, FE STN				
						HW-MW	7.8	L			7.84m: J SH PL, SM				
	7.87	CLAYSTONE; dark grey; fine 7.92m: becoming tuffaceous													
	7.98														

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

PLANT: Multidrill 4.0T
METHOD: Diatube coring from 0.0m to 0.13, solid flight auger with TC bit to 5.2m, NMLC coring to 9.67m
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING: HQ to 5.2m

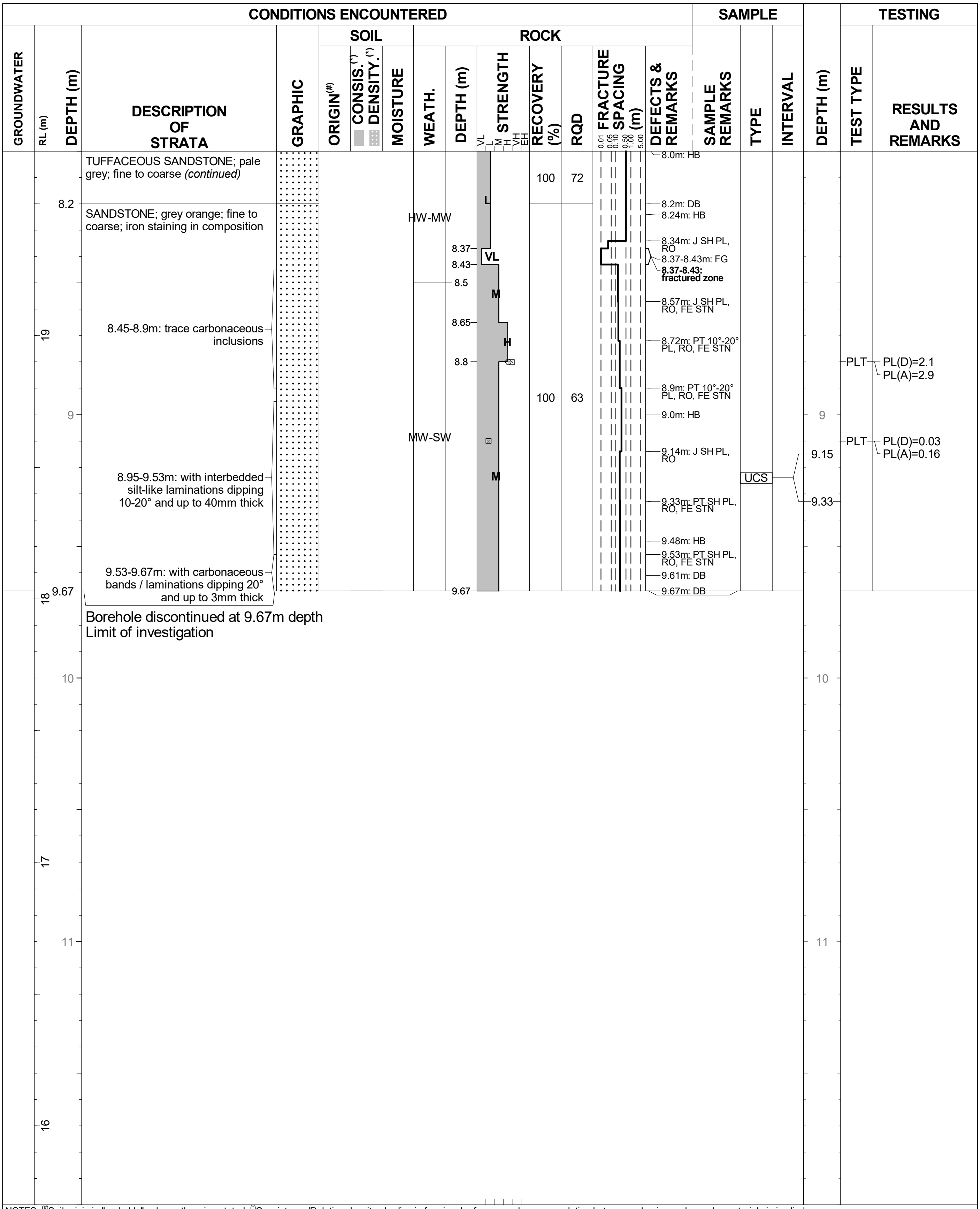
LOGGED: Reiher-Smith

Refer to explanatory notes for symbol and abbreviation definitions

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd	**SURFACE LEVEL:** 27.7 AHD	**LOCATION ID:** 4
PROJECT: Proposed New Building	**COORDINATE** **E:**369640.3 **N:** 6373766.0	**PROJECT No:** 102070.02
LOCATION: 75-81 Chelmsford Road, Metford	**DATUM/GRID:** MGA94 Zone 56	**DATE:** 25/01/23
	DIP/AZIMUTH: 90°/---	**SHEET:** 3 of 3



8.2 — TUFFACEOUS SANDSTONE; pale grey; fine to coarse *(continued)*

9.67 — SANDSTONE; grey orange; fine to coarse; iron staining in composition

8.45-8.9m: trace carbonaceous inclusions

8.95-9.53m: with interbedded silt-like laminations dipping 10-20° and up to 40mm thick

9.53-9.67m: with carbonaceous bands / laminations dipping 20° and up to 3mm thick

Borehole discontinued at 9.67m depth
Limit of investigation

Defects & remarks:

- 8.0m: HB
- 8.2m: DB
- 8.24m: HB
- 8.34m: J SH PL, RO
- 8.37-8.43m: FG
- **8.37-8.43: fractured zone**
- 8.57m: J SH PL, RO, FE STN
- 8.72m: PT 10°-20° PL, RO, FE STN
- 8.9m: PT 10°-20° PL, RO, FE STN
- 9.0m: HB
- 9.14m: J SH PL, RO
- 9.33m: PT SH PL, RO, FE STN
- 9.48m: HB
- 9.53m: PT SH PL, RO, FE STN
- 9.61m: DB
- 9.67m: DB

Testing:

- 8.8 PLT — PL(D)=2.1, PL(A)=2.9
- PLT — PL(D)=0.03, PL(A)=0.16

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

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PLANT: Multidrill 4.0T **OPERATOR:** Traccess (Scott Kennedy) **LOGGED:** Reiher-Smith

METHOD: Diatube coring from 0.0m to 0.13, solid flight auger with TC bit to 5.2m, NMLC coring to 9.67m **CASING:** HQ to 5.2m

REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

Refer to explanatory notes for symbol and abbreviation definitions

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Bore 4 – 5.2 m to 9.67 m
Box 1 of 2

Bore 4 – 5.2 m to 9.67 m
Box 2 of 2

Douglas Partners Geotechnics / Environment / Groundwater	**Core Photoplates** **Proposed New Building** **75-81 Chelmsford Road, Metford**	PROJECT:	102070.02
		PLATE No:	3
		REV:	0
	CLIENT: Maitland City Council	DATE:	24-Mar-23

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.7 AHD
COORDINATE **E:**369613.4 **N:** 6373776.0
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 5
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 1 of 2

CONDITIONS ENCOUNTERED								SAMPLE				TESTING AND REMARKS	
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN[#]	CONSIS.[*] ▨ DENSITY.[*] ▦	MOISTURE	REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
No free groundwater observed		0.02	Softfill Material			NA	NA						
		0.13	FILL/ CONCRETE; grey; with aggregates sub-rounded to sub-angular up to 20mm in size, visible steel reinforcement		FILL	NA	NA						
			(CH) Silty CLAY, with sand; brown grey mottled orange; clay fraction high plasticity; sand fraction fine to medium		RES	VST TO H	<PL =PL		D		0.3	PP	350-550
		0.4	(CL) Silty CLAY, trace sand; pale grey; clay fraction low plasticity; sand fraction fine						D		0.5		
	27												
		1	1.0-1.24m: pale grey mottled orange (iron staining in composition)						D U50		1.0 1.24	PP	>400
	26												
		2	2.0m: colour change to grey brown								2		
					RES	H	<PL						
									D U50		2.5 2.72	PP	>400
	25												
		3									3		
	24												

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

PLANT: Multidrill 4.0T
METHOD: Diatube coring from 0.0m to 0.13m, solid flight auger with TC bit to 6.0m (auger refusal)
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

OPERATOR: Traccess (Scott Kennedy)
CASING:

LOGGED: Reiher-Smith

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Refer to explanatory notes for symbol and abbreviation definitions

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Paynter Dixon Constructions Pty Ltd
PROJECT: Proposed New Building
LOCATION: 75-81 Chelmsford Road, Metford

SURFACE LEVEL: 27.7 AHD
COORDINATE E:369613.4 **N:** 6373776.0
DATUM/GRID: MGA94 Zone 56
DIP/AZIMUTH: 90°/---

LOCATION ID: 5
PROJECT No: 102070.02
DATE: 25/01/23
SHEET: 2 of 2

CONDITIONS ENCOUNTERED								SAMPLE				TESTING AND REMARKS	
GROUNDWATER	RL (m)	DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC	ORIGIN(#)	CONSIS.(*) DENSITY.(*)	MOISTURE	REMARKS	TYPE	INTERVAL	DEPTH (m)	TEST TYPE	RESULTS AND REMARKS
			(CL) Silty CLAY, trace sand; pale grey; clay fraction low plasticity; sand fraction fine *(continued)*		RES	H			D		4.0		
									U50		4.0–4.26		
		4.5	(CL) Silty CLAY, trace sand; dark grey; low plasticity; extremely weathered sandstone		XWM	XWR	<PL						
	23												
		5							D		5.0		
									D		5.5		
	22												
		6.0	Borehole discontinued at 6.00m depth Auger refusal on sandstone bedrock						D		6.0		
	21												
		7									7		
	20												

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

PLANT: Multidrill 4.0T
OPERATOR: Traccess (Scott Kennedy)
LOGGED: Reiher-Smith
METHOD: Diatube coring from 0.0m to 0.13m, solid flight auger with TC bit to 6.0m (auger refusal)
CASING:
REMARKS: Coordinates obtained using differential GPS, typical accuracy ±0.1m.

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Refer to explanatory notes for symbol and abbreviation definitions

Appendix B

Laboratory Testing Results

Material Test Report

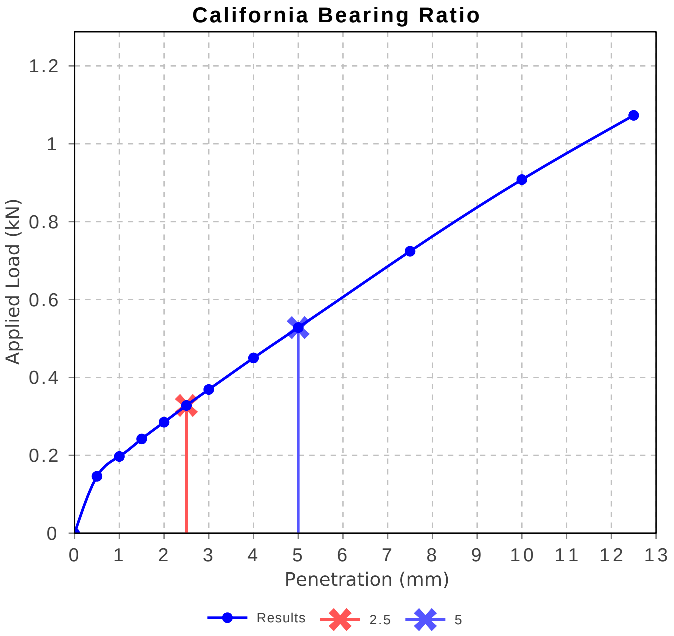
Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Newcastle Laboratory
15 Callistemon Close Warabrook Newcastle NSW 2310
Phone: (02) 4960 9600
Email: Peter.Gorseski@douglaspartners.com.au

Report Number:	**102070.02-1**
Issue Number:	1
Date Issued:	02/03/2023
Client:	Paynter Dixon Constructions Pty Ltd Level 2/2 Richardson Place, North Ryde NSW 2113
Contact:	Clive Furnass
Project Number:	102070.02
Project Name:	Proposed New Building
Project Location:	75-81 Chelmsford Road, Metford NSW
Work Request:	9586
Sample Number:	NC-9586A
Date Sampled:	02/02/2023
Dates Tested:	06/02/2023 - 28/02/2023
Sampling Method:	Sampled by Douglas Partners *The results apply to the sample as received*
Sample Location:	**2, Depth: 0.3 - 0.8m**
Material:	Silty Clay

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Peter Gorseski
Laboratory Manager
Laboratory Accreditation Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**2.5**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)	1.62		
Optimum Moisture Content (%)	19.5		
Laboratory Density Ratio (%)	100.5		
Laboratory Moisture Ratio (%)	99.5		
Dry Density after Soaking (t/m^3)	1.61		
Field Moisture Content (%)	15.5		
Moisture Content at Placement (%)	19.3		
Moisture Content Top 30mm (%)	25.6		
Moisture Content Rest of Sample (%)	19.4		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	189.8		
Swell (%)	1.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0.0		



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Results relate only to the items tested/sampled.

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
Email: Peter.Gorseski@douglaspartners.com.au

Report Number:	**102070.02-1**
Issue Number:	1
Date Issued:	02/03/2023
Client:	Paynter Dixon Constructions Pty Ltd Level 2/2 Richardson Place, North Ryde NSW 2113
Contact:	Clive Furnass
Project Number:	102070.02
Project Name:	Proposed New Building
Project Location:	75-81 Chelmsford Road, Metford NSW
Work Request:	9586
Sample Number:	NC-9586B
Date Sampled:	02/02/2023
Dates Tested:	06/02/2023 - 15/02/2023
Sampling Method:	Sampled by Douglas Partners *The results apply to the sample as received*
Sample Location:	**1, Depth: 1.00 - 1.38m**
Material:	Silty Clay

ilac-MRA NATA

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Peter Gorseski
Laboratory Manager
Laboratory Accreditation Number: 828

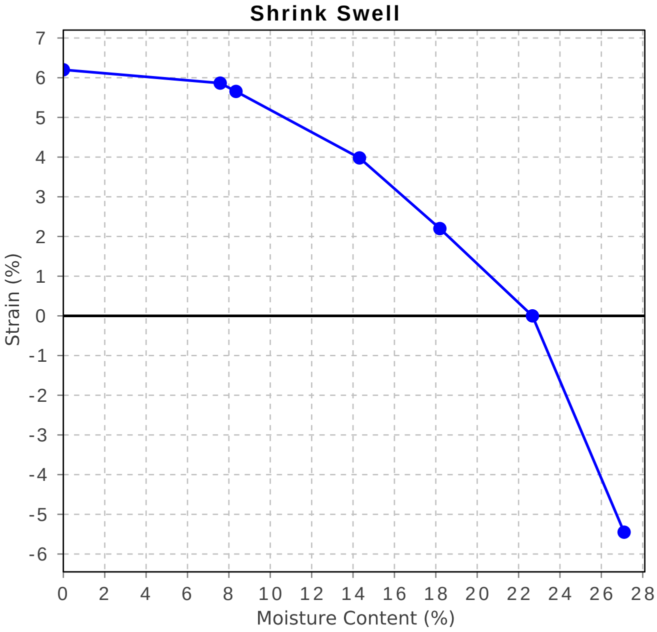
Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	**5.0**
Visual Description	Silty Clay

* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	**6.2**
Estimated % by volume of significant inert inclusions	0
Cracking	Uncracked
Crumbling	No
Moisture Content (%)	22.7

Swell Test	
Initial Pocket Penetrometer (kPa)	550
Final Pocket Penetrometer (kPa)	150
Initial Moisture Content (%)	22.8
Final Moisture Content (%)	27.1
Swell (%)	**5.4**

* NATA Accreditation does not cover the performance of pocket penetrometer readings.



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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
Email: Peter.Gorseski@douglaspartners.com.au

Report Number:	**102070.02-1**
Issue Number:	1
Date Issued:	02/03/2023
Client:	Paynter Dixon Constructions Pty Ltd Level 2/2 Richardson Place, North Ryde NSW 2113
Contact:	Clive Furnass
Project Number:	102070.02
Project Name:	Proposed New Building
Project Location:	75-81 Chelmsford Road, Metford NSW
Work Request:	9586
Sample Number:	NC-9586C
Date Sampled:	02/02/2023
Dates Tested:	06/02/2023 - 15/02/2023
Sampling Method:	Sampled by Douglas Partners *The results apply to the sample as received*
Sample Location:	**3, Depth: 1.00 - 1.39m**
Material:	Silty Clay

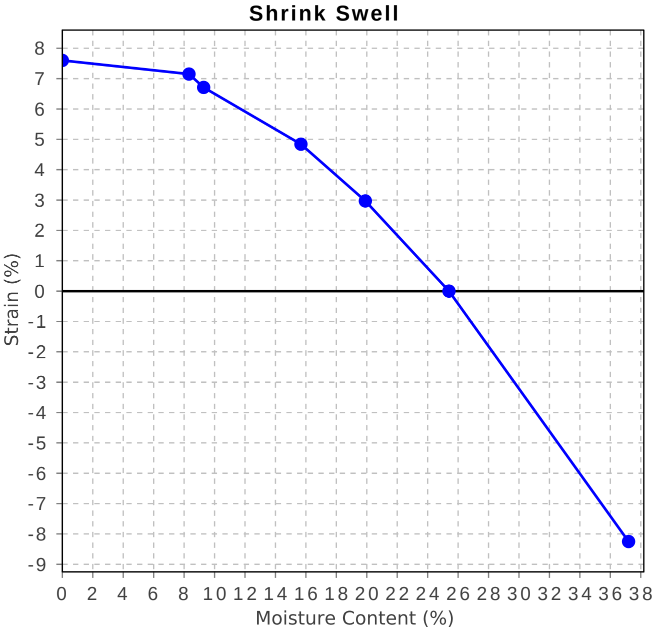
Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Peter Gorseski
Laboratory Manager
Laboratory Accreditation Number: 828

Shrink Swell Index (AS 1289 7.1.1 & 2.1.1)	
Iss (%)	**6.5**
Visual Description	Silty Clay
* Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.	

Core Shrinkage Test	
Shrinkage Strain - Oven Dried (%)	**7.6**
Estimated % by volume of significant inert inclusions	0
Cracking	Uncracked
Crumbling	No
Moisture Content (%)	25.4

Swell Test	
Initial Pocket Penetrometer (kPa)	590
Final Pocket Penetrometer (kPa)	120
Initial Moisture Content (%)	25.5
Final Moisture Content (%)	37.2
Swell (%)	**8.2**
* NATA Accreditation does not cover the performance of pocket penetrometer readings.	



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Results relate only to the items tested/sampled.

Appendix C

Drawing 1 – Test Location Plan

Drawing adapted from Metromap Image, dated 12 June 2022

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Paynter Dixon Constructions Pty Ltd
OFFICE: Newcastle
DRAWN BY: TDM
SCALE: 1:250 @ A3
DATE: 19.01.2023

TITLE: **Test Location Plan**
Proposed New Building
Maitland Christian School, Metford

PROJECT No: 102070.02
DRAWING No: 1
REVISION: 0