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Report on
Geotechnical Investigation

Proposed Budawang School Relocation
17 Croobyar Road, Milton

Prepared for
School Infrastructure New South Wales (SINSW)

Project 89390.02
April 2021

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Document History

Document details

Project No.	89390.02	Document No.	R.001.Rev1
Document title	Report on Geotechnical Investigation Proposed Budawang School Relocation		
Site address	17 Croobyar Road, Milton		
Report prepared for	School Infrastructure New South Wales (SINSW)		
File name	89390.02.R.001.Rev1		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	David Metcalf	Arthur Castrissios	6 November 2020
Revision 1	David Metcalf	Arthur Castrissios	29 April 2021

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Mr Michael Stern, School Infrastructure New South Wales (SINSW)
Revision 1	1	0	Mr Michael Stern, School Infrastructure New South Wales (SINSW)

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

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bsi. ISO 9001 Quality Management
FS 604853

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1. Introduction

This report presents the results of a geotechnical investigation undertaken within part of the former Shoalhaven Anglican College at 17 Croobyar Road, Milton. The investigation was commissioned in an email dated 29 September 2020 by Mr Michael Stern of School Infrastructure New South Wales (SINSW) and was undertaken in accordance with Douglas Partners' proposal WOL200347 dated 6/08/2020.

It is understood that the site is proposed to be re-developed for the potential relocation of the Budawang School. The proposed development of the site includes the demolition of some existing buildings and the construction of new school buildings along with proposed car parks and pavements. Site investigation was undertaken to provide sub-surface information for planning and design purposes for possible submission to Shoalhaven City Council with a Development Application.

The investigation comprised the drilling of boreholes with in-situ testing and sampling followed by laboratory testing of selected samples, analysis and reporting. Details of the work undertaken and the results obtained are given herein, together with comments relating to design and construction practice.

A preliminary masterplan showing the proposed development layout and existing buildings were provided by the client for the investigation. Two proposed (alternate) master plans were provided by the client, following the field work part of the investigation. An additional revised master plan was provided to DP by S.J.A Construction Services Pty Ltd. As requested, the revised master plan has been included in Appendix B of this report.

The investigation was undertaken concurrently with a contamination investigation, the results of which are given in a separate report (Project 89390.02.R.002.Rev0).

2. Background Information

A preliminary geotechnical investigation for a previously proposed development was undertaken by Douglas Partners Pty Ltd (Project 89390.00.R.001.Rev0 dated 15 March 2018). Eight test pits (Pits 1 – 8) were excavated within the grounds of the former Shoalhaven Anglican College as part of the 2018 investigation. Two of the test pits were excavated within the currently proposed development area (refer Drawing 1 in Appendix B). In summary, conditions encountered in these test pits comprised fill to depths of 1.2 m and 0.3 m, respectively, overlying silty clay and sandy clay to depths of 2.9 and 1.7 m, respectively, overlying monzonite bedrock.

3. Site Description

The site, which is part of Lot 200 in Deposited Plan 1192140 is an irregular shaped area of approximately 2.4 hectares with maximum north-south and east-west dimensions of 150 m and 160 m respectively (refer Drawing 1 in Appendix B). It is bounded to the west by Lot 1 in Deposited Plan 811690, to the north by a sewer pumping station and Croobyar Road, to the east by existing low-density residential development with the Princes Highway beyond, and to the south by the remainder of the former grounds of the Shoalhaven Anglican College. Surface levels generally fall in the westerly direction at grades of 1 in 20 to 1 in 90 with some near level terraces associated with the existing school infrastructure. The overall difference in level across the site is estimated to be about 8 m from the highest part of the site (north-eastern part) to the lowest (south-western part).

At the time of the investigation, single storey buildings associated with the former school were located in the northern and southern sections of the site (refer Drawing 1). A north to south trending drainage depression was located along the western boundary. Stands of trees were noted along the eastern and western boundaries and sporadically throughout the remainder of the site. An existing netball/basketball playing surface was located in the western part of the site. Other parts of the site were typically lightly grassed/landscaped (near existing buildings) or asphalt sealed (car parks and driveways). A sewer main diagonally crosses the site (refer Drawing 1).

4. Regional Geology & Acid Sulfate Soils

Reference to web-based mapping indicates that the site is underlain by Milton Monzonite (a medium to coarse grained igneous rock) of Mesozoic age. The results of the field work were consistent with the broad-scale geological mapping with monzonite intersected in the 6 of the 8 boreholes that intersected bedrock.

The NSW Acid Sulfate Soil Risk Map indicates that the site is in an area of *"no known occurrence of acid sulfate materials"* and is about 580 m from the nearest mapped area, which is identified as having a *"low probably of occurrence of acid sulfate materials (Lap2)"*. These low probability areas are noted as *"generally not expected to contain ASS materials, although highly localised occurrences may occur especially near boundaries with environments with a high probability of ASS occurrence"*.

5. Field Work

5.1 Methods

The field work comprised the drilling of eight boreholes (Bores 101 – 108) to depths of 2.2 – 5.5 m using a Kubota KX018-4 mini-excavator fitted with a 150 mm diameter auger attachment. The boreholes were logged on site by a geotechnical engineer who collected disturbed and bulk samples to assist in strata identification and for laboratory testing. Dynamic penetrometer tests (DPT, AS 1289 6.3.2) were undertaken adjacent to selected boreholes to assess the consistency of the upper 1.2 m of the subsurface profile.

The approximate borehole locations are shown on Drawing 1 in Appendix B. The surface levels to Australian Height Datum (AHD) and coordinates to Map Grid of Australia (MGA) were determined using a digital GPS receiver for which a typical accuracy of ±20 mm is expected.

5.2 Results

Details of the subsurface conditions encountered during the field investigation are provided on the borehole logs (refer Appendix B), which should be read in conjunction with the accompanying notes defining classification methods and descriptive terms.

The field work indicated slightly variable subsurface conditions, which were typically consistent with the results of the previous investigation. The succession of strata is broadly summarised as follows:

TOPSOIL / TOPSOIL FILL:	to depths of 0.1 – 0.2 m in all the boreholes;
FILL:	silty gravel fill to a depth of 1.0 m in Bore 101 and possible fill (sandy clay) in Bore 104 to a depth of 0.5 m;
CLAY:	variably firm to hard (but typically stiff to very stiff) clay and sandy clay to depths of 1.9 – 5.5 m in all boreholes. Bores 101 and 104 were terminated in very stiff to hard clays at depths of 5.5 and 4.0 m, respectively;
MONZONITE:	very low strength monzonite to the termination depths (on refusal of the auger) at depths of 2.2 – 4.0 m in Bores 102, 103 and 105 – 108.

The DPT test results recorded blow counts in the range 2 to 9 (but typically 3 to 7) blows per 150 mm of penetration, indicative of variably firm to very stiff in-situ conditions.

Groundwater seepage was observed at depths of 3.5 m, 3.4 m and 2.7 m in Bores 101, 104 and 107 (ie typically within with residual clay profile). No free groundwater was observed in the remaining boreholes during excavation. It is noted however, that the boreholes were immediately backfilled following excavation, sampling and logging which precluded longer term monitoring of groundwater levels. Furthermore, groundwater levels are affected by climatic conditions and soil permeability and will therefore vary with time.

6. Laboratory Testing

6.1 Plasticity

Selected samples were tested in the laboratory for determination of field moisture content, Atterberg limits and linear shrinkage. The detailed laboratory test report sheets are attached, with the results summarised in Table 1.

Table 1: Results of Laboratory Testing – Plasticity

Bore	Depth (m)	W_F (%)	W_L (%)	W_P (%)	PI (%)	LS (%)	Material
102	0.5	36.3	73	20	53	13.0	Clay
103	1.0	40.2	85	23	62	15.5	Clay
105	0.5	36.1	83	27	56	16.0	Clay
108	1.0	23.4	60	23	37	12.5	Sandy Clay

Where: W_F = Field Moisture Content; PI = Plasticity Index; W_P = Plastic Limit; LS = Linear shrinkage; W_L = Liquid Limit

The results indicate that the natural soils are of high plasticity and would be susceptible to shrinkage and swelling movements with changes in soil moisture content. The field moisture contents of the samples tested were up to 17.2 percentage points wet of the plastic limit.

6.2 California Bearing Ratio

Two samples collected from the boreholes were tested in the laboratory for the measurement of field moisture content, compaction and California bearing ratio (CBR). The detailed laboratory test report sheets are attached, with the results summarised in Table 2.

Table 2: Laboratory Test Results (Moisture Content, Compaction and CBR)

Bore	Depth (m)	FMC (%)	OMC (%)	MDD (t/m³)	CBR (%)	Material
103	0.5 – 1.0	39.5	37.0	1.31	2.5	Clay
107	0.5 – 1.0	34.2	34.0	1.35	3.0	Clay

Where: FMC = Field moisture content; OMC = Optimum moisture content; MDD = Maximum Dry Density; CBR = California Bearing Ratio

The results indicate CBR values of the subgrade clays (Bores 103 and 107) were 2.5% and 3.0% when compacted to 100% standard and tested under 4 day soaked conditions. Field moisture contents were in the range 0.2 to 2.5 percentage points wet of standard optimum.

6.3 Soil Aggressivity

Four samples of natural silty clay from Bores 102, 104, 105 and 108 at depths of 0.5 – 1.0 m were tested in a NATA accredited laboratory for measurement of pH, sufates, chlorides and electrical conductivity for assessment of soil aggressivity to concrete and steel. The detailed test report sheets are attached, with the results summarised in Table 3.

Table 3: Results of Laboratory Testing - Aggressivity

Bore	Depth (m)	pH	Chloride (mg/kg)	Sulfate (mg/kg)	EC (µS/cm)	Material
102	0.5	5.2	37	32	68	Clay
104	0.5	5.6	20	72	65	Clay
105	1.0	5.3	21	58	70	Clay
108	1.0	4.9	78	100	130	Clay

The absence of free groundwater in the boreholes and the inferred low permeability of the sampled clay-rich soils indicate that soils at all boreholes are in Condition “B” as defined in AS2159:2009.

The test results indicate that the samples tested can be classified as follows:

- Bore 102 (0.5 m): "mildly aggressive to concrete and non-aggressive to steel";
- Bore 104 (0.5 m): "non-aggressive to concrete and steel";
- Bore 105 (1.0 m): "mildly aggressive to concrete and non-aggressive to steel"; and
- Bore 108 (1.0 m): "mildly aggressive to concrete and non-aggressive to steel".

6.4 Acid Sulfate Soils

Two samples of fill collected from Bore 101 (0.5 m and 1.0 m depth) were tested in the DP laboratory for measurement of pH in H_2O (pH_F) and pH after oxidation with H_2O_2 (pH_{FOX}) using a calibrated pH meter.

The detailed results of the screening tests (pH_F and pH_{FOX}) are attached and are summarised below:

- The pH of the samples in H2O (pH_F) were 6.4 and 6.9 for the samples of fill from Bore 101 at depths of 0.5 m and 1.0 m, respectively; and
- The pH of the samples following addition of H2O2 (pH_{FOX}) were 6.5 and 5.8 for the samples of fill from Bore 101 at depths of 0.5 m and 1.0 m, respectively.

The field results did not include any positive indicators of acid sulfate conditions and as such, Net Acidity (Chromium Suite) testing was not considered necessary.

7. Proposed Development

It is understood that the development is still in a conceptual planning and design phase and as such, design details are yet to be provided. Based on the preliminary masterplans provided by the client, the proposed development will comprise the demolition of some existing buildings, followed by the construction of up to seven, single storey school buildings including new car parks and pavement areas. Proposed building locations and car park/pavement areas (based on the provided Masterplans, F1 and F2 dated 7 October 2020) are shown on Drawings 1 in Appendix B.

Although design details are yet to be finalised, bulk earthworks are expected to be minimal, with finished levels expected to near to existing levels. Excavations to depths of up to about 1 m are anticipated for removal of existing buildings and foundations, the construction of new foundations and the installation of services. As the design is in a preliminary planning phase, no loading information has been provided by the client. However, based on the expected single storey masonry construction, design loads are expected to be commensurate with typical residential construction.

8. Comments

8.1 General

It is understood that the project is in a preliminary planning and development phase. As such, geotechnical investigation was undertaken to provide comments with respect to reactivity, site classification, site preparation, earthworks and foundations, acid sulfate soils and pavements and for possible submission to Shoalhaven City Council with a Development Application. Once the design details (and structural loads) have been finalised, clarification of the comments provided within should be sought from DP and a revision to this report may be required.

8.2 Geotechnical Site Model

Based on the results of the investigation, the inferred subsurface geotechnical model underlying the site comprises:

- A surficial layer of topsoil fill over most of the site;
- A fill platform in the north-western part of the site, probably associated with re-profiling of the drainage depression near the western boundary of the site and possibly backfilling of the existing sewer main (refer Drawing 1 in Appendix B for location);
- Residual clays of firm to hard (but typically stiff to very stiff) consistency underlying the existing topsoil/fill. Clays in the western part of the site (ie near the drainage depression and proposed HS Admin building, refer Drawing 1 in Appendix B) were typically firm and wet of standard optimum and would be expected to quickly lose strength when exposed to construction traffic or inclement weather;
- Weathered rock (monzonite) underlying the residual clay with rock depth increasing from the south-eastern section (near Bore 8) to the western section (near Bores 101 and 104); and

- The groundwater table typically in excess of the investigation depths of 2.2 – 5.5 m, but variable and subject to preceding climatic conditions, particularly in the western part of the site, where clays were more water charged. Moderate seepage within the sandy clays and underlying weathered monzonite.

8.3 Seismic Considerations

Earthquake Hazard Maps published by the Australian Geological Survey Organisation are reproduced in AS1170.4:2007. The anticipated peak ground acceleration or acceleration coefficient for the Milton area is quoted as 0.8 m/sec^2 or 0.08 g. Furthermore, based on a comparison of the soil profile encountered during the field testing with those included in AS21170.4:2007, it is suggested that a Class C_e classification be adopted for design purposes.

8.4 Site Classification

Due to the presence of uncontrolled filling (in parts) the site would be currently classified as Class P in accordance with the requirements of AS2870:2011. The principal requirement for a Class P site is for design to be undertaken by a suitable qualified engineer using design principles that take into account the subsurface conditions.

Notwithstanding the P classification, the natural silty clay subsurface conditions in the vicinity of the proposed buildings (refer Drawing 1 in Appendix B for proposed building locations) are expected to comprise natural silty clays with characteristic surface movements commensurate with Class H2 (highly reactive) conditions from the reactivity viewpoint, when assessed in accordance with the requirements of AS 2870:2011.

It is noted however, that the above classifications are appropriate for the undeveloped site and are independent of proposed earthworks. Furthermore, reference to Clause 3.1.1 of the Code indicates that the footing details given within the code may not be appropriate for non-residential type structures. The main requirement is therefore for designs to be undertaken by a suitably qualified engineer using engineering principles which take into consideration the site conditions. Reference should be made to Section 8.9 for advice relating to foundation design.

8.5 Acid Sulfate Soils

Reference to the risk mapping published by NSW Department of Land and Water Conservation indicates the site is not in a mapped area of acid sulfate soils (ASS) and is about 580 m from the nearest mapped area. The site is underlain by residual soils and is above the level below which estuarine acid sulfate soils are typically encountered (RL12 m AHD). However, given the presence of fill of unknown origin, acid sulfate screening of disturbed samples of existing fill collected from Bore 101 was undertaken.

The results of the initial screening tests for pH following addition of H_2O_2 (pH_{fox}) of the samples of fill from Bore 101 indicated the pH values of 6.5 and 5.8, with pH drops of 0.0 and 1.1 when compared to respective pH in distilled water (pH_F) values. Initial pH (pH_f) values of ≤ 5.5 or final pH (pH_{fox}) values of ≤ 4.5 were not observed in any of the samples tested. The 'field' pH test results (refer Appendix C) indicated that the existing fill did include positive indicators of acid sulfate conditions. As such, Net Acidity (Chromium Suite) testing was not considered necessary.

In summary, the site is underlain by residual clays and some minor existing fill. The results of the limited testing of the fill encountered in Bore 101 indicated the existing fill was unlikely to include soils consistent with acid sulfate conditions. Similarly, the underlying residual clays are derived from weathering of the bedrock and hence are inconsistent with the formation of estuarine acid sulfate soils. As such, management of potential acid sulfate soils is not required for the proposed development.

8.6 Earthworks & Site Preparation

Site preparation should include the removal of topsoils and other deleterious materials (such as existing pavements and buildings, including foundations) from the proposed construction areas. In areas that require filling, all stripped surfaces should be test rolled in the presence of the geotechnical consultant. Any areas exhibiting significant deflections under test rolling should be appropriately treated by over-excavation and replaced with low plasticity fill placed in near horizontal layers no thicker than 250 mm compacted thickness. Each layer should be compacted to a minimum dry density ratio of 98% relative to standard compaction with placement moisture contents maintained within 2% of standard optimum. In pavement areas, the upper 0.5 m thickness of fill and the subgrade should be compacted to achieve at least 100% standard dry density ratio.

To validate site classifications, sufficient field inspections and in-situ testing of future earthworks should be undertaken in order to satisfy the requirements of a Level 1 inspection and testing service as defined in AS 3798:2007.

Notwithstanding the above, based on the results of the field work and laboratory testing, the upper clays in the north-western part of the site are wet of optimum and as such, the inclusion of bridging layers will probably be needed for pavement construction. As a guide, a bridging layer of 300 – 500 mm thickness will most likely be required for car parks and pavements in the vicinity of Bores 101 and 104. The need (or otherwise) for bridging layers will be dependent on the design subgrade level relative to existing, and is best determined on site following boxing to design levels.

Where the presence of deep, weak upper clays precludes removal and replacement, necessitating the adoption of thicker initial fill layers (which could occur, for example, near Bores 101 and 104), design of structures will need to include piers to the underlying stiff clays or weathered rock together with suspended slabs. This may affect the proposed HS Admin building, for example.

Prevailing weather conditions at the time of construction and the control that can be exercised over construction traffic will be critical in achieving satisfactory subgrade performance. If construction does not immediately follow subgrade preparation (thus exposing the subgrade to weather and traffic), subgrade deterioration should be expected, thus requiring rectification. In conjunction with the subgrade preparation procedures, consideration should also be given to installing temporary drainage systems prior to installation of the final works to mitigate the effects of inclement weather.

8.7 Batters

Short-term construction excavation batters could be designed based on 1H:1V for residual clays and weathered rock (if encountered), subject to inspection during construction and water seepage. Detailed assessment of excavation conditions and batter slopes should be undertaken once civil design works have progressed to the preliminary stage. Downslope batters (where proposed) for permanent filled embankments should be graded to a maximum gradient of 3H:1V and be protected by upslope and toe drains. Provision should also be made for the early grassing of all battered slopes in soil to assist in minimising erosion.

8.8 Retaining Structures

It is suggested that active earth pressures on cantilever or gravity retaining walls due to the retained soils be estimated using a triangular pressure distribution calculated as follows:

$$\sigma_z = \gamma.Ka.z$$

where	σ_z	=	horizontal pressure at depth z
	γ	=	unit weight of retained soil
		=	20 kN/m^3 for stiff clay and compacted fill
	Ka	=	active earth pressure coefficient
		=	0.3 for stiff clay and compacted fill (horizontal backfill only)
		=	0.15 for low strength monzonite

Design of retaining walls should make allowance for all superimposed or surcharge loads that will occur. Drainage must be provided behind the walls, or alternatively full hydrostatic pressure allowed for the design. In the event that hydrostatic pressures are allowed, densities of the retained soils can be reduced to the buoyant values.

Footing design could be based on an allowable base bearing pressure of 100 kPa on at least stiff clays or controlled fill and 500 kPa for weathered monzonite.

8.9 Foundations

Based on preliminary design details provided, the proposed buildings are expected to be of single-storey, relatively lightweight construction with structural loads likely commensurate with typical residential dwellings. As such, the construction of a high-level footing system (i.e. following site preparation in accordance with Section 8.6) would be appropriate.

High level footings founded on the existing at least stiff natural clays or controlled filling could be proportioned for an allowable base bearing pressure of **100 kPa.** The feasibility of the use of a high-level footing system would be subject to the proposed loads and resultant settlements. For isolated pad footings with loads of up to say 300 kN per pad, a 1.4 m square pad would be required. Settlements under these pads would be expected to be up to about 15 – 20 mm, with differential settlements of about half the total value expected.

If higher loads or settlement sensitive structures are proposed (i.e. if estimated settlements are beyond tolerable limits) then a deep foundation system (such as bored piers, pre-formed steel screw piles or driven timber piles) founded in the underlying weathered rock (at depths of 1.9 – 3.5 m over most of the site, but deeper than the termination depths of 5.5 and 4.0 m in the vicinity of Bores 101 and 104, based on the results of the investigation) would be required.

An allowable bearing pressure of **500 kPa** would be appropriate for very low and low strength monzonite. The principal advantage of a deep foundation system is that settlements (both total and differential) would be negligible. Higher bearing pressures in the low strength or greater monzonite could be possible but would be subject to the results of additional subsurface investigation (ie drilling including coring of the underlying rock). It is noted that the monzonite can rapidly increase in strength (to very high strength or greater).

Particular note is made for the requirement for footings to found below the zone of influence of the existing sewer main (refer Drawing 1 for approximate location) or on low strength rock, whichever occurs first. Reference must be made to Shoalhaven Water with respect to any specific design requirements that may be above the minimum geotechnical requirements.

Where cut and fill benches are proposed for individual structures, localised deepening of footings will be necessary to ensure uniform bearing is achieved. In particular, if partial rock foundations are exposed, then all footings must found on rock.

Footings should also be inspected by a suitably qualified geotechnical engineer prior to the placement of steel and of concrete to confirm the appropriateness of the bearing stratum for the adopted design pressures. Care must be exercised to ensure that DA Consent conditions are satisfied with respect to footing inspections, and the engineering discipline to undertake the inspections is appropriate (for example, a structural engineer cannot be used as a substitute for a geotechnical engineer).

Articulation should be included within masonry sections to allow for some differential movement without causing structural damage.

8.10 Car Parks and Pavements

Following site preparation in accordance with Section 8.6 (and based on the results of the subsurface investigation and allowing for some variability in subgrade conditions) a design CBR of 2% is considered appropriate for the natural clay fill subgrades. Surface and subsurface drainage should be installed and maintained to protect the pavements and subgrade. Further comments on pavement drainage are given in Section 8.11.

8.11 Site Maintenance and Drainage

Subsurface drainage should be provided to minimise moisture ingress into the pavement materials and subgrade areas. For pavement areas, it is suggested that subsurface drains, constructed with an invert level at least 0.5 m below subgrade level, be installed where appropriate (but around the perimeter as a minimum). This aspect should be reviewed on site during construction and should take into consideration the significance of other engineered drainage works proposed for the project. It should be noted that if the sub-base is of low permeability relative to the base layer, then the subsurface drain is required to intersect all pavement layers.

Subsurface drainage is especially required in pavement areas adjacent to lawns or garden beds, where the ingress of water into the pavement subgrade is likely. Preparation of the subgrade surface should be such that crossfalls for the surface drainage purposes are achievable across the final pavement.

Regular and on-going maintenance of the pavements, such as sealing of joints and surface cracks, will be required to minimise the potential of water ingress that could cause subgrade saturation and premature pavement failure.

All collected stormwater, groundwater and roof runoff from lots should be discharged into the stormwater disposal system. Similarly, effluent flows should be directed to the sewerage system.

The site should be maintained in accordance with the CSIRO publication "*Foundation Maintenance and Footing Performance: A Homeowners Guide*", a copy of which is attached. Whilst is must be accepted that minor cracking in most structures is inevitable, the guide describes suggested site maintenance practices aimed at minimising foundation or floor slab movement to keep cracking within acceptable limits.

9. Summary

A geotechnical investigation has been undertaken for the proposed Budawang School Relocation project at 17 Croobyar Road, Milton. The investigation comprised borehole drilling followed by laboratory testing of selected samples, engineering analysis and reporting.

The principal geotechnical items that need to be considered are as follows:

- The site is underlain by monzonite bedrock that typically increases in strength with depth;
- The presence of uncontrolled fill and initially wet near-surface soils in the north-western part of the site;
- The need for design to allow for the presence of an existing sewer carrier main; and
- Existing structures and services that will require demolition and removal prior to site re-development.

Based on the results of the investigation, the site is considered geotechnically suitable for the proposed Budawang School relocation development, with comments given in the report with respect to site preparation measures, likely reactivity site classifications, retaining wall design parameters, footing design parameters and drainage.

10. References

AS 1170.4, *Minimum design loads on structures – Part 4 Earthquake loads*, Standards Australia

AS 2159:2009, *Piling – Design and Installation*, Standards Australia

AS 2870:2011, *Residential Slabs and Footings*, Standards Australia.

AS 3798:2007, *Guidelines for Earthworks for Commercial and Residential Developments*, Standards Australia

11. Limitations

Douglas Partners (DP) Pty Ltd has prepared this report for this proposed Bundawang School relocation project at 17 Croobyar Road, Milton in accordance with DP's proposal WOL200347 dated 6/11/2020 and acceptance received from Mr Michael Stern dated 29/9/2020. The work was carried out under a modified SINSW consultancy agreement (*SINSW00964/20 Budawang SSP Geotech Consultancy* dated 28 September 2020. This report is provided for the exclusive use of School Infrastructure New South Wales (SINSW) 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 assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report. This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

Introduction

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

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

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

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

Douglas Partners

Description and Classification Methods

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

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

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

Transported soils may be further subdivided into:

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

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

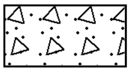
Symbols & Abbreviations

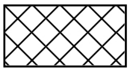
Graphic Symbols for Soil and Rock

General

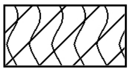
 Asphalt

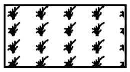
 Road base

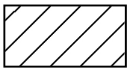
 Concrete

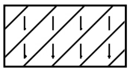
 Filling

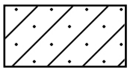
Soils

 Topsoil

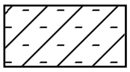
 Peat

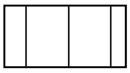
 Clay

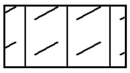
 Silty clay

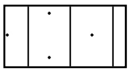
 Sandy clay

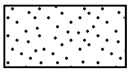
 Gravelly clay

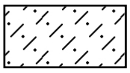
 Shaly clay

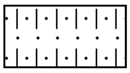
 Silt

 Clayey silt

 Sandy silt

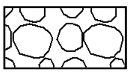
 Sand

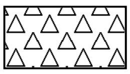
 Clayey sand

 Silty sand

 Gravel

 Sandy gravel

 Cobbles, boulders

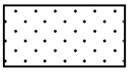
 Talus

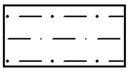
Sedimentary Rocks

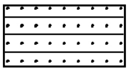
 Boulder conglomerate

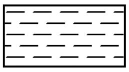
 Conglomerate

 Conglomeratic sandstone

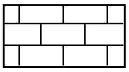
 Sandstone

 Siltstone

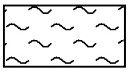
 Laminite

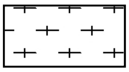
 Mudstone, claystone, shale

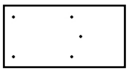
 Coal

 Limestone

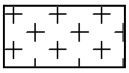
Metamorphic Rocks

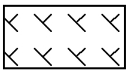
 Slate, phyllite, schist

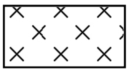
 Gneiss

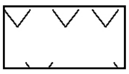
 Quartzite

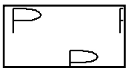
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

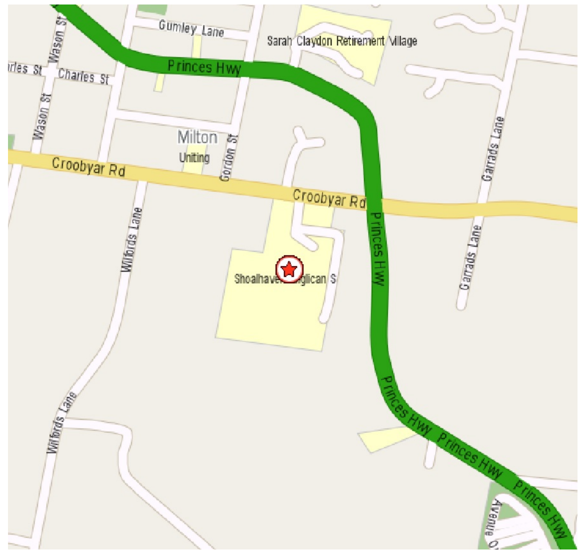
 Dacite, epidote

 Tuff, breccia

 Porphyry

Appendix B

Drawing 1
Master Plan, Project 190941, Drawing No. SSDA-2000 Issue D
Site Photographs
Borehole Logs (Bores 101 – 108)



Locality Plan

Legend

- Borehole Location
- Lot Boundary (Approx.)
- Proposed Pavement Area (T.B.C)
- Proposed Building (T.B.C)
- Approximate Proposed Development Boundary (T.B.C)
- Approximate Location of Existing Sewer Main

0 10 20 30 40 50 m

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: School Infrastructure NSW (SINSW) Pty Ltd		TITLE: **Test Location Plan**	PROJECT No: 89390.02
OFFICE: Wollongong	DRAWN BY: DJM	**Proposed Budawang School Relocation**	DRAWING No: 1
SCALE: 1:700 @ A3	DATE: 23/04/2021	**17 Croobyar Road, Milton**	REVISION: 2

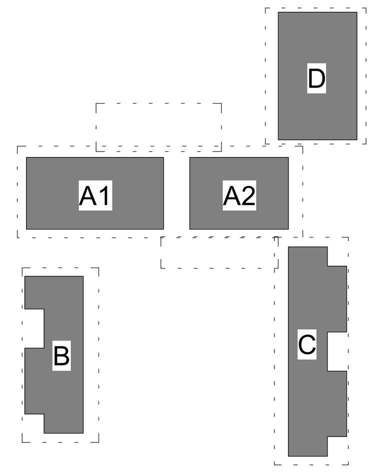
1 SITE PLAN
1 : 250

SITE PLAN LEGEND

- NEW LANDSCAPE
- NEW PATHWAYS
- NEW BUILDINGS
- ROADS
- PARKING SPACES
- BUDAWANG SCHOOL BOUNDARY
- EXISTING SEWERLINE
- EXISTING TREES
- PROPOSED TREES
- BOUNDARY TREES
- DOE OWNERSHIP
- LP LIGHT POLE

Amendments

Issue	Description	Date
A	ISSUE FOR INFORMATION	12/03/2021
B	ISSUE FOR INFORMATION	23/03/2021
C	ISSUE FOR INFORMATION	25/03/2021
D	ISSUE FOR SSDA	08/04/2021



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architecture interior design urban design landscape
nom architect M. Sheldon 3990

Project Title
BUDAWANG SCHOOL

Drawing Title
SITE PLAN

Scale	1 : 250
Drawing Created (date)	02/24/21
Drawing Created (by)	TKD
Plotted and checked by	RF
Verified	RF
Approved	RK

Project No	Drawing No	Issue
190941	SSDA-2000	D

9/04/2021 12:36:00 AM

Photo 1 – View across the north eastern section of the site towards a small creek and pump station.

Photo 2 – View to the north east across the front sporting field.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT:	89390.02
	Proposed Budawang School Relocation	PLATE No:	1
	17 Croobyar Road, Milton	REV:	0
	CLIENT: School Infrastructure NSW	DATE:	06/11/2020

Photo 3 – View to the north towards an existing building.

Photo 4 – View to the south showing existing buildings.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT:	89390.02
	Proposed Budawang School Relocation	PLATE No:	2
	17 Croobyar Road, Milton	REV:	0
	CLIENT: School Infrastructure NSW	DATE:	06/11/2020

Photo 5 – View across northern car park.

Photo 6 – View across southern car park.

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT:	89390.02
	Proposed Budawang School Relocation	PLATE No:	3
	17 Croobyar Road, Milton	REV:	0
	CLIENT: School Infrastructure NSW	DATE:	06/11/2020

Photo 7 – View east of the southernmost existing building at the site.

Photo 8 – View south behind existing tennis courts across a grassed fill platform.

BOREHOLE LOG

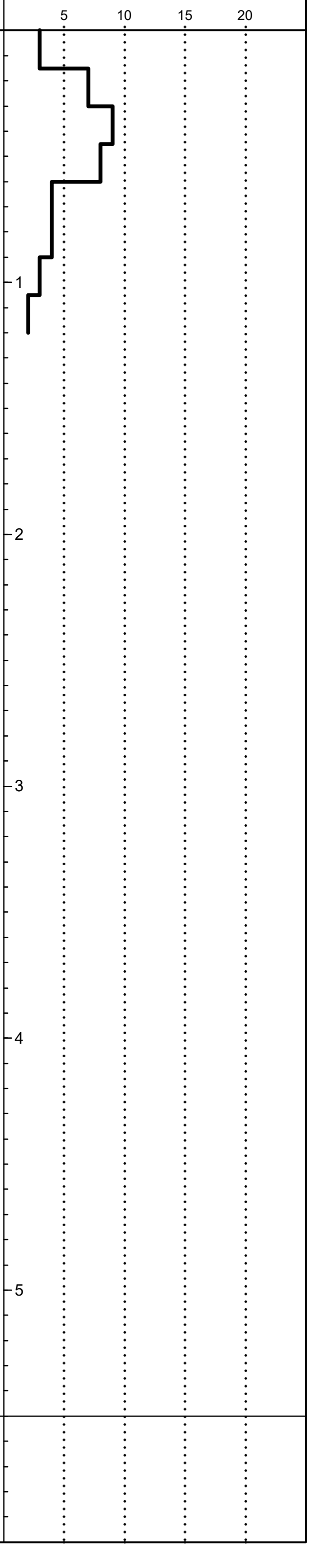
CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 48.0 AHD
EASTING: 267398
NORTHING: 6088349
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
48	0.1	TOPSOIL/SILT (ML): low plasticity, dark brown, w<PL	E	0.1		PID = 0.9	
		FILL/Silty GRAVEL (GP): medium gravel, brown and dark brown, dry	D E	0.5		PID = 1.0	
47	1.0	CLAY (CH): high plasticity, dark brown, with silt, w>PL, firm	D E	1.0		PID = 0.8	
			D E	1.5		pp = 50 PID = 0.6	
46	2		D	2.0		pp = 50	
		-becoming grey mottled green and brown, trace sand below 2.2m	D	2.5		pp = 75	
45	3	-becoming stiff below 3.0m	D	3.0		pp = 100	
	3.3	Sandy CLAY (CI): medium plasticity, brown red, fine to medium sand, w>PL, very stiff					▷ (3.5m)
44	4						
			D	4.5			
43	5						
	5.5	Bore discontinued at 5.5m -limit of investigation	D	5.5			

Dynamic Penetrometer Test (blows per 150mm)



RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: Groundwater seepage at 3.5m
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

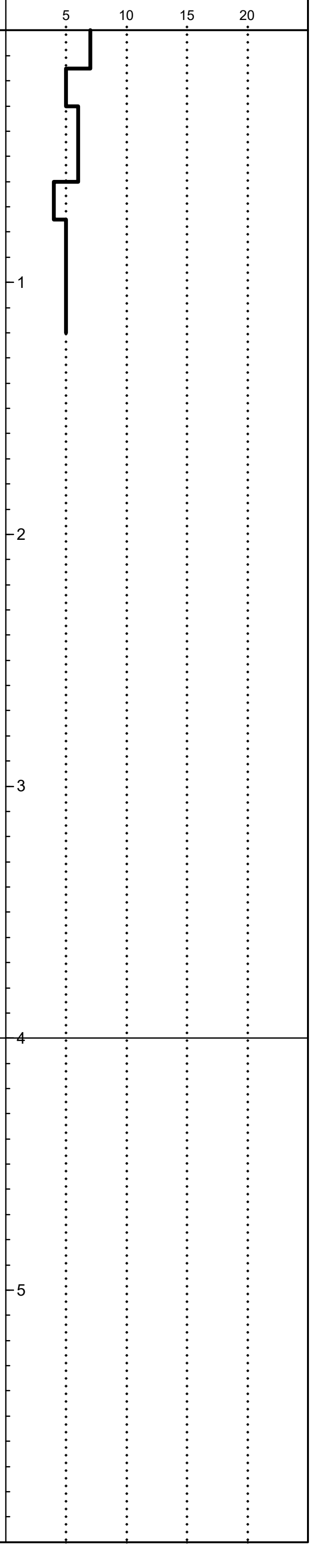
CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 50.7 AHD
EASTING: 267471
NORTHING: 6088360
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		TOPSOIL/SILT (ML): low plasticity, dark brown, w<PL		E	0.1		PID = 1.5	
	0.15	CLAY (CH): high plasticity, brown, w<PL, stiff		D E	0.5		pp = 100-150 PID = 0.1	
50	1	-with fine to medium sand below 0.8m		D E	1.0		pp = 150 PID = 0.3	
49				D E	1.5		PID = 0.2	
	1.8	Sandy CLAY (CI): medium plasticity, red brown, fine to medium sand, w~PL, stiff		D	2.0			
48	2			D	2.5			
	3	-very stiff to hard, below 3.0m		D	3.0			
47	3.5	MONZONITE: medium grained, orange brown, with extremely weathered rock bands, very low strength, highly weathered		D	3.5			
46	4.0	Bore discontinued at 4.0m -refusal on very low strength monzonite		D	4.0			
45	5							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: No free groundwater observed
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton
SURFACE LEVEL: 48.3 AHD
EASTING: 267430
NORTHING: 6088321
DIP/AZIMUTH: 90°/--
BORE No: 103
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments
0.0–0.15	TOPSOIL/SILT (ML): low plasticity, dark brown, w<PL	E	0.1		PID = 0.1
0.15–1.3	CLAY (CH): high plasticity, brown, with silt, w<PL, stiff to very stiff	D E	0.5		pp = 200 PID = 0.8
		B			
		D E	1.0		PID = 0.4
1.3–2.2	CLAY (CH): high plasticity, pale brown orange, with fine to medium sand, w>PL, stiff	D E	1.5		pp = 150 PID = 0.6
		D	2.0		pp = 150-200
2.2–2.9	Sandy CLAY (CI): medium plasticity, brown orange, fine to medium sand, w>PL, stiff to very stiff	D	2.5		pp = 150-200
2.9–3.0	MONZONITE: medium grained, brown orange red, with extremely weathered rock bands, very low strength, highly weathered	D	3.0		
3.0	Bore discontinued at 3.0m -refusal on very low strength monzonite				

Dynamic Penetrometer Test (blows per 1000mm): 5 10 15 20

RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: No free groundwater observed
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 46.7 AHD
EASTING: 267375
NORTHING: 6088306
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water
0.1	TOPSOIL/SILT (ML): low plasticity, dark brown, w~PL					
	Sandy CLAY (CH): high plasticity, pale brown orange, fine to medium sand, w>Pl, stiff (possible fill)	B D E	0.1		PID = 0.4	
0.5	CLAY (CH): high plasticity, brown, w>Pl, stiff	D E	0.5		PID = 0.4	
		B				
		D E	1.0		PID = 0.3	
	-firm below 1.2m					
		D	1.5		pp = 50-100	
	-stiff, with fine sand below 2.0m	D	2.0		pp = 75-100	
		D	2.5		pp = 100	
2.6	Sandy CLAY (CI): medium plasticity, red brown, fine to medium sand, w>PL, stiff					
		D	3.0			
						▷ (3.4)
	-becoming very stiff to hard below 3.5m	D	3.5			
4.0	Bore discontinued at 4.0m -refusal on hard sandy clay	D	4.0			

Dynamic Penetrometer Test (blows per 1000mm): 5 10 15 20

RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: Groundwater seepage at 3.4m
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

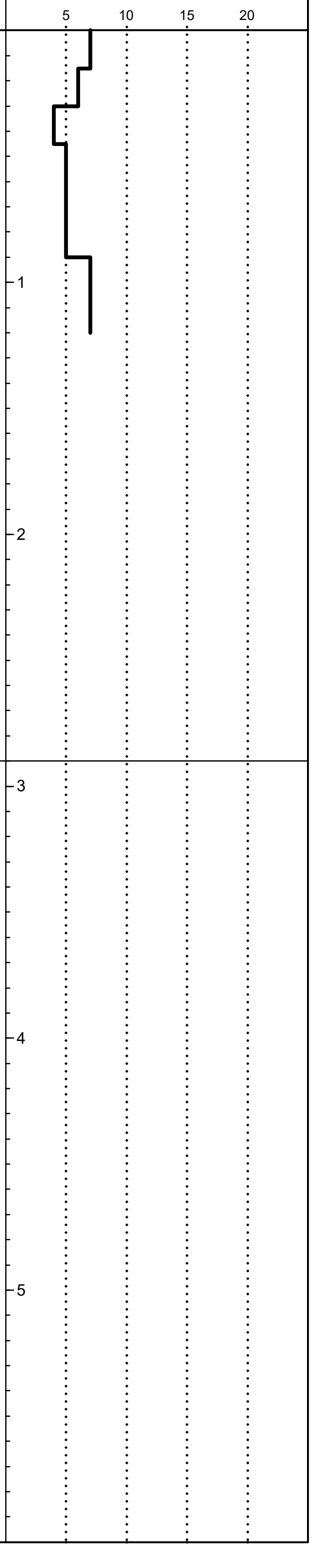
CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 51.4 AHD
EASTING: 267494
NORTHING: 6088304
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water
	0.1	TOPSOIL/SILT (ML): low plasticity, dark brown, w<pl		D	0.1		PID = 1.1	
51		CLAY (CH): high plasticity, brown, trace fine to medium sand, w~PL, stiff		E B	0.2			
				D E	0.5		pp = 150 PID = 0.2	
	1			D E	1.0		pp = 150-200 PID = 0.5	
50				D	1.5		pp = 100-150	
	1.9	-becoming stiff to very stiff below 1.8m						
	2	Sandy CLAY (CI): medium plasticity, fine to medium sand, w~PL, stiff		D	2.0			
49	2.5	MONZONITE: medium grained, brown with pale grey, with extremely weathered rock bands, very low strength, highly weathered		D	2.5			
	2.9	Bore discontinued at 2.9m -refusal on very low strength monzonite						
48	3							
47	4							
46	5							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: No free groundwater observed
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 50.2 AHD
EASTING: 267460
NORTHING: 6088279
DIP/AZIMUTH: 90°/--

BORE No: 106
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 1000mm)
50	0.15	TOPSOIL /SILT (ML): low plasticity, dark brown, trace clay, w<PL		E	0.1		PID = 3.9		
		CLAY (CH): high plasticity, brown, trace silt, w<PL, stiff							
	0.5	CLAY (CH): high plasticity, pale brown, trace fine to medium sand, w<PL, stiff to very stiff		D E	0.5		pp = 150-200 PID = 0.2		
	1			D E	1.0		pp = 200 PID = 0.2		
49	1.2	CLAY (CI): medium plasticity, pale orange brown, trace fine to medium sand, w<PL, very stiff		D E	1.5		pp = 200-250 PID = 0.4		
	2	-with extremely weathered rock bands below 2m		D	2.0				
48	2.3	MONZONITE: medium grained, brown orange, with extremely weathered bands, very low strength, highly weathered		D	2.5				
	2.5	Bore discontinued at 2.5m -refusal on very low strength monzonite							

Depth scale: 1, 2, 3, 4, 5. RL scale: 50, 49, 48, 47, 46, 45. DCP scale: 5, 10, 15, 20.

RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: No free groundwater observed
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

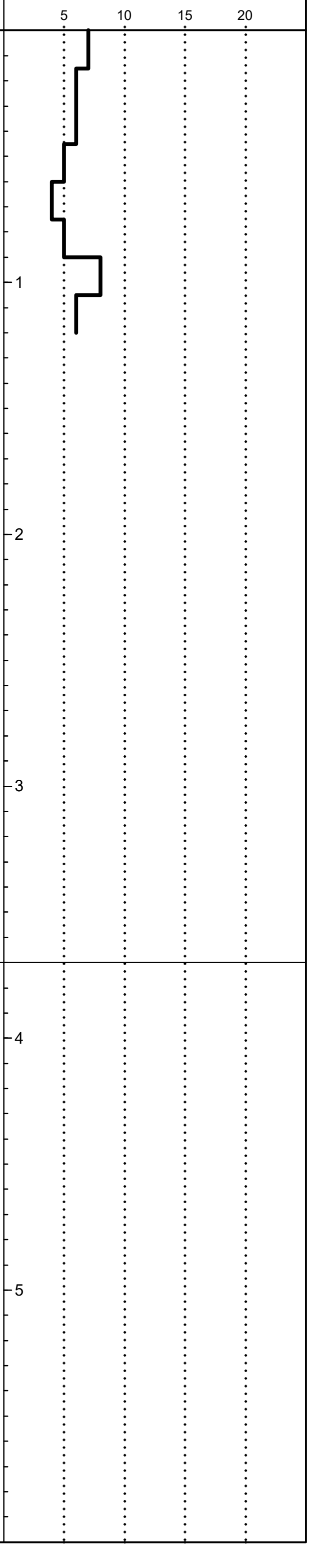
CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 48.0 AHD
EASTING: 267406
NORTHING: 6088243
DIP/AZIMUTH: 90°/--

BORE No: 107
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water
48		TOPSOIL/SILT (ML): low plasticity, dark brown, trace clay, w<PL		E	0.1		PID = 1.8	
	0.2	CLAY (CH): high plasticity, dark brown, with silt, w<PL, stiff						
	0.5	CLAY (CH): high plasticity, pale orange brown, trace fine to medium sand, w~PL, stiff -w<PL, below 0.8m		D E	0.5		pp = 200 PID = 18.2	
				B				
47	1			D E	1.0		pp = 150 PID = 0.2	
				D E	1.5		pp = 100-150	
	1.8	Sandy CLAY (CI): medium plasticity, pale orange brown, fine to medium sand, w<Pl, stiff						
46	2	-becoming w>PL, below 2.2m		D	2.0		pp = 100	
				D	2.5			▷
45	3			D	3.0			
				D	3.5			
	3.7	Bore discontinued at 3.7m -refusal on very low strength monzonite						
44	4							
43	5							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: Groundwater seepage at 2.7m
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: School Infrastructure New South Wales (SINSW)
PROJECT: Proposed Budawang School Relocation
LOCATION: 17 Croobyar Road, Milton

SURFACE LEVEL: 52.5 AHD
EASTING: 267490
NORTHING: 6088240
DIP/AZIMUTH: 90°/--

BORE No: 108
PROJECT No: 89390.02
DATE: 13/10/2020
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 1000mm)
	0.2	TOPSOIL/SILT (ML): low plasticity, dark brown, trace fine sand, w<PL		D E	0.1		PID = 6.4		
52	0.6	CLAY (CH):high plasticity, brown mottled dark brown, with silt, w<PL, stiff		D E	0.5		pp = 150 PID =.6		
	1	Sandy CLAY (CI): medium to high plasticity, brown, fine to medium sand, w<PL, stiff		D E	1.0		pp = 150		
51		-becoming very stiff, with extremely weathered rock bands below 1.3m		D E	1.5				
	1.9								
	2	MONZONITE: medium grained, brown with grey, with extremely weathered bands, very low strength, highly weathered		D	2.0				
	2.2	Bore discontinued at 2.2m -refusal on very low strength monzonite							
50	3								
49	4								
48	5								
47									

RIG: Kubota KX018-4 mini-excavator **DRILLER:** Clinton Taylor **LOGGED:** FH **CASING:** Uncased
TYPE OF BORING: Solid flight auger - TC bit
WATER OBSERVATIONS: No free groundwater observed
REMARKS: w =moisture content, PL = plastic limit

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix C

Acid Sulfate Soil Screening Results
Chain of Custody
Laboratory Test Results

RESULTS OF ACID SULFATE SCREENING TESTS

Client: School Infrastructure NSW (SINSW) Pty Ltd
Project: **Proposed Budawang School Relocation**
Project Location: **17 Croobyar Road, Milton**

Project No: **89390.02**
pH Meter: ☐ TPS with Ionode IJ46/WP80 pH/Temp. Electrode
☑ PH Scan 2
Calibration Buffer: ☑ pH4
☑ pH7
☑ pH10

Sample Location	Depth (m)	pH_F (in distilled water)	pH_{FOX} (oxidised in H_sO_s)			Strength of Reaction	Soil Description
		Date: 21/10/2020	Date: 21/10/2020	Date:	Date:	(1,2,3,4)*	
		Time: 11:00am	Time: 11:40am	Time:	Time:	F **	
101	0.5	6.41	6.45			3F	Fill: Silty Gravel
101	1.0	6.86	5.81			3F	Fill Silty Gravel

Legend: * *1 denotes no or slight effervescence*
2 denotes moderate effervescence
3 denotes vigorous effervescence
4 denotes "volcano" ie. very vigorous effervescence, gas evolution and heat
** F after reaction number indicates a bubbling/frothy reaction (organics)

Operator: DPM

Date: 21/10/2020

Douglas Partners
Geotechnics / Environment / Groundwater

CHAIN OF CUSTODY DESPATCH SHEET

Project No:	89390.02	**Suburb:**	Milton	**To:**	Envirolab Services
Project Name:	Proposed SSP	**Order Number**	89390.02 A		Ashley Street, Chatswood
Project Manager:	David Metcalf	**Sampler:**	Fiona Henry	**Attn:**	Simon Song
Emails:	kenton.horsley@douglaspartners.com.au		david.metcalf@douglaspartners.com.au	**Phone:**	99106200
Date Required:	Standard			**Email:**	Samplereceipt@envirolabservices.com.au
Prior Storage:	Fridge	Do samples contain 'potential' HBM?	NO		

Sample ID	Lab ID	Date Sampled	Sample Type (S - soil, W - water)	Container Type (G - glass, P - plastic)	Analytes: Aggressivity Suite									Notes/preservation
102/0.5	1	13/10/20	S	G&P	X									PLEASE ISSUE A SEPARATE
104/0.5	2	13/10/20	S	G&P	X									SAMPLE RECEIPT / RESULTS
105/1.0	3	13/10/20	S	G&P	X									FOR THE AGGRESSIVITY
108/1.0	4	13/10/20	S	G&P	X									PART OF THIS PROJECT
														(Thankyou!) :)
PQL (S) mg/kg														ANZECC PQLs req'd for all water analytes ☐

Envirolab Services
ENVIROLAB
12 Ashley St
Chatswood NSW 2067
Ph: (02) 9910 6200
Job No: 253917
Date Received: 21.10.20
Time Received: 1515
Received By: MO
Temp: Cool/Ambient
Cooling: Ice/Icepack
Security: Intact/Broken/None

PQL = practical quantitation limit. If none given, default to Laboratory Method Detection Limit

Lab Report/Reference No: 253917

Metals to Analyse: 8HM unless specified here: 8HM + Mn

Total number of samples in container: **Relinquished by:** DPM **Transported to laboratory by:**

Send Results to: Douglas Partners Pty Ltd **Address:** **Phone:** **Fax:**

Signed: **Received by:** Michael Opie ELS **Date & Time:** 21.10.20 1515

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS 253917

Client Details

Client	Douglas Partners Unanderra
Attention	Kenton Horsley
Address	Unit 1, 1 Luso Drive, Unanderra, NSW, 2526

Sample Details

Your Reference	**89390.02, Milton**
Number of Samples	4 Soil
Date samples received	21/10/2020
Date completed instructions received	21/10/2020

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	28/10/2020
Date of Issue	27/10/2020

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Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Nancy Zhang, Laboratory Manager

Soil Aggressivity					
Our Reference	UNITS	253917-1	253917-2	253917-3	253917-4
Your Reference		102/0.5	104/0.5	105/1.0	108/1.0
Date Sampled		13/10/2020	13/10/2020	13/10/2020	13/10/2020
Type of sample		Soil	Soil	Soil	Soil
pH 1:5 soil:water	pH Units	5.2	5.6	5.3	4.9
Electrical Conductivity 1:5 soil:water	µS/cm	68	65	70	130
Resistivity in soil*	ohm m	150	150	140	78
Chloride, Cl 1:5 soil:water	mg/kg	37	20	21	78
Sulphate, SO4 1:5 soil:water	mg/kg	32	72	58	100

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	101	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	91	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	108	[NT]

Result Definitions

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

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

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

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH/EC
Samples were out of the recommended holding time for this analysis.

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater

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Email: anes.ibricic@douglaspartners.com.au

Report Number:	**89390.02-1**
Issue Number:	1
Date Issued:	04/11/2020
Client:	School Infrastructure New South Wales (SINSW) Level 8, 259 George Street, Sydney NSW 2000
Contact:	Michael Stern
Project Number:	89390.02
Project Name:	Proposed Bundawang SSP Relocation
Project Location:	17 Croobyar Road, MILTON
Work Request:	6565
Sample Number:	WO-6565A
Date Sampled:	13/10/2020
Dates Tested:	21/10/2020 - 03/11/2020
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**103, Depth: 0.5-1.0**
Material:	Clay

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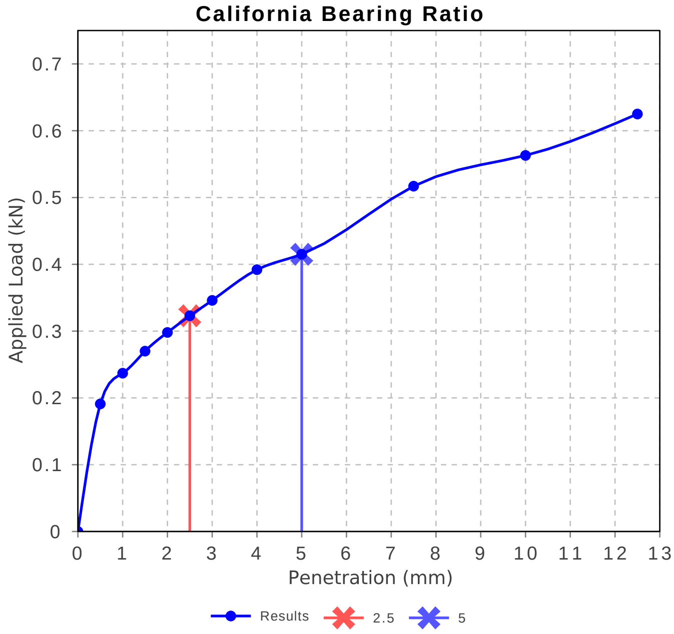
Approved Signatory: Anes Ibricic
Laboratory Manager

NATA Accredited Laboratory Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 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)	1.31		
Optimum Moisture Content (%)	37.0		
Laboratory Density Ratio (%)	100.0		
Laboratory Moisture Ratio (%)	100.0		
Dry Density after Soaking (t/m^3)	1.28		
Field Moisture Content (%)	39.5		
Moisture Content at Placement (%)	37.1		
Moisture Content Top 30mm (%)	43.7		
Moisture Content Rest of Sample (%)	38.2		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	96.1		
Swell (%)	2.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Material Test Report

Report Number:	**89390.02-1**
Issue Number:	1
Date Issued:	04/11/2020
Client:	School Infrastructure New South Wales (SINSW)
	Level 8, 259 George Street, Sydney NSW 2000
Contact:	Michael Stern
Project Number:	89390.02
Project Name:	Proposed Bundawang SSP Relocation
Project Location:	17 Croobyar Road, MILTON
Work Request:	6565
Sample Number:	WO-6565B
Date Sampled:	13/10/2020
Dates Tested:	21/10/2020 - 02/11/2020
Sampling Method:	Sampled by Engineering Department
	The results apply to the sample as received
Sample Location:	**107, Depth: 0.5-1.0**
Material:	Clay

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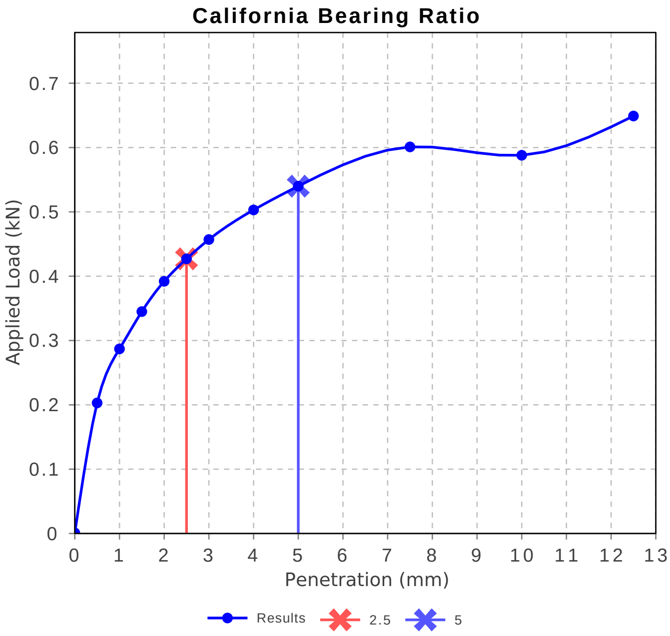
Laboratory Manager

NATA Accredited Laboratory Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	**3.0**		
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)	1.35		
Optimum Moisture Content (%)	34.0		
Laboratory Density Ratio (%)	100.5		
Laboratory Moisture Ratio (%)	99.5		
Dry Density after Soaking (t/m^3)	1.34		
Field Moisture Content (%)	34.2		
Moisture Content at Placement (%)	33.7		
Moisture Content Top 30mm (%)	40.9		
Moisture Content Rest of Sample (%)	35.6		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	48.4		
Swell (%)	1.5		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Material Test Report

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Report Number: **89390.02-1**
Issue Number: 1
Date Issued: 04/11/2020
Client: School Infrastructure New South Wales (SINSW)
Level 8, 259 George Street, Sydney NSW 2000
Contact: Michael Stern
Project Number: 89390.02
Project Name: Proposed Bundawang SSP Relocation
Project Location: 17 Croobyar Road, MILTON
Work Request: 6565
Sample Number: WO-6565C
Date Sampled: 13/10/2020
Dates Tested: 21/10/2020 - 02/11/2020
Sampling Method: Sampled by Engineering Department
The results apply to the sample as received
Sample Location: **102, Depth: 0.5m**
Material: Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	73		
Plastic Limit (%)	20		
Plasticity Index (%)	**53**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	13.0		
Cracking Crumbling Curling	Curling		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	36.3

Material Test Report

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Issue Number:	1
Date Issued:	04/11/2020
Client:	School Infrastructure New South Wales (SINSW) Level 8, 259 George Street, Sydney NSW 2000
Contact:	Michael Stern
Project Number:	89390.02
Project Name:	Proposed Bundawang SSP Relocation
Project Location:	17 Croobyar Road, MILTON
Work Request:	6565
Sample Number:	WO-6565D
Date Sampled:	13/10/2020
Dates Tested:	21/10/2020 - 02/11/2020
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**103, Depth: 1.0m**
Material:	Clay

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Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	85		
Plastic Limit (%)	23		
Plasticity Index (%)	**62**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	15.5		
Cracking Crumbling Curling	Cracking & Curling		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	40.2

Material Test Report

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Issue Number:	1
Date Issued:	04/11/2020
Client:	School Infrastructure New South Wales (SINSW) Level 8, 259 George Street, Sydney NSW 2000
Contact:	Michael Stern
Project Number:	89390.02
Project Name:	Proposed Bundawang SSP Relocation
Project Location:	17 Croobyar Road, MILTON
Work Request:	6565
Sample Number:	WO-6565E
Date Sampled:	13/10/2020
Dates Tested:	21/10/2020 - 03/11/2020
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**105, Depth: 0.5m**
Material:	Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	83		
Plastic Limit (%)	27		
Plasticity Index (%)	**56**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	16.0		
Cracking Crumbling Curling	Curling		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	36.1

Material Test Report

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Report Number:	**89390.02-1**
Issue Number:	1
Date Issued:	04/11/2020
Client:	School Infrastructure New South Wales (SINSW) Level 8, 259 George Street, Sydney NSW 2000
Contact:	Michael Stern
Project Number:	89390.02
Project Name:	Proposed Bundawang SSP Relocation
Project Location:	17 Croobyar Road, MILTON
Work Request:	6565
Sample Number:	WO-6565F
Date Sampled:	13/10/2020
Dates Tested:	21/10/2020 - 03/11/2020
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**108, Depth: 1.0m**
Material:	Sandy Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	60		
Plastic Limit (%)	23		
Plasticity Index (%)	**37**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	12.5		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	23.4

Appendix D

CSIRO Foundation Publication

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

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

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

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

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

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

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

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

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

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

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

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

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

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage

As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

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

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

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

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

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

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

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

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

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

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

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

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

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

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

Prevention/Cure

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

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

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

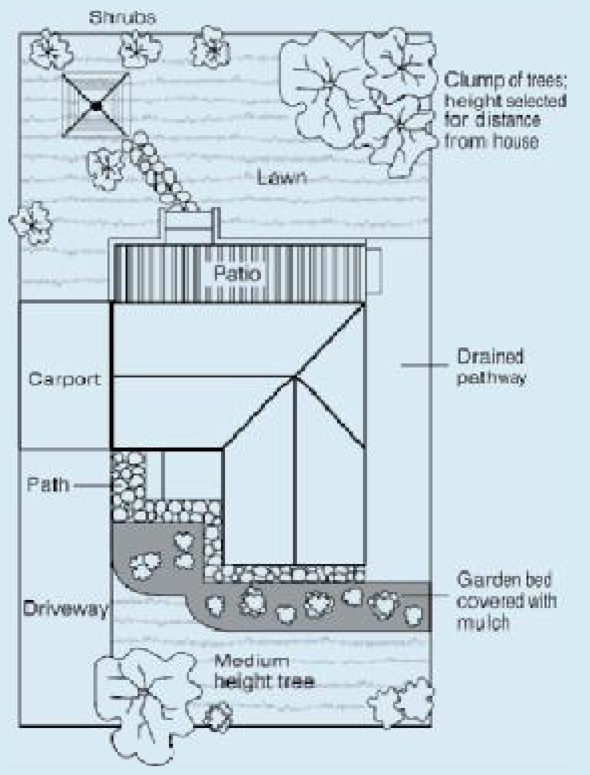
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

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

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

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

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

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

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

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

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

Remediation

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

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

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Distributed by

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