Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Supplementary Geotechnical Investigation

High School Redevelopment
Argyle Street, Picton, NSW

Prepared for
Billard Leece Partnership Pty Ltd

Project 34252.02
April 2018

Integrated Practical Solutions

Douglas Partners
Geotechnics / Environment / Groundwater

Document History

Document details

Project No.	34252.02	Document No.	R.004.Rev1
Document title	Report on Supplementary Geotechnical Investigation High School Redevelopment		
Site address	Argyle Street, Picton, NSW		
Report prepared for	Billard Leece Partnership Pty Ltd		
File name	34252.02.R.004.Rev1		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Rasoul Machiani	Michael Thom	15 August 2017
Revision 1	Rasoul Machiani	Michael Thom	4 April 2018

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Billard Leece Partnership Pty Ltd – Mr Shane Wood
Revision 1	1	0	Billard Leece Partnership Pty Ltd – Mr Shane Wood

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.

	Signature	Date
Author		4 April 2018
Reviewer	pp for MT	4 April 2018

bsi. ISO 9001 Quality Management

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Table of Contents

Report on Supplementary Geotechnical Investigation
High School Redevelopment
Argyle Street, Picton, NSW

1. Introduction

This report presents the results of a supplementary geotechnical investigation undertaken for proposed redevelopment works within Picton High School at Argyle Street, Picton, NSW. The investigation was commissioned in an email dated 30June 2017 by Mr Shane Wood of Billard Leece Partnership Pty Ltd (Architects) and was undertaken in accordance with Douglas Partners' (DP) email proposal dated 8 June 2017.

DP understands that the proposed redevelopment works will include the removal of some demountable buildings across the site and the construction of new teaching blocks, associated facilities and pavements.

In the absence of conceptual design details, a preliminary geotechnical investigation was undertaken by DP consisting of ten test pits across the school site (Project No. 34252.02.P.001). A conceptual design drawing was provided for a supplementary geotechnical investigation that shows the location of proposed two and three storey buildings. The detailed design information of proposed new permanent buildings and cut-fill plans are yet to be finalized.

The supplementary investigation included the drilling of cored boreholes and laboratory testing of selected samples. Details of the work undertaken and the results obtained are given within this report, together with comments relating to foundation design and earthworks. This report should be read in conjunction with the Preliminary Geotechnical Investigation Report.

DP have undertaken Contamination Assessment (Project No. 34252.02.P.002) and Hazmat Survey (Project 34252.02.P.003) for the development, both of which have been reported separately.

2. Site Description

Picton High School is located approximately 90 km to the south-west of the Sydney CBD and is a rectangular shaped area of some 6 ha. Maximum north-south and east-west dimensions are approximately 200 m and 290 m respectively. The school site is bounded by Argyle Street to the west, residential properties to the north, and vacant land to the south and east. The school is currently occupied by 32 existing buildings comprising permanent and demountable structures of various sizes, a car park and playing fields.

The school site is located within undulating rises with overall topographic relief of approximately 8 m from the highest parts (approximately RL218 m, relative to the Australian height datum) within the eastern and western portions to the lowest part (approximately RL 210 m) within the mid northern portion of the site. However, it has been partially levelled by cutting and filling to create flat platforms for the existing structures.

Approximately, two-thirds of the site is covered by the existing structures and carparks. These structures are generally located within the western portion of the site extending toward the middle portion. Two sporting fields are noted along the eastern and southern boundaries of the school site. At the time of the investigation, the vegetation across these open spaces was limited to well-maintained light grass. Medium size trees were present along the northern boundary with scattered trees noted between the existing buildings.

3. Regional Geology

Reference to the Wollongong-Port Hacking 1:100,000 Geological Sheet (Ref 1) indicates that the site is underlain by Ashfield Shale (mapping unit Rwa) of the Wianamatta Group of Triassic age. The Ashfield Shale typically comprises shale, siltstone, claystone and laminite with coal bands, all of which weathered to form clays of high plasticity. The results of the investigation were consistent with the geological mapping, with fine grained lithic sandstone of variable weathering and seams of siltstone encountered in the boreholes.

4. Field Work Methods

The field work comprised the drilling of three boreholes (Bores 101 – 103) to a depth of 8.0 m using a Christie Engineering trailer drilling rig using a combination of continuous solid flight augers with a nominal 100 mm diameter and ‘NMLC’ rotary coring techniques and water flush with steel casing to obtain continual rock core samples. Standard penetration tests (AS 1289.6.3.1) were also carried out at a depth of 1.0 m within all boreholes whilst augering. The standard penetration test procedure is given in the attached notes and the penetration ‘N’ value obtained during testing is shown on the borehole logs.

The fieldwork was undertaken by a geotechnical engineer who logged the boreholes and collected disturbed samples to assist in strata identification and for laboratory testing. Following logging, testing and sampling, each borehole was backfilled and the ground surface reinstated to its previous level.

The borehole locations were nominated by the client and located on site prior to the investigation using differential GPS unit for which an accuracy of ± 20 mm is typical. The location of boreholes are shown on Drawing 1 (Appendix A). The surface levels were obtained using the differential GPS unit.

All field measurements and mapping for this project have been carried out using the Geodetic Datum of Australia 1994 (GDA94) and the Map Grid of Australia 1994 (MGA94 Zone 56). All reduced levels are given in relation to Australian Height Datum (AHD).

5. Field Work Results

The boreohle logs are included in Appendix B, and should be read in conjunction with the accompanying standard notes that define classification methods and descriptive terms. Relatively uniform conditions were encountered underlying the site with the general succession of strata is broadly summarised as follows:

- ASPHALTIC CONCRETE – 50 mm tick in Bores 101 & 102;
- TOPSOIL – brown silty clay (filling) to a depth of 0.1 m in Bore 103;
- FILLING –generally brown silty clay with some gravel to depths of 0.7 – 2.0 m in all boreholes;
- SILTY CLAY – very stiff grey/brown with seams of extremely weathered shale in Bores 101 & 102 to depths of 1.8 – 2.3 m; and
- BEDROCK – medium to high strength siltstone and sandstone at depths of 2.2 – 2.9 m, continued to the termination depth of boreholes.

No free groundwater was observed in the boreholes during auger drilling and for the short time that they were left open. The introduction of water into the boreholes during the rotary coring and the immediate backfilling of the test locations precluded any long-term observations of groundwater levels that might be present. It's noted that groundwater levels are affected by factors such as weather conditions and can fluctuate with time.

6. Laboratory Testing

Selected samples from the test pits excavated for the preliminary geotechnical investigation were tested in the laboratory for measurement of field moisture content, Atterberg limits, shrink-swell and California bearing ratio (CBR). The results were provided in the preliminary geotechnical investigation report.

6.1 Point Test Testing

Selected rock core samples were tested in the laboratory for measurement of point load strength index ($Is_{(50)}$) to estimate rock strength at variable depths. The detailed laboratory test report sheets are given in Appendix C and the values of $Is_{(50)}$ are shown on the borehole logs.

6.2 Soil Aggressivity

Selected samples from the boreholes were tested in the laboratory for aggressivity assessment by measuring pH, sulphates, chlorides, electrical conductivity. The detailed test report sheets are given in Appendix D, with the results summarised in Table 1.

Table 1: Results of Laboratory Testing – Aggressivity

Bore	Depth (m)	pH	Chloride (mg/kg)	Sulphate (mg/kg)	EC (µS/cm)	Material
101	1.0	5.9	10	21	36	Silty clay
101	3.1	9.0	37	<10	190	Siltstone
102	1.0	5.3	210	27	210	Silty Clay

The exposure classification of the surface of concrete and steel piles was determined in accordance with AS 2159 – 1996 (Ref 2) as detailed in Table 6.4.2 (c) and Table 6.5.2 (c) which indicates the soils tested would be classified as *"non aggressive"* to concrete and steel.

7. Proposed Development

It is understood that the redevelopment works comprise the removal of selected demountable buildings within the site and the construction of new permanent teaching blocks. The proposed permanent buildings are understood to be two and three storey and are expected to be founded on piers constructed within good quality rock. The design loads and other detail design information of the structures are unknown at the time of writing this report.

8. Comments

8.1 General

Comments are provided in the following sections on development constraints related to geotechnical and geological factors to assist in the foundation design of the proposed two and three storey buildings. As detailed design of the proposed redevelopment works has not been undertaken, the comments given must also be considered as being preliminary in nature. Once details are available, they should be forwarded to DP for review to determine if comments given within this report are appropriate or require revision.

8.2 Subsurface Conditions and Rock Strength

The following comments are based on the surface and subsurface profiles encountered during the investigation and the results of laboratory testing of selected samples collected at the borehole locations. The boreholes have indicated that subsurface conditions underlying the site typically comprise asphalt or topsoil to a depth of 0.1 – 0.4 m underlain by filling to depths of 0.7 – 2.0 m. The filling is underlain by silty clays and low strength siltstone and sandstone to depths within the range 2.2 – 2.9 m. This in turn overlaid bedrock of medium to high strength condition to the final depth of boreholes.

The bedrock from the cored boreholes has been classified in accordance with Reference 3 and depths/RLs of each rock class are summarised in Table 2.

Table 2: Depth/Level of Rock Classes

Bore	RL Depth (m)	Thickness (m)	Rock Class	
			Sandstone	Shale
101 Surface Level: 217.7m AHD	214.7 – 214.0	0.7	-	IV
	214.0 – 212.0	2.0	-	III
	212.0 – 209.6	2.4	III	-
102 Surface Level: 214.6m AHD	212.3 – 211.5	0.8	IV	-
	211.5 – 206.5	5.0	III	-
103 Surface Level: 215.6m AHD	213.4 – 212.9	0.5	IV	-
	212.9 – 207.5	5.4	III	-

The cored borehole logs indicate that the rock structure is mainly governed by horizontal to sub-horizontal (0º – 10º) bedding and horizontal to steeply-inclined (0° – 45°) jointing observed mainly in fractured siltstone. The fracture spacings shown on the recovered core samples show 'highly fragmented' siltstone to depths of 5.6 m in Bore 101 (Approx. RL 212 m AHD). Medium strength sandstone was encountered in the boreholes at RL's 212 – 213 m AHD and identified as 'moderately fractured' to 'unbroken' (fracture spacings of 100 – 1000 mm).

8.3 Foundations

The results of the investigation indicates that good quality weathered rock will be expected at depths ranging from 2.5 – 4.0 m at the borehole locations, and hence, pending the required excavation depth, deep foundations in the form of bored or driven piles would be suitable options to accommodate the loads of the proposed two or three storey buildings. The use of shallow footings may only be justified for the lightly loaded structures founded in controlled filling or stiff natural clay.

Based on the results of the field investigation and laboratory testing, retaining wall and building footings could be proportioned using the maximum design parameters presented in Table 3. The footing recommendations and design parameters for any given strata will need to be confirmed following the completion of design stage when the final excavation depth, footing size and design loads are specified.

Table 3: Estimated Design Parameters

Material		Ultimate Base Bearing Pressures (kPa) (1)	Ultimate Shaft Adhesion Pressures (kPa) (2)	Allowable Base Bearing Pressures (kPa) (3)	Allowable Shaft Adhesion Pressures (kPa)	Allowable Lateral Resistance (kPa)
Controlled fill		-	-	100	-	-
Very stiff to hard clay		-	-	200	-	-
Sandstone	Class V	5000	200	1200	100	400
	Class IV	8000	400	3500	350	1200
	Class III	25000	1000	6000	600	2000
Shale	Class V	3000	100	700	70	200
	Class IV	6000	150	1000	100	300
	Class III	20000	750	3500	350	1200

Notes (1) The values are in accordance with Pells et al- 1998 (Ref3);
(2) Ultimate values occur at large settlements (generally >5% of the minimum footing width);
(3) Values can only be adopted for clean sockets of roughness category R2 or better. Values may need to be reduced to account for smear;
(4) Value for rock based on settlements of <1% of minimum footing width.

Base bearing and shaft adhesion values have also been provided for Limit State design. The geotechnical strength reduction factor Φg of 0.45 shall be applied in accordance with AS2159-2009, Table 4.3.2 based on the available information.

Reference should be made to the borehole logs (Appendix B) and Table 2 with respect to the depth/levels of the various bearing strata

8.4 Earthworks

It is considered that some bulk earthworks including the removal of existing structures and underlying moisture affected or unsuitable material will be expected. The final earthworks plans have not been finalized at the time of preparing this report. It can be inferred from the conceptual design drawing that a lower ground floor is incorporated in the proposed buildings. Filling is expected to be limited to grading the site surface for light demountable buildings, pavement construction and installation of services.

8.4.1 Site Preparation

It is recommended that all filling be placed and compacted in accordance with Level 1 requirements (AS3798 – 2007). To prepare the site for the construction of new buildings, the following procedures are suggested.

- Stripping of vegetation and organic topsoils (to expected maximum depths of 0.3 m) and separately stockpiled for use in landscaping or removed off site;
- Stripping of uncontrolled fill and unsuitable material within the footprint of the proposed buildings and pavements. Inspection of the stripped surface by a geotechnical engineer;

- Compaction of the exposed surface with at least of 8 passes of a 12 tonne (minimum dead weight) roller, followed by test rolling in the presence of a geotechnical engineer. Where soft spots are identified, they should be excavated and then backfilled using a suitable granular material. Additional filling may also be required to elevate building platforms. All filling should be placed in 250 mm (loose thickness) layers and compacted with placement moisture contents within the range of -2% to +2% of OMC in order to limit surface deflection during proof rolling.
- Surface drainage should be maintained at all times by adopting appropriate cross-falls across the site. Surface drainage should be installed as soon as is practicable in order to capture and remove surface flows to prevent erosion and softening of the exposed surface.

Filling delivered to site must be approved by the geotechnical consultant prior to delivery to site. Highly reactive clay filling should be avoided.

Site observations and laboratory test results have indicated the presence of high plasticity silty clays in some areas which could be adversely affected by inclement weather. Whilst these soils are typically of a stiff to very stiff consistency when dry, they can rapidly lose strength during rainfall and subsequent partial saturation and result in difficult trafficability conditions.

Conventional sediment and erosion control measures should be implemented during the construction phase, with exposed surfaces to be topsoiled and vegetated as soon as practicable following the completion of earthworks.

8.4.2 Excavation

All topsoil, filling, natural soils and bedrock up to very low to low strength should be readily removed using a conventional medium sized excavator fitted with a toothed bucket possibly with some light ripping in the weathered bedrock. These conditions were generally encountered to depths of about 2.0 – 3.0 m within all borehole locations

The excavation is expected to include any moisture affected material within the footprint of demolished buildings and then extend further to the design level at the base of the lower ground level.

Where low to medium strength rock were encountered, these areas will, for the most part, be adequately removed during bulk earthworks using a large excavator with some light to medium ripping. However, larger plant may provide greater excavation efficiency particularly during drilling of pier foundations. Medium to high strength rock will offer greater resistance to light ripping. These areas will require pneumatic/hydraulic hammering equipment in combination with rock sawing and/or grinding to achieve the required cut depths.

Due to the proximity of surrounding buildings and presence of filling at shallow depth, the vibration resulting from the excavation could cause damage to the underground services or demountable and brick structures. It is recommended, if the use of percussive equipment is required within 40 m of any vibration sensitive structures, vibration monitoring should be undertaken. If the monitoring indicates unacceptable levels of vibration, then the use of non-percussive (i.e.: rock sawing and ripping) excavation methods will be required. This requirement however, will need to be determined on site once the details of the bulk earthworks and proposed excavation equipment are known.

Anticipated equipments required for excavations are given as a guide only. Rock strength and quality are expected to vary within the footprint of the proposed buildings. Assessment of excavation difficulties are best determined by intending contractors based on inspection of the core samples, the equipment they have at their disposal and the experience of the operators. For information on soil and rock types and indicative strength, reference must be made to the individual logs which are included in Appendix B.

8.4.3 Reuse of Excavated Materials

Generally, the filling, natural clays and bedrock of up to low strength encountered during the investigation, will be suitable for reuse as engineered filling within the site. The material should not contain any particle sizes greater than 150 mm as these may cause inadequate compaction, and should not contain silts due to their propensity for saturation and erosion. It is expected that the extremely weathered or low strength rock should readily break down beneath the weight of the rollers. However, bedrock of medium strength or higher may potentially need to be crushed using a rock crusher.

Topsoil and other deleterious materials will not be suitable as a fill material but could be stockpiled for potential use in landscaping or alternatively, removal from site.

8.4.4 Batter Slopes

While cut slopes within the clays may often stand vertically and unsupported (provided no nearby structures are present) for short periods of time, they will rapidly lose strength upon exposure to weather. A maximum batter slope of 1(H):1(V) is recommended for unsurcharged temporary slopes in stiff clays. The maximum batter slope should be reduced to 3(H):1(V) for temporary batters in uncontrolled filling.

Where the slopes are to be vegetated to prevent erosion, a maximum final batter slope of 3(H):1(V) is recommended. If batters greater than 4 m in height are required, the inclusion of a 3 m wide intermediate bench every 5 m in height, is recommended to reduce the effects of scour and erosion.

Where filling batters are formed, similar parameters to those recommended for cut slopes can be adopted. However, it is recommended that whilst the slope is being formed the batters should be over-filled in near-horizontal lifts and cut back to form the design grades.

8.5 Excavation Support

Once bulk excavations are required, temporary or permanent batters at recommended batter angles may not be feasible due to insufficient space for batters adjacent of the excavation.

The design of shoring will therefore be required for subsurface materials as batters steeper that those suggested in Section 8.4 are not expected to remain stable for a long period of time. The design should take account of the lateral loads due to adjacent structures.

Pending the final excavation depth, the following options may be adopted for retaining the excavations in this project. The feasible options would include either anchored soldier piles (drilled at maximum 2.4 m spacings) with close shuttering / shotcrete infill panels or contiguous piling. In the absence of details of adjacent footings being available, contiguous piles should be used for excavations adjacent to neighbouring buildings. Contiguous piling is the cheapest form of concrete pile wall, however, is not a water retaining structure and may not be suitable for any material due to gaps between piles.

Excavation of panels for shotcreting at anchored soldier piles option should be staged to allow a hit and miss approach with the first panel extending no more than 1.0 m below the base of the adjacent building foundation, including the reinforcement overlap. The next row of panels should not exceed 1.5 m with subsequent panels not exceeding 2 m in height.

Drainage is normally provided behind shotcrete walls. The sprayed concrete wall should provide adequate structural support, however it may be appropriate to install a false wall (single brickwork or block work) for aesthetic purposes and to avoid dampness. Care should be exercised in construction to ensure that anchors are installed progressively with excavation (and stressed up) and that the shotcreting is carried out at regular intervals to limit the exposed sections. The first row of anchors should be installed as high as possible and stressed up to 80% of its working load prior to excavation of the next row of panels.

A high capacity piling rig will be required to penetrate the high strength rock. Otherwise, the piers may refuse in the high strength rock, well above the excavation levels and additional anchors may need to be installed in the toe of each pier to provide support/restraint of the structure and rock mass.

As a result of moderately to steeply-inclined jointing especially in fractured siltstone and potential for 'wedge-type' failures within the batters, allowance will also need to be made for the support of the fractured rock where contiguous walling is not installed. The support requirements will depend on a number of factors including extent of disturbance during excavation; orientation (bearing), persistence (lateral continuity) and spacing (horizontal separation) of jointing; clay infilling of open jointing; and groundwater. As such, detailed design should be reviewed and verified by DP to ensure the allowance has been made for variable subsurface strata encountered.

As a guide, in addition to the soldier piles, preliminary design of infilled panel sections should allow for the application of a steel mesh-reinforced shotcrete layer with a minimum nominal thickness of 150 mm where permanent support is required or 75 mm for temporary support. Due to the highly fractured nature of the rock stratum at shallow depth, the installation of a rock bolts may be considered to support the temporary excavations batters based on inspections carried out by an engineering geologist. The final required bolt lengths can only be determined following assessment of fracture characteristics observed in the face.

Earth pressures acting on multi-anchored shoring structures and retaining walls can be estimated on the basis of a trapezoidal pressure distribution (i.e.: triangular to 0.25 H, uniform from 0.25 H to 0.75 H and triangular decreasing to zero from 0.75 H to H) with depth using appropriate values of bulk density and active (Ka) or 'at rest' (Ko) lateral earth pressure coefficients as set out in Table 4.

Table 4: Suggested Lateral Earth Pressure Design Parameters – Retaining Structures

Retained Material	Bulk Density (kN/m^3)	K_0	Ka	
			Short Term	Long Term
Stiff to hard clay and extremely weathered rock	20	0.6	0.25	0.3
Very low strength siltstone and sandstone	22	0.45	0.3	0.35
Medium strength or greater siltstone and sandstone	22	-	10 kPa*	10 kPa*

* A uniform pressure of 10 kPa should be adopted for the support of the medium strength sandstone to account for possible defects, but subject to inspection during the early stages of excavation to confirm bedding/jointing and revision of lateral restraint, if appropriate.

'At rest' pressure coefficients are appropriate where support must be provided to boundaries and where movement intolerant services or adjacent structures are present. Surcharge lateral pressure due to any adjacent structure will also need to be taken into account where the footings found on low strength or weaker rock or unfavourably orientated jointing is encountered.

The current investigation is not suggesting any indication of groundwater table to the limit of investigation. In the event that, tanked basement is required for this project, full hydrostatic pressure should be allowed for in design. As such, densities of the retained soils can be appropriately reduced to the buoyant values. Where applicable, superimposed surcharge loads due to adjacent driveways and developments should also be accommodated in the design of such structures.

Where appropriate, lateral restraint may also be developed by embedding piles below the base of the excavation and developing passive pressure. Suggested ultimate passive resistance values are given in Table 5 may be adopted below one pile diameter beneath the bulk excavation level and should incorporate a factor of safety to limit wall movement.

Table 5: Suggested Ultimate Passive Pressure Values

Material	Ultimate Passive Pressure (kPa)
Extremely low and very low strength siltstone	300
Low strength siltstone and sandstone	1200
Medium or greater strength siltstone and sandstone	4000

Where engineer-designed retaining walls are proposed, the following measures should be incorporated into the design:

- Backfilling of the void between the wall and the slope using imported, free draining granular material connected into a drainage pipe at the base of the wall;
- Capping of the backfill (where exposed) with compacted clay or concrete to prevent surface runoff entering the backfill;
- Provision of an open drain to collect and divert surface runoff from ponding above the wall;

- For horizontal backfill or retained soils, design based on an average bulk unit weight for retained material of 20 kN/m^3 and on a triangular earth pressure distribution based on an active earth pressure coefficient of (K_a) 0.3 for compacted filling and natural clay where no movement sensitive structures are located within a horizontal distance of 2H (where H is the vertical height of the retained zone) of the rear of the wall;
- Where there are movement sensitive structures located within the abovementioned critical zone, an at rest pressure coefficient (K_0) of 0.6 should be adopted; and
- If hydrostatic pressures are allowed, soil densities could be reduced to the buoyant values.

If an adequate drainage medium is not provided behind the retaining wall, then hydrostatic pressures must be incorporated within the design with soil parameters reduced to their buoyant values.

8.6 Earthquake Actions – Sub-soil Class

The site stratigraphy comprises minor filling and topsoil underlain by stiff to hard silty clays, overlying bedrock at depths ranging from 1.8 m to 2.3 m within the footprint of the proposed structure. Therefore, the site's sub-soil class when assessed in accordance with AS 1170.4 – 2007 (Ref 4) is considered a rock site and a classification of Class B_e is suggested.

9. Summary

The investigation included the drilling of three cored boreholes to a depth of 8.0 m within the proposed school site at the nominated locations by the client. The boreholes have indicated that subsurface conditions underlying the site generally comprise variable depths of filling and topsoil overlying silty clay and clay of very stiff to hard consistency. Rock was encountered in all boreholes on first contact at depths of between within the range 1.8 m to 2.3 m.

Bearing capacity recommendations are provided in Section 8.3. The site preparation, earthworks and excavation support recommendations are to be undertaken in accordance with Sections 8.4 and 8.5.

Consideration must be given to the preliminary nature of the investigation and potential for variability in the subsurface condition across the site. Once design is suitably advanced, DP must review the plans to determine if the comments given within are appropriate or if additional investigations are required.

10. References

1. Geology of 1:100 000 Wollongong – Port Hacking Geological Series Sheet No 9029 – 9129, Dept of Mines, (1985).
2. Australian Standard AS 2159 – 2009 "*Piling – Design and Installation*".
3. Foundations on Shales and Sandstones in the Sydney Region, Pells *et al*, Australian Geomechanics Journal (1998).
4. AS 1170.4 – 2007, "Structural Design Actions – Part 4: Earthquake Actions in Australia".

5. AS 1170.4 – 1993, "Structural Design Actions – Part 4: Earthquake Actions in Australia".
6. AS 3798 – 2007, "Guidelines on Earthworks for Commercial and Residential Developments".

11. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report (or services) for the proposed redevelopment works at Picton High School in accordance with DP's email proposal dated 8 June 2017 and acceptance received from Mr Shane Wood dated 30 June 2017. The work was carried out under projects General Terms and Conditions. This report is provided for the exclusive use of Billard Leece Partnership Pty Ltd for this project only and for the purposes as described in the report. It should not be used for other projects 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.

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

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

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

Douglas Partners Pty Ltd

Appendix A

About This Report
Drawing 1

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.

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

Rock Descriptions

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 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx 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 $Is_{(50)}$

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 loner 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:

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

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

Sampling Methods

Douglas Partners

Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, 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.

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Billard Leece Partnership Pty Ltd	
OFFICE: Macarthur	DRAWN BY: IKA
SCALE: As shown	DATE: 02.03.17

TITLE: **Borehole Locations**
Proposed Picton High School Development
480 Argyle Street, Picton, NSW

N
MGA

PROJECT No: 34252.02
DRAWING No: 4-1
REVISION: 1

Appendix B

Borehole Logs (Bores 101 – 103)

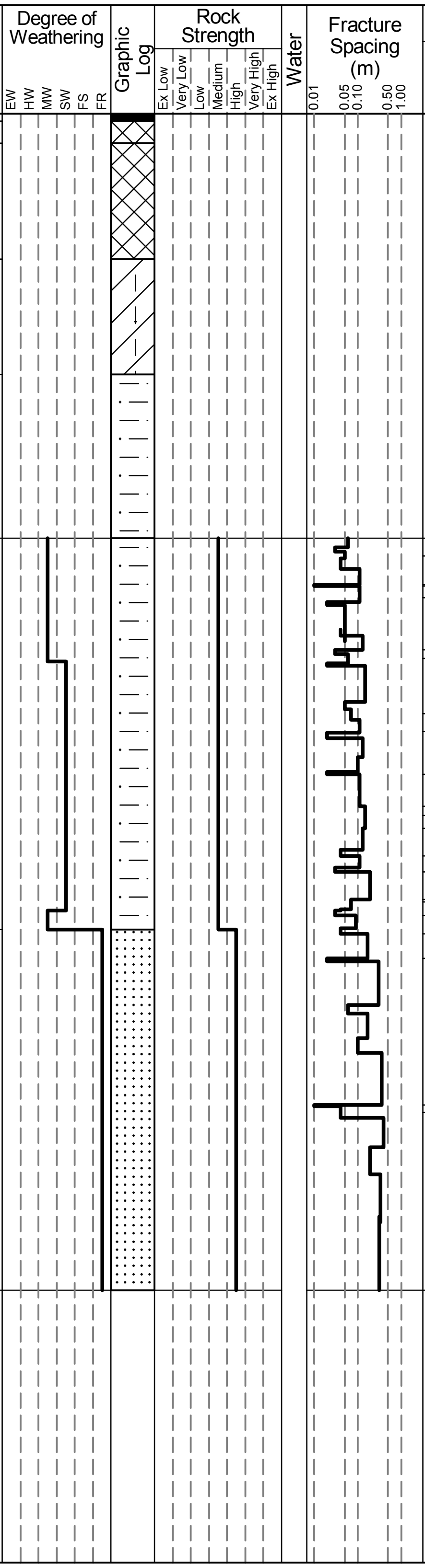
BOREHOLE LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 217.7 mAHD
EASTING: 279469
NORTHING: 6213685
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 34252.02
DATE: 10/7/2017
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE					
0.2	FILLING - dark brown gravelly sand, moist		D			
	FILLING - brown silty clay with some sand and gravel, MC~PL		D			
1.0	SILTY CLAY - very stiff, grey brown silty clay with low strength, highly weathered iron indurated shale bands	Note: Unless otherwise stated, rock is fractured along planar, smooth, iron stained bedding planes dipping at 0-10°	D S			6,8,9 N = 17
1.8	SILTSTONE - low strength, highly weathered, grey siltstone		D D			
2.93	SILTSTONE - medium strength, slightly weathered, fractured, grey siltstone - becoming slightly weathered below 3.78m - becoming moderately weathered below 5.5m	3m: Cs 40mm thick 3.05m: Cs 45mm thick 3.25m: J, 85°, sv, cu, ro, cln, 30mm 3.26m: B, 4°, sh, pl, sm, fe stn 3.34m: J, 85°, cu, ro, cln 100mm long 3.7m: Cs 60mm thick 3.76m: J, 75°, cu, ro, clay co 50mm long 4.14m: J, 75°, un, sm, cln 80mm long 4.26m: J, 75°, cu, sm, cln 50mm long 4.56m: J, 25°, pl, sm, clay co 4.78m: J, 25°, pl, sm, clay co 4.84m: J, v, pl, sm, vn, clay 60mm 4.93m: J, 25°, pl, ro, cln 5.12m: J, 25°, pl, ro, cln 5.23m: J, 25°, pl, sm, fe stn 5.42m: J, 25°, pl, sm, cln	C C C	100 100 100	61 73 63	PL(A) = 0.71 PL(A) = 0.67 PL(A) = 0.38 PL(A) = 0.5 PL(A) = 0.43
5.63	SANDSTONE - high strength, fresh stained, fractured to slightly fractured, grey brown to red brown fine grained sandstone - becoming slightly fractured, medium grained below 6.89m	5.44m: J, 85°, cu, cm cln 90mm long 5.53m: J, 25°, sh, pl, sm, clay co 5.66m: J, 45°, pl, ro, fe stn 5.83m: J, 60°, pl, sm, fe stn 6.84m: J, 85°, un, ro, fe stn 170mm long 6.89m: J, 25°, pl, sm, fe stn	C C	100 100	91 100	PL(A) = 2.77 PL(A) = 1.21
8.12	Bore discontinued at 8.12m - limit of investigation					



RIG: Custom Christie Eng. Trailer Rig **DRILLER:** BG Drilling **LOGGED:** JHB **CASING:** HW to 2.8m
TYPE OF BORING: 110mm auger to 2.8m, NMLC to 8.12m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

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)

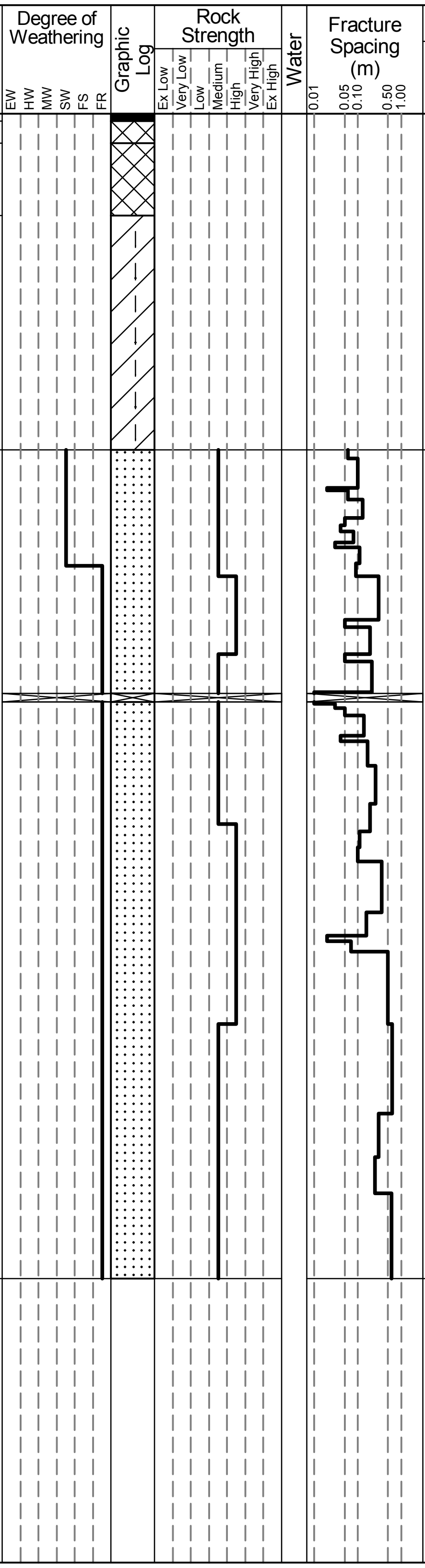
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 214.6 mAHD
EASTING: 279580
NORTHING: 6213660
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 34252.02
DATE: 10/7/2017
SHEET 1 OF 1



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	0.05	ASPHALTIC CONCRETE		D			
	0.2	FILLING - dark brown gravelly sand, moist		D			
214	0.7	FILLING - grey brown silty clay, MC~PL					
	1	SILTY CLAY - stiff, grey brown silty clay, MC~PL	Note: Unless otherwise stated, rock is fractured along planar, smooth, clay coated or iron stained bedding planes dipping at 0-10°	D			pp = 300-400
				S			4,7,8 N = 15
213		- with low strength, highly weathered shale bands below 1.5m					
	2			D			
212	2.32	SANDSTONE - medium strength, moderately weathered, fractured, red brown to grey brown medium grained sandstone	2.38m: J, 25°, cu, ro, clay co	C	100	70	PL(A) = 0.73
	3	- becoming high strength, fractured to slightly fractured below 3.19m		C	95	62	PL(A) = 1.03
211		- becoming medium strength below 3.73m					
	4 4.0 4.06		4m: CORE LOSS: 60mm				PL(A) = 0.8
210	5	- becoming high strength below 4.9m		C	100	94	PL(A) = 1.41
209	6	- becoming slightly fractured below 5.78m					PL(A) = 1.25
208		- becoming medium strength below 6.28m	6.28m: J, 40°, cu, ro, clay co	C	100	93	
	7						PL(A) = 0.82
207			7.45m: B, 10°, sh, pl, sm, cbs co	C	100	100	PL(A) = 0.97
	8 8.04	Bore discontinued at 8.04m - limit of investigation					
206	9						
205							

RIG: Custom Christie Eng. Trailer Rig **DRILLER:** BG Drilling **LOGGED:** JHB **CASING:** HW to 2.2m
TYPE OF BORING: 110mm auger to 2.2m, NMLC to 8.04m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

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)

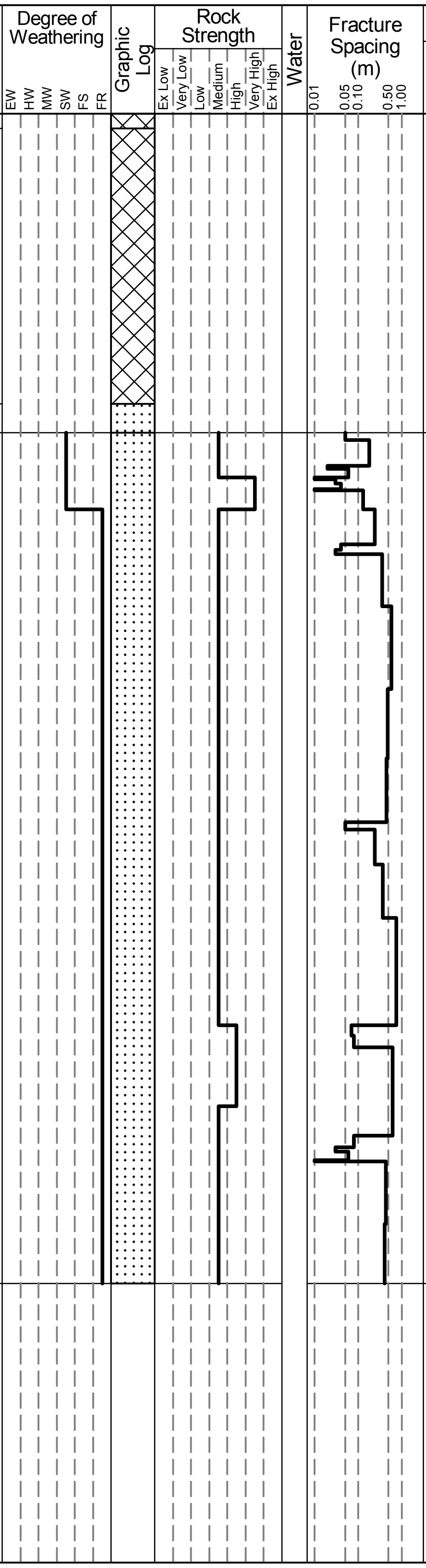
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 215.6 mAHD
EASTING: 279677
NORTHING: 6213737
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 34252.02
DATE: 11/7/2017
SHEET 1 OF 1



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	0.1	FILLING - brown silty clay (topsoil)		D			
215		FILLING - appears poorly compacted, grey brown silty clay with a trace of gravel, MC~PL		D			
	1		Note: Unless otherwise stated, rock is fractured along planar to curved, smooth, clay coated bedding planes dipping at 0-15°	D			
214				S			4,4,3 N = 7
	2 2.0	SANDSTONE - extremely low to very low strength, brown and grey medium grained sandstone					
213	2.2	SANDSTONE - medium strength, moderately weathered, fractured, red brown to light grey medium grained sandstone with very high strength band		C	100	72	
			2.43m: J, 25°, sh, pl, ro, clay co	D			PL(A) = 3.95
	3	- becoming fresh stained, slightly fractured below 3.04m	3.04m: Cs 10mm thick	C	100	83	PL(A) = 0.93
212	4						
211			4.45m: J, 28°, sh, pl, sm, cln	C	100	97	PL(A) = 0.94
	5		5.18m: B, 15°, sh, pl, sm, cln				
210							PL(A) = 0.82
	6	- becoming high strength below 6.29m	6.36m: J, 6°, sh, pl, ro, cln	C	100	90	PL(A) = 1.42
209	7	- becoming medium strength below 6.85m					PL(A) = 0.98
208				C	100	82	
	8 8.07	Bore discontinued at 8.07m - limit of investigation					PL(A) = 0.52
207	9						
206							

RIG: Custom Christie Eng. Trailer Rig **DRILLER:** BG Drilling **LOGGED:** JHB **CASING:** HW to 2.2m
TYPE OF BORING: 110mm auger to 2.2m, NMLC to 8.07m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

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

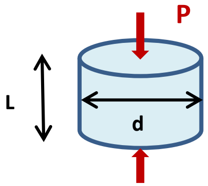
Point Load Test Results

AXIAL POINT LOAD TEST - ALONG CORE AXIS

Project: Proposed Picton High School Redevelopment
Client: Billard Leece Partnership Pty Ltd
Date: 12/07/2017

Project No. 34252.02
Tested by: Joel Brauer
Page: 1

Bore	Depth (m)	Rock Type	Diameter of core (mm)	Length of Core (mm)	Failure Load (kN)	Equivalent Area	Equivalent diameter^2	Uncorrected Point Load Strength	Equivalent diameter	Correction Factor	Point Load Strength	Estimated Strength
			d	L	P	A = dxL	$De^2 = 4A/\pi$	$Is = (Px1000)/De^2$	De	$F = (De/50)^{0.45}$	$Is_{(50)} = Fx\ Is$	
1	3.14	Siltstone	50	41	1.84	2050	2610	0.70	51.09	1.01	0.71	M
1	3.96	Siltstone	50	41	1.72	2050	2610	0.66	51.09	1.01	0.67	M
1	4.6	Siltstone	50	31	0.79	1550	1974	0.40	44.42	0.95	0.38	M
1	5.36	Siltstone	50	22	0.79	1100	1401	0.56	37.42	0.88	0.50	M
1	5.5	Siltstone	50	21	0.66	1050	1337	0.49	36.56	0.87	0.43	M
1	6.16	Sandstone	50	41	7.17	2050	2610	2.75	51.09	1.01	2.77	H
1	7.77	Sandstone	50	43	3.25	2150	2737	1.19	52.32	1.02	1.21	H
2	2.84	Sandstone	50	36	1.71	1800	2292	0.75	47.87	0.98	0.73	M
2	3.51	Sandstone	50	40	2.61	2000	2546	1.02	50.46	1.00	1.03	H
2	4.1	Sandstone	50	47	2.31	2350	2992	0.77	54.70	1.04	0.80	M
2	5	Sandstone	50	46	3.99	2300	2928	1.36	54.12	1.04	1.41	H
2	6	Sandstone	50	32	2.64	1600	2037	1.30	45.14	0.95	1.24	H
2	7.41	Sandstone	50	35	1.87	1750	2228	0.84	47.20	0.97	0.82	M
2	7.91	Sandstone	50	29	1.91	1450	1846	1.03	42.97	0.93	0.97	M
3	2.55	Sandstone	50	37	9.44	1850	2355	4.01	48.53	0.99	3.95	VH
3	3.37	Sandstone	50	29	1.84	1450	1846	1.00	42.97	0.93	0.93	M
3	4.42	Sandstone	50	32	2.01	1600	2037	0.99	45.14	0.95	0.94	M



for valid tests $0.3d < L < d$

AXIAL POINT LOAD TEST - ALONG CORE AXIS

Project: Proposed Picton High School Redevelopment

Client: Billard Leece Partnership Pty Ltd

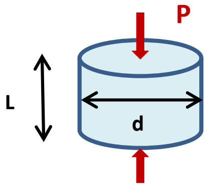
Date: 12/07/2017

Project No. 34252.02

Tested by: Joel Brauer

Page: 2

Bore	Depth (m)	Rock Type	Diameter of core (mm)	Length of Core (mm)	Failure Load (kN)	Equivalent Area	Equivalent diameter^2	Uncorrected Point Load Strength	Equivalent diameter	Correction Factor	Point Load Strength	Estimated Strength
			d	L	P	A = dxL	$De^2 = 4A/\pi$	$Is = (Px1000)/De^2$	De	$F = (De/50)^{0.45}$	$Is_{(50)} = Fx\ Is$	
3	5.51	Sandstone	50	25	1.44	1250	1592	0.90	39.89	0.90	0.82	M
3	6.43	Sandstone	50	36	3.32	1800	2292	1.45	47.87	0.98	1.42	H
3	7	Sandstone	50	49	2.91	2450	3119	0.93	55.85	1.05	0.98	M
3	8.03	Sandstone	50	44	1.41	2200	2801	0.50	52.93	1.03	0.52	M



for valid tests 0.3d < L < d

Appendix D

Aggressivity Test Results

12 Ashley Street, Chatswood, NSW 2067
tel: +61 2 9910 6200

email: sydney@envirolab.com.au
envirolab.com.au

Envirolab Services Pty Ltd - Sydney | ABN 37 112 535 645

CERTIFICATE OF ANALYSIS 171268

Client:

Douglas Partners Pty Ltd Smeaton Grange
18 Waler Crescent
Smeaton Grange
NSW 2567

Attention: Tom Mrdjen, Joel Brauer

Sample log in details:

Your Reference:	**34252.02, Picton H.S**
No. of samples:	3 soils
Date samples received / completed instructions received	13/07/17 / 13/07/17

Analysis Details:

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

Report Details:

Date results requested by: / Issue Date:	20/07/17 / 20/07/17
Date of Preliminary Report:	Not Issued

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025 - Testing **Tests not covered by NATA are denoted with *.**

Results Approved By:

David Springer
General Manager

NATA
ACCREDITED FOR
TECHNICAL
COMPETENCE

Misc Inorg - Soil				
Our Reference:	UNITS	171268-1	171268-2	171268-3
Your Reference	-------------	BH101: SPT 1.0	BH101: C 3.1-3.4	BH102: SPT 1.0
Depth	------------	1.0	3.1-3.4	1.0
Date Sampled		10/07/2017	10/07/2017	10/07/2017
Type of sample		Soil	Soil	Soil
Date prepared	-	17/07/2017	17/07/2017	17/07/2017
Date analysed	-	17/07/2017	17/07/2017	17/07/2017
pH 1:5 soil:water	pH Units	5.9	9.0	5.3
Electrical Conductivity 1:5 soil:water	µS/cm	36	190	210
Chloride, Cl 1:5 soil:water	mg/kg	10	37	210
Sulphate, SO4 1:5 soil:water	mg/kg	21	<10	27

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

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			17/07/2017	171268-3	17/07/2017 || 17/07/2017	LCS-1	17/07/2017
Date analysed	-			17/07/2017	171268-3	17/07/2017 || 17/07/2017	LCS-1	17/07/2017
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	171268-3	5.3 || 5.1 || RPD: 4	LCS-1	103%
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	171268-3	210 || 220 || RPD: 5	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	171268-3	210 || 200 || RPD: 5	LCS-1	99%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	171268-3	27 || 20 || RPD: 30	LCS-1	110%

Report Comments:

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

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

Quality Control Definitions

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

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

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

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

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

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

Appendix E

Preliminary Geotechnical Investigation Report (34252.02.R.001)

Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Preliminary Geotechnical Investigation

Proposed Picton High School Redevelopment
Argyle Street, Picton

Prepared for
Billard Leece Partnership Pty Ltd

Project 34252.02
March 2017

Douglas Partners
Geotechnics / Environment / Groundwater

Document History

Document details

Project No.	34252.02	Document No.	R.001
Document title	Report on Preliminary Geotechnical Investigation Proposed Picton High School Redevelopment		
Site address	Argyle Street, Picton		
Report prepared for	Billard Leece Partnership Pty Ltd		
File name	34252.02.P.001.Rev0		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	R Machiani	G W McIntosh	16 March 2017

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Billard Leece Partnership Pty Ltd

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.

	Signature	Date
Author		16 March 2017
Reviewer	for G.W.M	16 March 2017

bsi. ISO 9001 Quality Management

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Table of Contents

Report on Preliminary Geotechnical Investigation
Proposed Picton High School Redevelopment
Argyle Street, Picton

1. Introduction

This report presents the results of a preliminary geotechnical investigation undertaken for proposed redevelopment works within Picton High School at Argyle Street, Picton. The investigation was commissioned in an email dated 12 January 2017 by Mr Shane Wood of Billard Leece Partnership Pty Ltd (Architects) and was undertaken in accordance with Douglas Partners' (DP) proposal MAC1600384 dated 21 November 2016.

DP understands that the proposed redevelopment works will include the removal of some demountable buildings across the site and the construction of new teaching blocks, associated facilities and pavements. The detailed design information of proposed new permanent buildings and cut-fill plans are yet to be finalized.

The investigation included the excavation of test pits and laboratory testing of selected samples. Details of the work undertaken and the results obtained are given within this report, together with comments relating to design and construction practice.

The investigation discussed within was undertaken concurrently with a Contamination Assessment (Project 34252.02.P.002) and Hazmat Survey (Project 34252.02.P.003), both of which will be reported separately.

2. Site Description

Picton High School is located approximately 90 km to the south-west of the Sydney CBD and is a rectangular shaped area of some 6 ha. Maximum north-south and east-west dimensions are approximately 200 m and 290 m respectively. The school site is bounded by Argyle Street to the west, residential properties to the north, and vacant land to the south and east. The school is currently occupied by 32 existing buildings comprising permanent and demountable structures of various sizes, a car park and playing fields.

The school site is located within undulating rises with overall topographic relief of approximately 8 m from the highest parts (approximately RL218 m, relative to the Australian height datum) within the eastern and western portions to the lowest part (approximately RL 210 m) within the mid northern portion of the site. However, it has been partially levelled by cutting and filling to create flat platforms for the existing structures.

Approximately, two-thirds of the site is covered by the existing structures and carparks. These structures are generally located within the western portion of the site extending toward the middle portion. Two sporting fields are noted along the eastern and southern boundaries of the school site. At the time of the investigation, the vegetation across these open spaces was limited to well-maintained light grass. Medium size trees were present along the northern boundary with scattered trees noted between the existing buildings.

3. Regional Geology

Reference to the Wollongong-Port Hacking 1:100,000 Geological Sheet (Ref 1) indicates that the site is underlain by Ashfield Shale (mapping unit Rwa) of the Wianamatta Group of Triassic age. The Ashfield Shale typically comprises shale, siltstone, claystone and laminite with coal bands, all of which weathered to form clays of high plasticity. The results of the investigation were consistent with the geological mapping, with shale and siltstone of variable weathering with seams of fine grained lithic sandstone encountered in the test pits.

4. Field Work Methods

The field work comprised the excavation of 10 test pits (Pits 1 – 10) to depths of 0.5 – 2.5 m with a Takeuchi TB145 excavator fitted with a 300 mm bucket. The fieldwork was undertaken by a geotechnical engineer who collected undisturbed samples (in 50mm thin walled tubes), disturbed samples and bulk samples to assist in strata identification and for laboratory testing. Following logging, testing and sampling, each test pits were backfilled and the ground surface reinstated to its previous level.

The test pit locations were nominated by the client and located on site prior to the investigation using differential GPS unit for which an accuracy of ± 20 mm is typical. The location of test pits are shown on Drawing 1 (Appendix A). The surface levels were obtained using the differential GPS unit.

All field measurements and mapping for this project have been carried out using the Geodetic Datum of Australia 1994 (GDA94) and the Map Grid of Australia 1994 (MGA94). All reduced levels are given in relation to Australian Height Datum (AHD).

5. Field Work Results

The test pit logs are included in Appendix B, and should be read in conjunction with the accompanying standard notes that define classification methods and descriptive terms. Relatively uniform conditions were encountered underlying the site with the general succession of strata broadly summarised as follows:

- TOPSOIL – generally brown / grey silty clay (topsoil filling in Pits 1 – 4 & 7) with rootlets and trace gravel to depths 0.1 – 0.3 m in all test pits with the exception of Pits 5 and 6;
- FILLING – generally brown silty clay with some gravel to depths of 0.4 – 1.0 m in Pits 1 – 7;
- CLAY – very stiff to hard red/brown silty clay with seams of extremely weathered shale in Pits 1 – 5 & 8 to depths within the range 0.9 – 2.3 m; and

- BEDROCK – extremely low strength to medium strength shale and sandstone generally at depths within the range 0.9 m to 2.3 m in all test pits, other than Pits 9 and 10 where extremely low strength shale was directly underlying the topsoil.

No free groundwater was observed in the test pits during excavation and for the short time that they were left open. It is noted, however, that the test pits were immediately backfilled following logging and sampling which precluded longer term monitoring of any groundwater levels that might be present. It's noted that groundwater levels are affected by factors such as weather conditions and can fluctuate with time.

6. Laboratory Testing

Selected samples from the test pits were tested in the laboratory for measurement of field moisture content, Atterberg limits, shrink-swell and California bearing ratio (CBR). The CBR tests were carried out on samples compacted to approximately 100% dry density ratio relative to standard compaction at standard optimum moisture content. The samples were then soaked for four days under surcharge loadings of 4.5 kg. The detailed laboratory test report sheets are given in Appendix C, with the results summarised in Table 1 – 3.

Table 1: Results of Atterberg Limits Testing

Pit No.	Depth (m)	W_F (%)	W_L (%)	W_P (%)	PI (%)	LS (%)	Material
2	2.0	15.3	38	17	21	8.0	Clay
3	1.5	21.4	62	26	36	11.5	Clay
4	0.2 – 0.5	17.7	43	24	19	8.5	Filling
7	0.5 – 0.7	6.3	27	20	7	5.0	Filling
8	0.5 – 0.6	11.8	65	25	40	15.0	Gravelly clay
10	0.5	12.5	55	22	33	11.0	Shale

Where W_F = Field moisture content
W_L = Liquid limit
LS = Linear shrinkage
W_P = Plastic limit
PI = Plasticity Index

The results indicate that the natural clays encountered on site appear to be of intermediate to high plasticity and as such, would be expected to be susceptible to shrinkage and swelling movements due to seasonal moisture variations.

Table 2: Results of Shrink Swell Testing

Pit No.	Depth (m)	W_F (%)	I_{ss} (%/ΔpF)	Material
6	0.5 – 0.9	8.3	0.3	Filling
8	0.3 – 0.7	19.9	0.4	Gravelly clay

Where W_F = Field moisture content
I_{ss} = Shrink-Swell Index

The shrink-swell index (I_{ss}) test results indicate the gravelly clays are of low shrink-swell potential due to changes in soil moisture content. However, considering the results of Atterberg limits and linear shrinkage tests on the fine grained portions (passing 0.075 mm) of same material, a moderate shrink-swell potential is suggested for silty clays underlying the site.

Table 3: Results of California Bearing Ratio Testing

Pit No	Depth (m)	FMC (%)	OMC (%)	MDD (t/m^3)	Swell (%)	CBR (%)	Material
1	0.5 – 0.7	17.6	22.8	1.49	0.7	4	Filling
2	0.5 – 0.7	18.5	25.5	1.55	0.2	3.5	Filling
3	0.5 – 0.7	15.0	21.2	1.60	0.2	6	Gravelly clay
7	0.3 – 0.5	9.3	14.0	1.85	0.7	3.5	Filling
9	0.5 – 0.7	9.8	15.2	1.77	1.1	6	Shale

Where FMC = Field moisture content OMC = Optimum moisture content
MDD = Maximum dry density CBR = California bearing ratio

The results of the field moisture content tests (at the time of the sampling) listed in Table 3 indicate the soils ranged between approximately 4.7 – 7% dry of standard optimum moisture content.

7. Proposed Development

It is understood that the redevelopment works comprising the removal of selected demountable buildings within the site and the construction of new permanent teaching blocks are proposed. Some of the demountable buildings are proposed to be relocated as a part of this project. The proposed permanent buildings are likely to be one or two storey relatively lightweight structures and are expected to be founded on pads/shallow piers constructed within suitable natural material or controlled filing. However, the quantity, locations, design loads and other design information of the structures are unknown at this time. As parts of the redevelopment works, the construction of access roads and installation of services will also be required.

8. Comments

8.1 General

The following comments are based on the surface and subsurface profiles encountered in the test pits. Comments are provided in the following sections on development constraints related to geotechnical and geological factors to assist in the foundation design of the proposed new buildings. As detailed design of the proposed redevelopment works has not been undertaken, the comments given must also be considered as being preliminary in nature. Once details are available, they should be forwarded to DP for review to determine if comments given within this report are appropriate or require revision.

8.2 Subsurface Conditions

The investigation findings have indicated that near-surface conditions underlying the site generally comprise topsoil and filling to depths 0.3 – 1.0 m, overlying silty clay and stiff to hard gravelly clay in all test pits except for Pits 9 and 10 where topsoil was directly underlain by extremely weathered shale. Bedrock comprising weathered shale and lithic sandstone were found in all other test pits on first contact at depths of 0.9 – 1.8 m and continued to the auger refusal depths within the range 0.5 – 2.5 m.

8.3 Site Classification

It is inferred that fill material found throughout the site was not placed in accordance with recognised standards and would be considered 'uncontrolled' in accordance with the requirements of AS3798 (Ref 2), unless documents indicating the fill material has been placed in a controlled manner supplied by the client.

Based on the subsurface conditions encountered during the investigation, due to presence of uncontrolled filling deeper than 0.4 m, the site would be classified as Class P in accordance with AS 2870 (Ref 3). The natural soils underlying the site would be equivalent to Class M as described in AS 2870 (Ref 3).

8.4 Earthworks

8.4.1 Site Preparation

It is recommended that all filling be placed and compacted in accordance with Level 1 requirements (AS3798 – 2007). To prepare the site for the construction of new buildings, the following procedures are suggested.

- Stripping of vegetation and organic topsoils (to expected maximum depths of 0.3 m) and separately stockpiled for use in landscaping or removed off site;
- Stripping of uncontrolled fill and unsuitable material within the footprint of the proposed buildings. Inspection of the stripped surface by a geotechnical engineer;
- Compaction of the exposed surface with at least of 8 passes of a 12 tonne (minimum dead weight) roller, followed by test rolling in the presence of a geotechnical engineer. Where soft spots are identified, they should be excavated and then backfilled using a suitable granular material. Additional filling may also be required to elevate building platforms. All filling should be placed in 250 mm (loose thickness) layers and compacted with placement moisture contents within the range of -2% to +2% of OMC in order to limit surface deflection during proof rolling.
- Surface drainage should be maintained at all times by adopting appropriate cross-falls across the site. Surface drainage should be installed as soon as is practicable in order to capture and remove surface flows to prevent erosion and softening of the exposed surface.

Filling delivered to site must be approved by the geotechnical consultant prior to delivery to site. Highly reactive clay filling should be avoided.

Site observations and laboratory test results have indicated the presence of high plasticity silty clays in some areas which could be adversely affected by inclement weather. Whilst these soils are typically of a stiff to very stiff consistency when dry, they can rapidly lose strength during rainfall and subsequent partial saturation and result in difficult trafficability conditions.

Conventional sediment and erosion control measures should be implemented during the construction phase, with exposed surfaces to be topsoiled and vegetated as soon as practicable following the completion of earthworks.

8.4.2 Excavation

All topsoil, filling, natural soils and bedrock up to very low to low strength should be readily removed using a conventional medium sized excavator with a toothed bucket.

The final earthworks plans have not been finalized at the time of preparing this report. The excavation are expected to be limited to the removal of moisture affected material within the footprint of demolished buildings and replacing by suitable filling and drilling for piers or footing of new structures and installation of services. Bucket refusal on weathered rock was encountered in all test pits. Where low to medium strength rock was encountered, these areas will, for the most part, be adequately removed during bulk earthworks using a large excavator with some light to medium ripping. However, larger plant may provide greater excavation efficiency particularly during drilling of pier foundations.

Medium to high strength rock was not encountered in the test pits. If encountered during detailed excavation for services or foundations, these areas will offer greater resistance to light ripping and are likely to require pneumatic/hydraulic hammering equipment in combination with rock sawing and/or grinding to achieve the required cut depths for this project.

Due to the proximity of surrounding buildings and presence of filling at shallow depth, the vibration resulting from the excavation could cause damage to the underground services or demountable and brick structures. It is recommended, if the use of percussive equipment is required within 40 m of any vibration sensitive structures, vibration monitoring should be undertaken. If the monitoring indicates unacceptable levels of vibration, then the use of non-percussive (ie: rock sawing and ripping) excavation methods will be required. This requirement however, will need to be determined on site once the details of the bulk earthworks and proposed excavation equipment are known.

Anticipated equipments required for excavations are given as a guide only. Additional drilling investigation within the footprint of proposed structures is recommended to more accurately define the interface of filling, natural soil and provide quantitative information on the rock material properties where deep excavation within the rock profiles are expected. Where rock is encountered at design finished surface level, it is recommended that a minimum of 300 mm of topsoil be placed over the surface in order to better promote revegetation of the surface.

For information on soil and rock types and indicative strength, reference must be made to the individual logs which are included in Appendix B. Tenderers must make their own assessment of excavation condition with the information given on the test pit logs provided as preliminary information only.

8.4.3 Reuse of Excavated Materials

Generally, the filling, natural clays and bedrock of up to low strength encountered during the investigation, will be suitable for reuse as engineered filling within the site. The material should not contain any particle sizes greater than 150 mm as these may cause inadequate compaction, and should not contain silts due to their propensity for saturation and erosion. Topsoil and other deleterious materials will not be suitable as a fill material but could be stockpiled for potential use in landscaping or alternatively, removal from site.

8.4.4 Batter Slopes

While cut slopes within the clays may often stand vertically and unsupported (provided no nearby structures are present) for short periods of time, they will rapidly lose strength upon exposure to weather. A maximum batter slope of 1(H):1(V) is recommended for unsurcharged temporary slopes in stiff clays. The maximum batter slope should be reduced to 3(H):1(V) for temporary batters in uncontrolled filling.

Where the slopes are to be vegetated to prevent erosion, a maximum final batter slope of 3(H):1(V) is recommended. If batters greater than 4 m in height are required, the inclusion of an intermediate bench every 5 m in height, approximately 3 m wide, is recommended to reduce the effects of scour and erosion.

Where filling batters are formed, similar parameters to those recommended for cut slopes can be adopted. However, it is recommended that whilst the slope is being formed the batters should be over-filled in near-horizontal lifts and cut back to form the design grades.

8.5 Retaining Walls

Where engineer-designed retaining walls are proposed, the following measures should be incorporated into the design:

- Backfilling of the void between the wall and the slope using imported, free draining granular material connected into a drainage pipe at the base of the wall;
- Capping of the backfill (where exposed) with compacted clay or concrete to prevent surface runoff entering the backfill;
- Provision of an open drain to collect and divert surface runoff from ponding above the wall;
- For horizontal backfill or retained soils, design based on an average bulk unit weight for retained material of 20 kN/m^3 and on a triangular earth pressure distribution based on an active earth pressure coefficient of (K_a) 0.3 for compacted filling and natural clay where no movement sensitive structures are located within a horizontal distance of 2H (where H is the vertical height of the retained zone) of the rear of the wall;
- Where there are movement sensitive structures located within the abovementioned critical zone, an at rest pressure coefficient (K_0) of 0.6 should be adopted;
- If hydrostatic pressures are allowed, soil densities could be reduced to the buoyant values.

If an adequate drainage medium is not provided behind the retaining wall, then hydrostatic pressures must be incorporated within the design with soil parameters reduced to their buoyant values.

8.6 Footings

The proposed redevelopment is expected to comprise one and two storey buildings. It is anticipated that the buildings will be of relatively light weight constrcution.

Design of footings for the structures can only be undertaken once the final design loads and finished levels have been determined. As a guide however and based on the results of the subsurface investigation and the range of soils encountered, preliminary footing design could be based on the parameters presented in Table 4.

Table 4: Preliminary Footing Design Parameters

Material	Allowable Base Bearing Pressures (kPa)
Stiff clay or controlled filling	150
Very stiff clays or stronger	200
Weathered rock	500

8.7 Subgrade Parameters

The results of laboratory testing on the samples tested are included in Table 3. The laboratory testing gave CBR values within the range 3.5% – 4% for filling and CBR values of 4% and 6% for gravelly clay and extremely weathered shale respectively.

To allow for natural variations in subsurface conditions, it is suggested that a design CBR value of 3.5% be adopted as a basis of pavement design.

Drainage measures should be adopted to ensure that the subgrade and pavements do not become saturated in service. The exposed subgrade should be closely inspected at the time of construction to ensure that material of lower than the assumed design strength does not support the pavement at any locations. Should weaker subgrade material be encountered, consideration should be given to removing and replacing the weak strata with a higher material, or reassessing the pavement design.

Effective erosion and sedimentation control measures should be installed maintained for the duration of the construction. Furthermore, adequate drainage of all working areas shall be maintained throughout the period of construction to ensure run-off of water without ponding except where ponding forms part of a planned erosion and sedimentation control system.

To promote long term performance of the pavements, sub soil drainage and related features should also be considered to minimise moisture ingress and subsequent pavement failure.

9. Summary

The investigation included the excavation of test pits within the proposed school site at the nominated locations by the client. The collected undisturbed and bulk soil samples were returned to out NATA accredited laboratory for measurement of field moisture content, plasticity, shrink swell Index and CBR value of subsurface material.

The test pits have indicated that subsurface conditions underlying the site generally comprise variable depths of filling and topsoil overlying silty clay and clay of very stiff to hard consistency. Rock was encountered in all test pits on first contact at depths of between within the range 0.3 m to 2.3 m.

The site preparation and earthworks are to be undertaken in accordance with Section 8.4. The site has been classified Class P due to presence of uncontrolled filling deeper than 0.4 m and existing structures. The preliminary bearing capacity parameters for the design of footings are given in Table 4.

The results of CBR testing indicate the CBR values within the range 3.5 – 6% for near-surface material underlying the site. It is suggested that a design CBR value of 3.5% be adopted as a basis of pavement design.

Consideration must be given to the preliminary nature of the investigation and potential for variability in the subsurface condition across the site. Once design is suitably advanced, DP must review the plans to determine if the comments given within are appropriate or if additional investigations are required.

10. References

1. Geology of 1:100 000 Wollongong – Port Hacking Geological Series Sheet No 9029 – 9129, Dept of Mines, (1985).
2. Australian Standard AS 3798 – 2007 *Guidelines on Earthworks for Commercial and Residential Developments.*
3. Australian Standard AS 2870 – 2011 *Residential Footings and Slabs.*
4. AUSTROADS, *"Guide to Pavement Technology – Part 2: Pavement Structural Design",* 2012.

11. Limitations

Douglas Partners (DP) has prepared this report (or services) for the proposed redevelopment works at Picton High School in accordance with DP's proposal dated 21 November 2016 and acceptance received from Mr Shane Wood dated 12 January 2017. The work was carried out under projects General Terms and Conditions. This report is provided for the exclusive use of Billard Leece Partnership Pty Ltd for this project only and for the purposes as described in the report. It should not be used for other projects or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

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

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

Douglas Partners Pty Ltd

Appendix A

About This Report
Drawing 1

About this Report

Douglas Partners

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

Cone Penetration Tests

Douglas Partners

Introduction

The Cone Penetration Test (CPT) is a sophisticated soil profiling test carried out in-situ. A special cone shaped probe is used which is connected to a digital data acquisition system. The cone and adjoining sleeve section contain a series of strain gauges and other transducers which continuously monitor and record various soil parameters as the cone penetrates the soils.

The soil parameters measured depend on the type of cone being used, however they always include the following basic measurements

- Cone tip resistance q_c
- Sleeve friction f_s
- Inclination (from vertical) i
- Depth below ground z

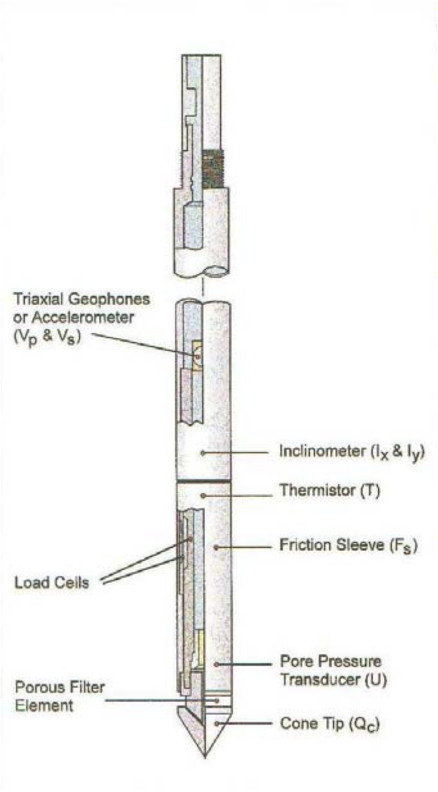


Figure 1: Cone Diagram

The inclinometer in the cone enables the verticality of the test to be confirmed and, if required, the vertical depth can be corrected.

The cone is thrust into the ground at a steady rate of about 20 mm/sec, usually using the hydraulic rams of a purpose built CPT rig, or a drilling rig. The testing is carried out in accordance with the Australian Standard AS1289 Test 6.5.1.

Figure 2: Purpose built CPT rig

The CPT can penetrate most soil types and is particularly suited to alluvial soils, being able to detect fine layering and strength variations. With sufficient thrust the cone can often penetrate a short distance into weathered rock. The cone will usually reach refusal in coarse filling, medium to coarse gravel and on very low strength or better rock. Tests have been successfully completed to more than 60 m.

Types of CPTs

Douglas Partners (and its subsidiary GroundTest) owns and operates the following types of CPT cones:

Type	Measures
Standard	Basic parameters (q_c, f_s, i & z)
Piezocone	Dynamic pore pressure (u) plus basic parameters. Dissipation tests estimate consolidation parameters
Conductivity	Bulk soil electrical conductivity (σ) plus basic parameters
Seismic	Shear wave velocity (V_s), compression wave velocity (V_p), plus basic parameters

Strata Interpretation

The CPT parameters can be used to infer the Soil Behaviour Type (SBT), based on normalised values of cone resistance (Qt) and friction ratio (Fr). These are used in conjunction with soil classification charts, such as the one below (after Robertson 1990)

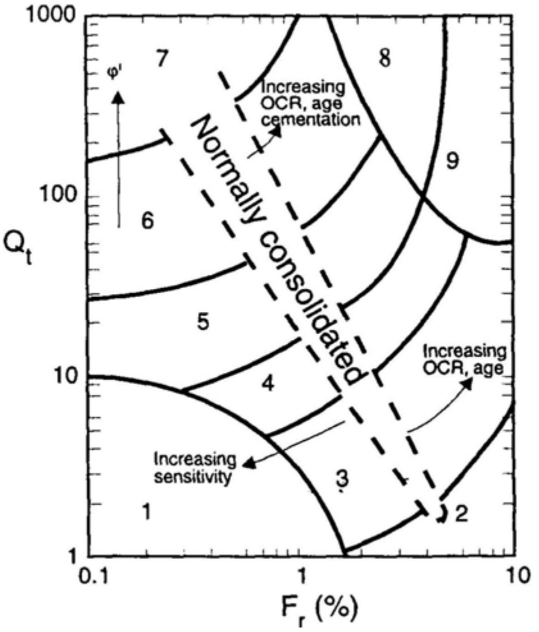


Figure 3: Soil Classification Chart

DP's in-house CPT software provides computer aided interpretation of soil strata, generating soil descriptions and strengths for each layer. The software can also produce plots of estimated soil parameters, including modulus, friction angle, relative density, shear strength and over consolidation ratio.

DP's CPT software helps our engineers quickly evaluate the critical soil layers and then focus on developing practical solutions for the client's project.

Engineering Applications

There are many uses for CPT data. The main applications are briefly introduced below:

Settlement

CPT provides a continuous profile of soil type and strength, providing an excellent basis for settlement analysis. Soil compressibility can be estimated from cone derived moduli, or known consolidation parameters for the critical layers (eg. from laboratory testing). Further, if pore pressure dissipation tests are undertaken using a piezocone, in-situ consolidation coefficients can be estimated to aid analysis.

Pile Capacity

The cone is, in effect, a small scale pile and, therefore, ideal for direct estimation of pile capacity. DP's in-house program ConePile can analyse most pile types and produces pile capacity versus depth plots. The analysis methods are based on proven static theory and empirical studies, taking account of scale effects, pile materials and method of installation. The results are expressed in limit state format, consistent with the Piling Code AS2159.

Dynamic or Earthquake Analysis

CPT and, in particular, Seismic CPT are suitable for dynamic foundation studies and earthquake response analyses, by profiling the low strain shear modulus G_0. Techniques have also been developed relating CPT results to the risk of soil liquefaction.

Other Applications

Other applications of CPT include ground improvement monitoring (testing before and after works), salinity and contaminant plume mapping (conductivity cone), preloading studies and verification of strength gain.

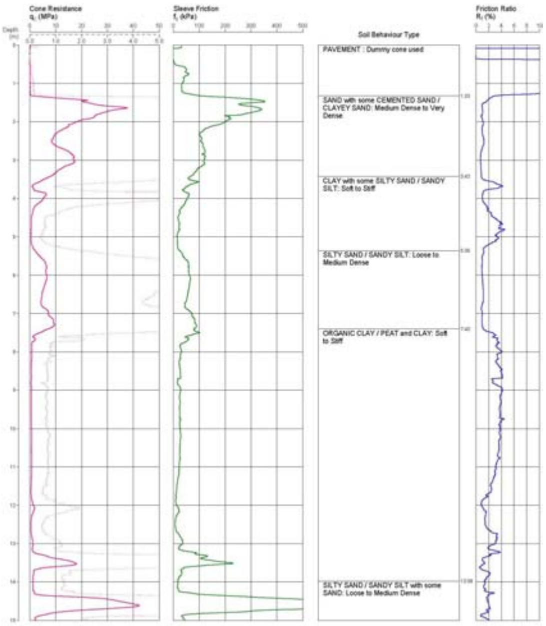


Figure 4: Sample Cone Plot

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 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx 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 $Is_{(50)}$

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 loner 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

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.

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, 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.

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Billard Leece Partnership Pty Ltd
OFFICE: Macarthur
DRAWN BY: IKA
SCALE: As shown
DATE: 6.03.2017

TITLE: **Approximate Test Pit Locations**
Proposed Picton High School Development
480 Argyle Street, Picton, NSW

N
MGA

PROJECT No: 34252.02
DRAWING No: 1
REVISION: 0

Appendix B

Test Pit Logs (Pits 1 – 10)

TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 218.5 mAHD
EASTING: 279530
NORTHING: 6213581

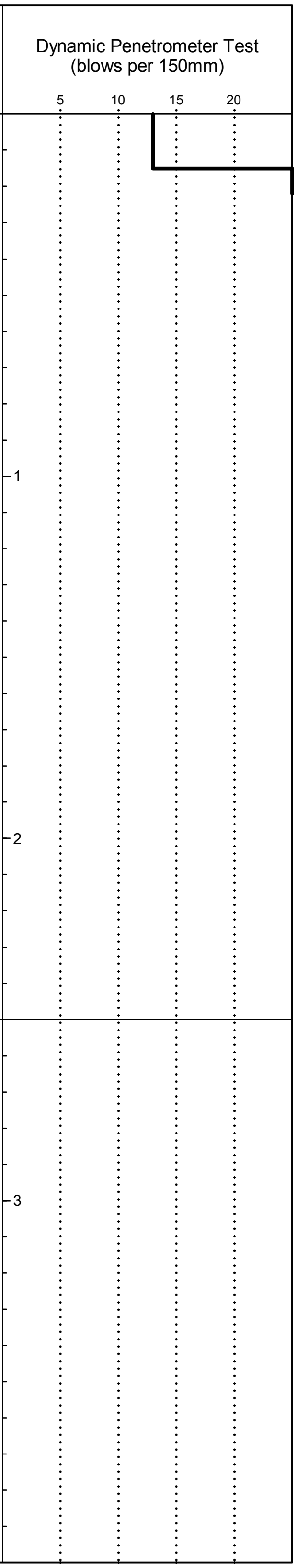
PIT No: 1
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log
	0.0	FILLING - brown silty clay with some organics (topsoil)	☒
	0.2	FILLING - brown and yellow gravel (ripped sandstone)	☒
218	0.4	FILLING - brown silty clay with some gravel and rootlets, MC~PL	☒
	1.0	CLAY - red brown silty clay, MC~PL	▨
217			
	2.3	SHALE - low to medium strength, highly weathered, grey and brown shale	
216	2.5	Pit discontinued at 2.5m - refusal on low to medium strength shale	
215			

Sampling & In Situ Testing

Type	Depth	Sample	Results & Comments
D/B	0.0		PID<1
	0.2		
D	0.5		PID<1
B	0.5		
	0.7		
D	1.0		PID<1
D	1.5		PID<1
D	2.0		PID<1
D	2.5		PID<1

Water



RIG: Takeuchi TB145 excavator - 300mm bucket

LOGGED: NJG

SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

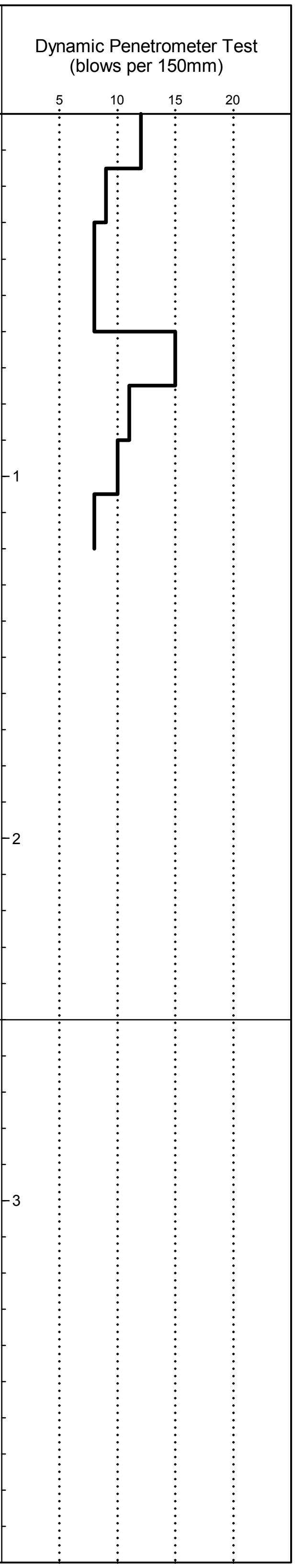
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 215.3 mAHD
EASTING: 279571
NORTHING: 6213636

PIT No: 2
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water
				Type	Depth	Sample	Results & Comments	
		FILLING - brown silty clay with some organics (topsoil)		D	0.0		PID<1	
	0.2	FILLING - brown silty clay, MC<PL			0.2			
215								
				D	0.5		PID<1	
	0.9	CLAY - very stiff, brown and grey silty clay, MC~PL						
	1			D	1.0		PID<1	
214								
				D	1.5		PID<1	
	2			D	2.0		PID<1	
213	2.3	SANDSTONE - low to medium strength, highly weathered, brown and grey sandstone						
	2.5	Pit discontinued at 2.5m - refusal on low to medium strength sandstone		D	2.5		PID<1	
	3							
212								



RIG: Takeuchi TB145 excavator - 300mm bucket

LOGGED: NJG

SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

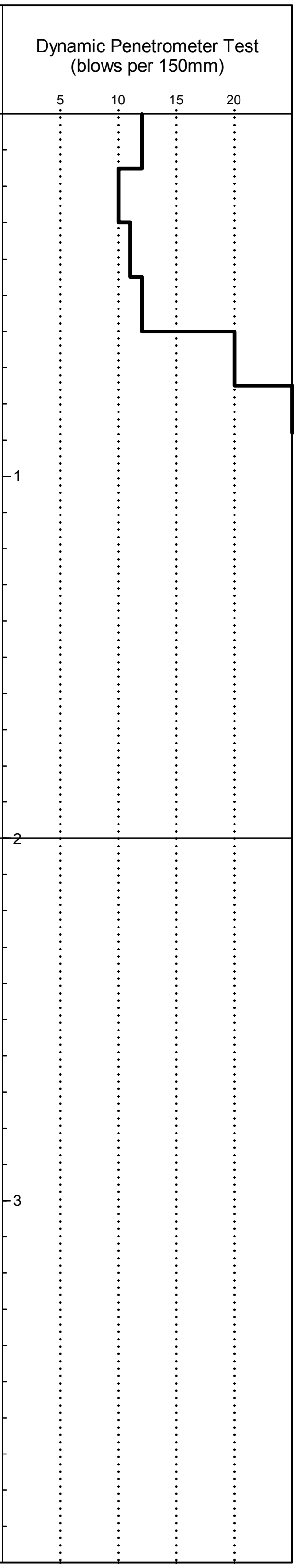
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 218.1 mAHD
EASTING: 279509
NORTHING: 6213640

PIT No: 3
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
218		FILLING - brown silty clay with some organics (topsoil)		D	0.0 0.2			
	0.2	FILLING - brown silty clay, MC~PL		D	0.5			
	0.6	CLAY - hard, brown gravelly clay, MC~PL		B	0.6 0.8			
217	1	- becoming red brown and grey, silty below 1.0m		D	1.0			
				D	1.5			
	1.8	SILTSTONE - low to medium strength, grey and brown siltstone						
216	2 2.0	Pit discontinued at 2.0m - refusal on low to medium strength siltstone		D	2.0			
215	3							



RIG: Takeuchi TB145 excavator - 300mm bucket

LOGGED: NJG

SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

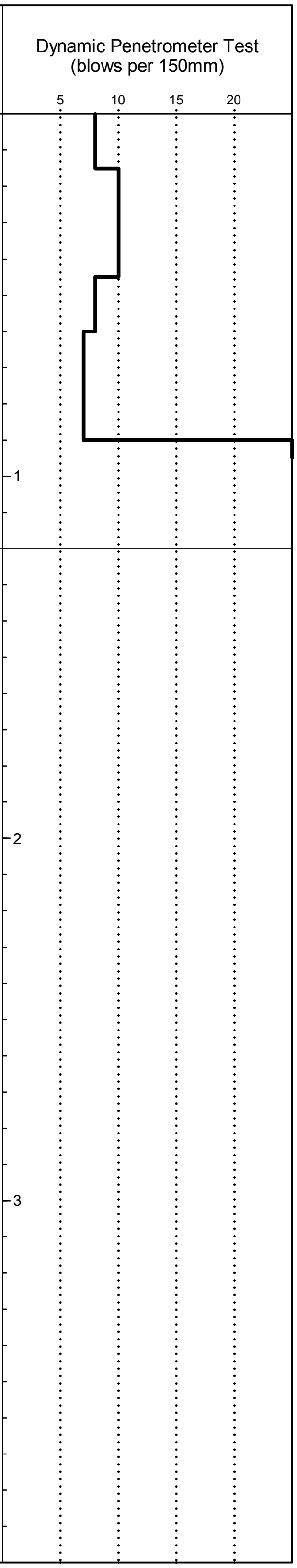
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 216.2 mAHD
EASTING: 279464
NORTHING: 6213733

PIT No: 4
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILLING - brown silty clay with some organics (topsoil)		D	0.0		PID<1	
216	0.2	FILLING - brown silty clay with some gravel, MC<PL		U_{50}	0.2			
	0.5	CLAY - red brown silty clay, MC~PL		D	0.5		PID<1	
					0.6			
	0.9	SHALE - low to medium strength, highly weathered, grey and brown shale		D	1.0		PID<1	
215	1.2	Pit discontinued at 1.2m - refusal on low to medium strength shale						
214	2							
213	3							



RIG: Takeuchi TB145 excavator - 300mm bucket **LOGGED:** NJG **SURVEY DATUM:** MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

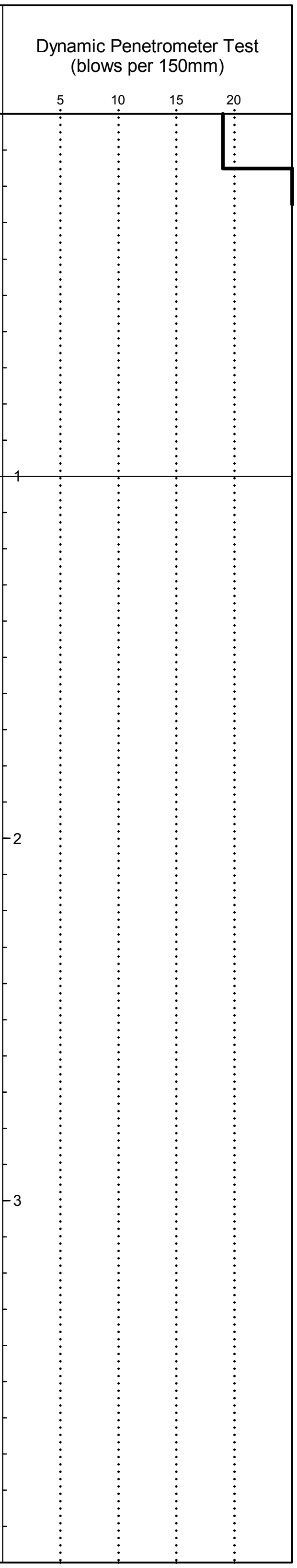
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 212.9 mAHD
EASTING: 279556
NORTHING: 6213772

PIT No: 5
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILLING - brown and grey clayey gravel		D*	0.0 0.2		PID<1	
	0.4	CLAY - very stiff, red brown silty clay, MC~PL		D B	0.5 0.7		PID<1	
212	0.9	SHALE - low to medium strength, highly weathered, brown and grey shale		D	1.0		PID<1	
	1.0	Pit discontinued at 1.0m - refusal on low to medium strength shale						
211	2							
210	3							
209								



RIG: Takeuchi TB145 excavator - 300mm bucket
LOGGED: NJG
SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS: * Replicate sample BD2/230117 collected

☐ 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

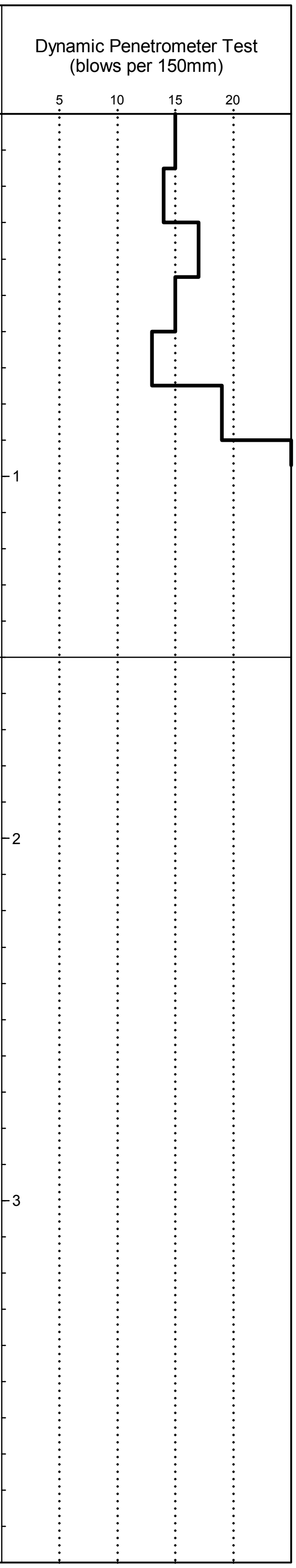
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 214.1 mAHD
EASTING: 279653
NORTHING: 6213715

PIT No: 6
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
214		FILLING - brown and grey gravelly clay, MC<PL	▦	D	0.0		PID<1	
					0.2			
				D	0.5		PID<1	
				U_{50}				
	0.9	SANDSTONE - very low to low strength, highly weathered, orange brown sandstone	░		0.9			
	1			D	1.0		PID<1	
213								
		- becoming low to medium strength below 1.4m						
	1.5	Pit discontinued at 1.5m - refusal on low to medium strength sandstone		D	1.5		PID<1	
	2							
212								
	3							
211								



RIG: Takeuchi TB145 excavator - 300mm bucket **LOGGED:** NJG **SURVEY DATUM:** MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

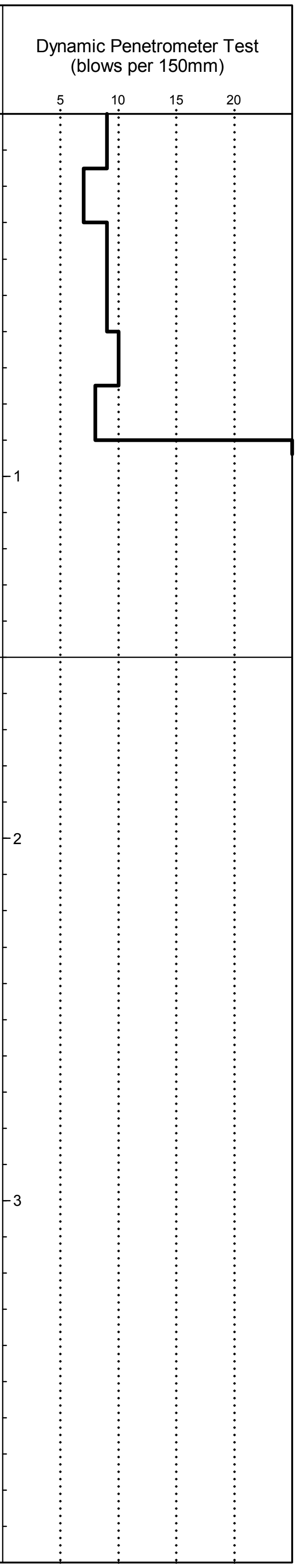
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 215.5 mAHD
EASTING: 279685
NORTHING: 6213771

PIT No: 7
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILLING - brown silty clay with some organics (topsoil)	☒	D	0.0 0.2		PID<1	
215	0.2	FILLING - brown silty clay, MC<PL	☒	D B	0.5 0.7		PID<1	
	0.9	SANDSTONE - very low to low strength, highly weathered, orange brown sandstone - becoming low to medium strength below 1.4m	⠿	D D	1.0 1.5		PID<1 PID<1	
214	1.5	Pit discontinued at 1.5m - refusal on low to medium strength sandstone						
	2							
213								
	3							
212								



RIG: Takeuchi TB145 excavator - 300mm bucket
LOGGED: NJG
SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

TEST PIT LOG

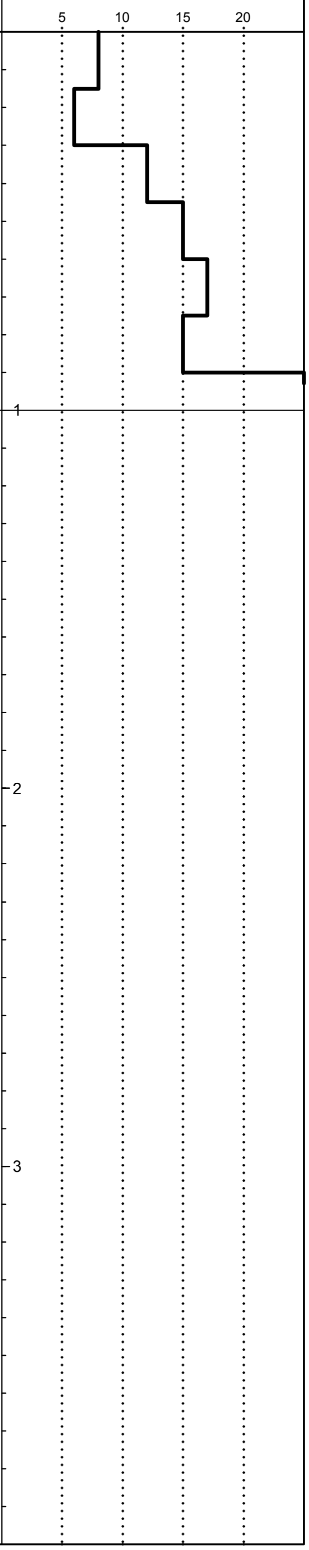
CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 216.3 mAHD
EASTING: 279739
NORTHING: 6213747

PIT No: 8
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		TOPSOIL - brown silty clay with some organics	D*	0.0 0.2		PID<1	
216	0.3	CLAY - hard, brown gravelly clay, trace weathered shale, MC~PL	U$_{50}$ D	0.3 0.5 0.7		PID<1	
	0.9	SHALE - low to medium strength, highly weathered, grey and brown shale	D	1.0		PID<1	
	1.0	Pit discontinued at 1.0m - refusal on low to medium strength shale					
215	2						
214	3						
213							

Dynamic Penetrometer Test
(blows per 150mm)



RIG: Takeuchi TB145 excavator - 300mm bucket
LOGGED: NJG
SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS: * Replicate sample BD3/230117 collected

☐ 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

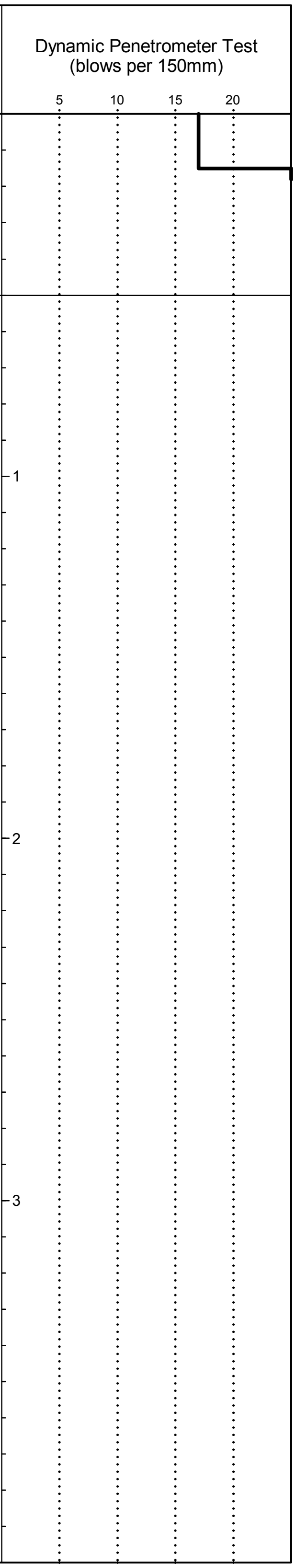
TEST PIT LOG

CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 216.9 mAHD
EASTING: 279759
NORTHING: 6213689

PIT No: 9
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		TOPSOIL - brown silty clay with some organics		D	0.0 – 0.2		PID<1	
	0.1	SHALE - very low to low strength, highly weathered, grey and brown shale		B	0.3 – 0.5			
	0.5	Pit discontinued at 0.5m - limit of investigation		D	0.5		PID<1	
216	1			D	1.0		PID<1	
215	2							
214	3							
213								



RIG: Takeuchi TB145 excavator - 300mm bucket
LOGGED: NJG
SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

TEST PIT LOG

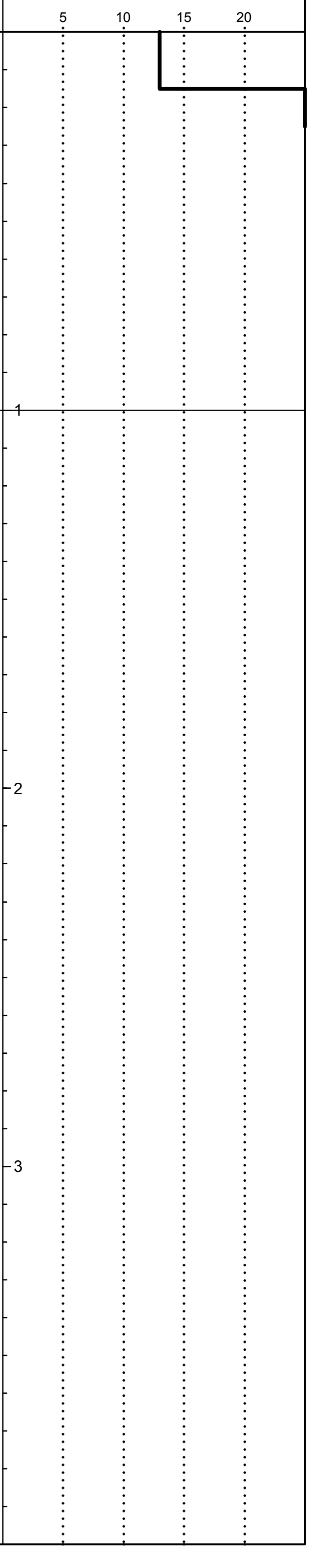
CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

SURFACE LEVEL: 216.0 mAHD
EASTING: 279678
NORTHING: 6213620

PIT No: 10
PROJECT No: 34252.02
DATE: 23/1/2017
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
216		TOPSOIL - brown silty clay with some organics		D	0.0 0.2		PID<1	
	0.2	SHALE - extremely low to very low strength, extremely to highly weathered, grey and brown shale		D	0.5		PID<1	
		- becoming low to medium strength below 0.9m						
215	1 1.0	Pit discontinued at 1.0m - limit of investigation		D	1.0		PID<1	
214	2							
213	3							

Dynamic Penetrometer Test (blows per 150mm)



RIG: Takeuchi TB145 excavator - 300mm bucket
LOGGED: NJG
SURVEY DATUM: MGA94 Zone 56

WATER OBSERVATIONS: No free groundwater observed

REMARKS:

☐ 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

Laboratory Test Results

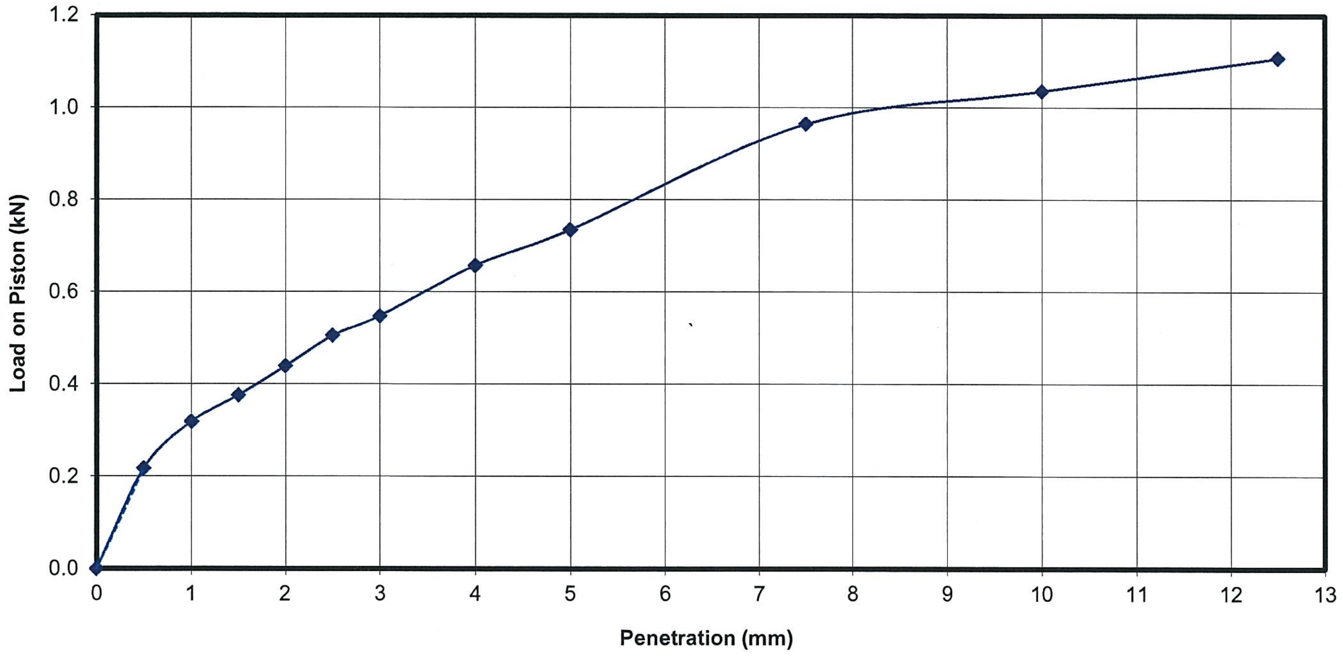
Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of California Bearing Ratio Test

Client : Billard Leece Partnership Pty Ltd
Project : Proposed Picton High School Redevelopment
Location : Argyle Street, Picton
Test Location : TP1
Depth / Layer : 0.5 - 0.7m

Project No. : 34252.02
Report No. : MA17-364
Report Date : 15/02/2017
Date Sampled : 23/01/2017
Date of Test: 9/02/2017
Page: 1 of 1



Description: FILLING - Brown silty clay with some gravel

Sampling Method(s): Sampled By DP Engineering
Test Method(s): AS 1289.6.1.1, AS 1289.2.1.1

Remarks:

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 99% of STD MDD
MOISTURE RATIO: 99% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.7%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		22.6	1.49
After soaking		25.4	1.48
After test	Top 30mm of sample	25.1	-
	Remainder of sample	25.6	-
Field values		17.6	-
Standard Compaction	(OMC/MDD)	22.8	1.49

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5mm**	**4.0**

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory No 828
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested:	LL
Checked:	AJS

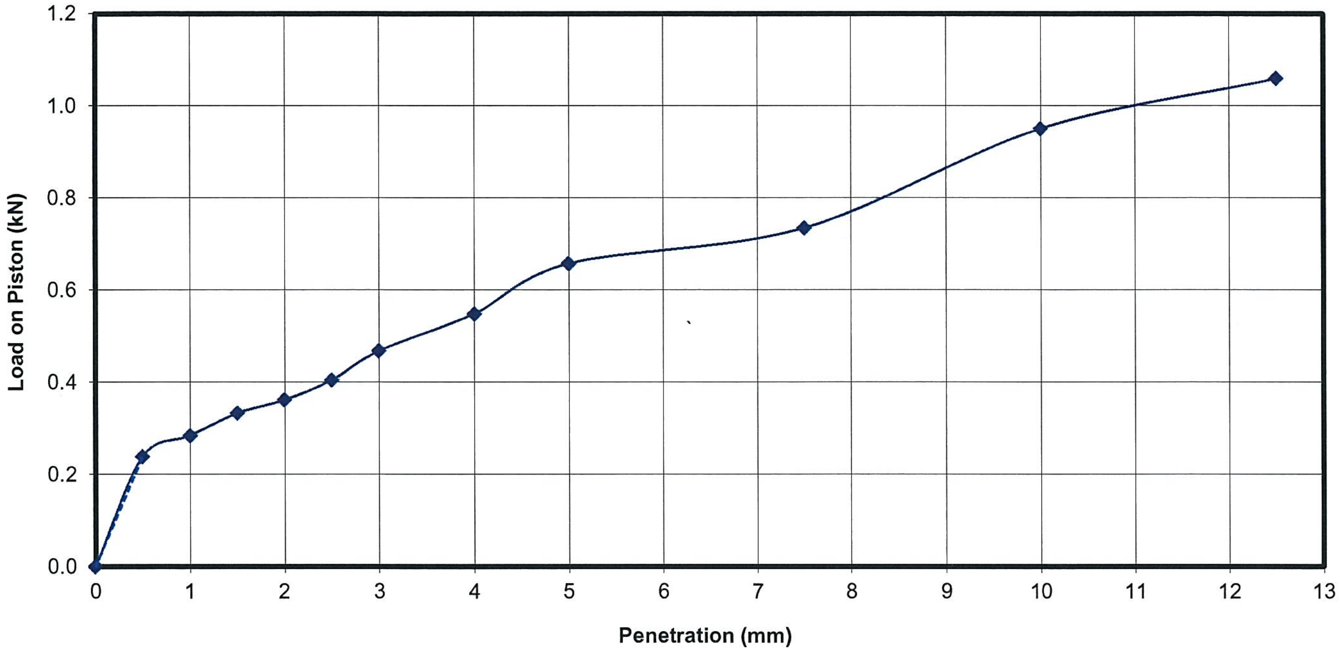
A.J. Sweetland
Laboratory Manager

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of California Bearing Ratio Test

Client : Billard Leece Partnership Pty Ltd
Project No. : 34252.02
Report No. : MA17- 035
Project : Proposed Picton High School Redevelopment
Report Date : 15/02/2017
Date Sampled : 23/01/2017
Location : Argyle Street, Picton
Date of Test: 9/02/2017
Test Location : TP2
Depth / Layer : 0.5 - 0.7m
Page: 1 of 1



Description: FILLING - Brown silty clay

Sampling Method(s): Sampled By DP Engineering
Test Method(s): AS 1289.6.1.1, AS 1289.2.1.1

Remarks:

Percentage > 19mm: 0.0%
LEVEL OF COMPACTION: 99% of STD MDD
SURCHARGE: 4.5 kg
SWELL: 0.2%
MOISTURE RATIO: 100% of STD OMC
SOAKING PERIOD: 4 days

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		25.4	1.55
After soaking		26.5	1.54
After test	Top 30mm of sample	26.3	-
	Remainder of sample	28.2	-
Field values		18.5	-
Standard Compaction	(OMC/MDD)	25.5	1.55

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**5.0mm**	**3.5**

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory No 828
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested: LL
Checked: AJS

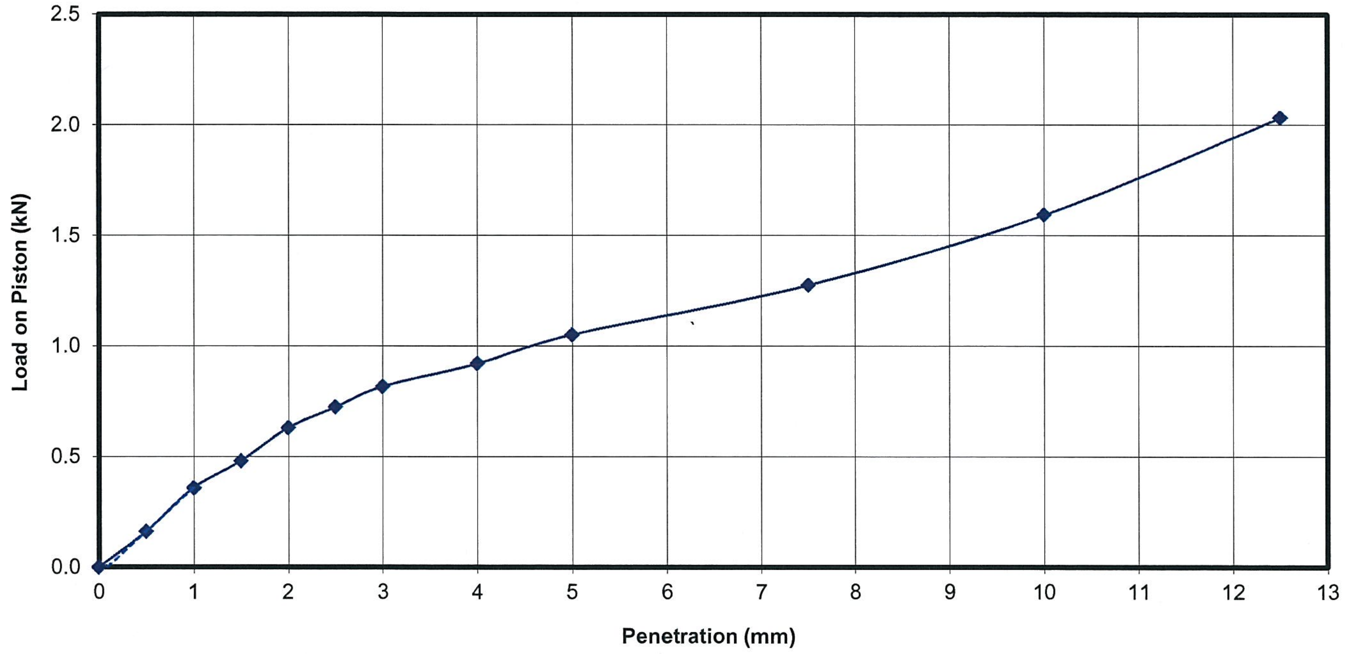
A.J. Sweetland
Laboratory Manager

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of California Bearing Ratio Test

Client :	Billard Leece Partnership Pty Ltd	**Project No. :**	34252.02
		Report No. :	MA17-036
Project :	Proposed Picton High School Redevelopment	**Report Date :**	15/02/2017
		Date Sampled :	23/0120/17
Location :	Argyle Street, Picton	**Date of Test:**	9/02/2017
Test Location :	TP3		
Depth / Layer :	0.5 - 0.7m	**Page:**	1 of 1



Description: CLAY - Brown gravelly clay

Sampling Method(s): Sampled By DP Engineering
Test Method(s): AS 1289.6.1.1, AS 1289.2.1.1

Remarks:

Percentage > 19mm: 0.0%
LEVEL OF COMPACTION: 99% of STD MDD
SURCHARGE: 4.5 kg
SWELL: 0.2%
MOISTURE RATIO: 100% of STD OMC
SOAKING PERIOD: 4 days

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		21.2	1.58
After soaking		22.9	1.58
After test	Top 30mm of sample	21.7	-
	Remainder of sample	22.6	-
Field values		15.0	-
Standard Compaction	(OMC/MDD)	21.2	1.60

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5mm**	**6**

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory No 828
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested: LL
Checked: AJS

A.J. Sweetland
Laboratory Manager

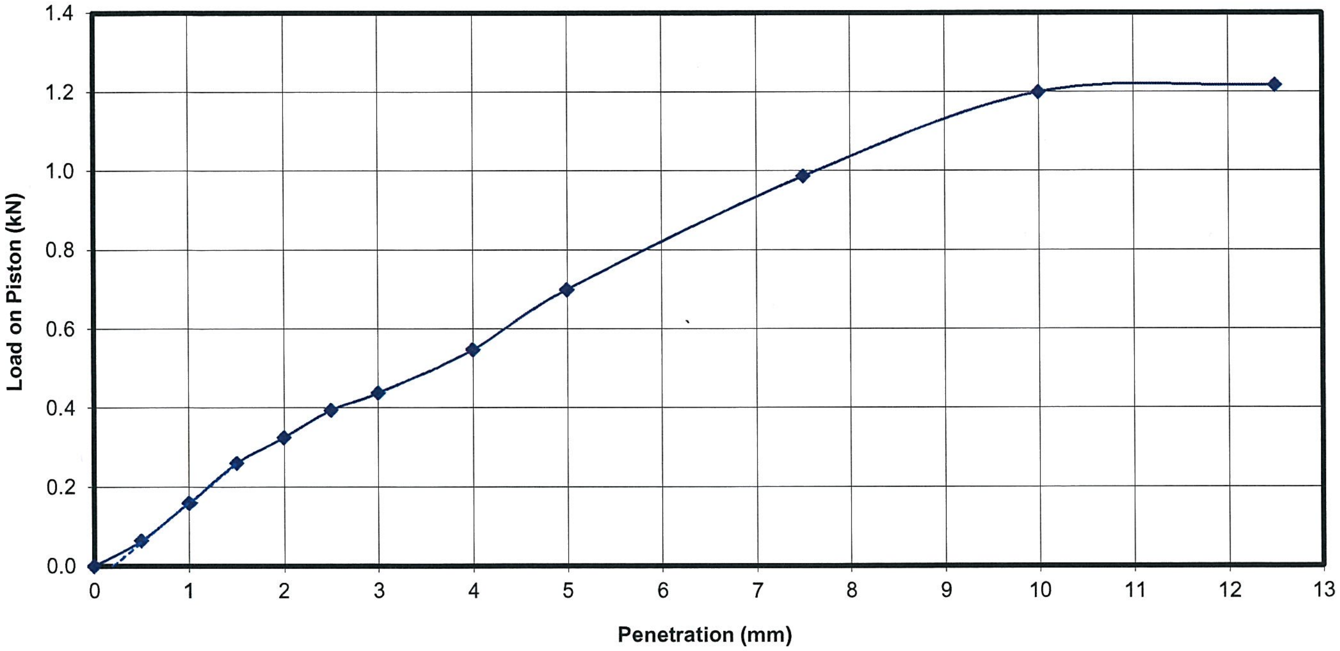
Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of California Bearing Ratio Test

Client : Billard Leece Partnership Pty Ltd
Project : Proposed Picton High School Redevelopment
Location : Argyle Street, Picton
Test Location : TP7
Depth / Layer : 0.3 - 0.5m

Project No. : 34252.02
Report No. : MA17-037
Report Date : 15/02/2017
Date Sampled : 23/01/2017
Date of Test: 9/02/2017
Page: 1 of 1



Description: FILLING - Brown silty clay

Sampling Method(s): Sampled By DP Engineering
Test Method(s): AS 1289.6.1.1, AS 1289.2.1.1

Remarks:

Percentage > 19mm: 0.0%
LEVEL OF COMPACTION: 101% of STD MDD
MOISTURE RATIO: 99% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.7%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		13.9	1.86
After soaking		16.3	1.85
After test	Top 30mm of sample	17.0	-
	Remainder of sample	16.1	-
Field values		9.3	-
Standard Compaction	(OMC/MDD)	14.0	1.85

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**5.0mm**	**3.5**

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory No 828
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested:	LL
Checked:	AJS

A.J. Sweetland
Laboratory Manager

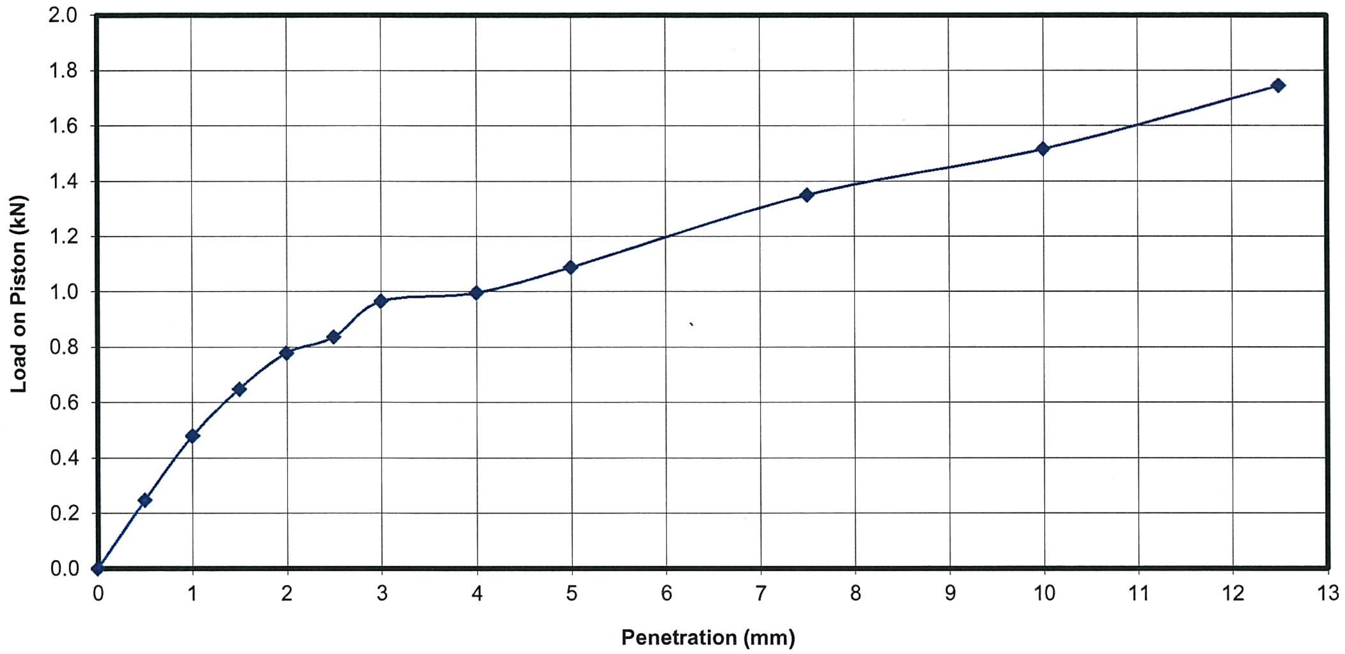
Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of California Bearing Ratio Test

Client : Billard Leece Partnership Pty Ltd
Project : Proposed Picton High School Redevelopment
Location : Argyle Street, Picton
Test Location : TP9
Depth / Layer : 0.5 - 0.7m

Project No. : 34252.02
Report No. : MA17-038
Report Date : 15/02/2017
Date Sampled : 23/0120/17
Date of Test: 9/02/2017
Page: 1 of 1



Description: SHALE - Grey and brown shale

Sampling Method(s): Sampled By DP Engineering
Test Method(s): AS 1289.6.1.1, AS 1289.2.1.1

Remarks: (Excluded)

Percentage > 19mm: 7.0%
LEVEL OF COMPACTION: 101% of STD MDD
SURCHARGE: 4.5 kg
SWELL: 1.1%
MOISTURE RATIO: 100% of STD OMC
SOAKING PERIOD: 4 days

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		15.2	1.79
After soaking		18.0	1.77
After test	Top 30mm of sample	19.8	-
	Remainder of sample	17.3	-
Field values		9.8	-
Standard Compaction	(OMC/MDD)	15.2	1.77

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5mm**	**6**

NATA Accredited Laboratory No 828
The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested:	LL
Checked:	AJS

A.J. Sweetland
Laboratory Manager

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
18 Waler Crescent
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

Results of Moisture Content, Plasticity and Linear Shrinkage Tests

Client: **Billard Leece Partnership Pty Ltd**

Project: Proposed Picton High School Redevelopment

Location: Argyle Street, Picton

Project No: 34252.02
Report No: MA17-098
Report Date: 2/02/2017

Date Sampled: 23/01/2017
Date of Test: 23/01/2017
Page: 1 of 1

Test Location	Depth (m)	Description	Code	W_F %	W_L %	W_P %	PI %	*LS %
TP2	2.0	SILTY CLAY – Brown & grey silty clay	2,5	15.3	38	17	21	8.0
TP3	1.5	SILTY CLAY – Red brown & grey silty clay	2,5	21.4	62	26	36	11.5
TP4	0.2 – 0.6	FILLING – Brown silty clay	2,5	17.7	43	24	19	8.5
TP7	0.5 – 0.7	FILLING – Brown silty clay	2,5	6.3	27	20	7	5.0
TP8	0.5 – 0.6	GRAVELLY CLAY – Brown gravelly clay	2,5	11.8	65	25	40	15.0
TP10	0.5	SHALE – Grey brown shale	2,5	12.5	55	22	33	11.0

Legend:

W_F	Field Moisture Content
W_L	Liquid limit
W_P	Plastic limit
PI	Plasticity index
LS	Linear shrinkage from liquid limit condition

Test Methods:

Moisture Content:	AS 1289 2.1.1
Liquid Limit:	AS 1289 3.1.2
Plastic Limit:	AS 1289 3.2.1
Plasticity Index:	AS 1289 3.3.1
Linear Shrinkage:	AS 1289 3.4.1

Code:

Sample history for plasticity tests

1. Air dried
2. Low temperature (<50ºC) oven dried
3. Oven (105ºC) dried
4. Unknown

Method of preparation for plasticity tests

5. Dry sieved
6. Wet sieved
7. Natural

*Specify if sample crumbled CR or curled CU

Sampling Methods: Sampled By DP Engineering

Remarks:

NATA Accredited Laboratory Number: 828

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards. Accredited for compliance with ISO/IEC 17025

ACCREDITED FOR TECHNICAL COMPETENCE

Tested: JP
Checked: AJS

A J Sweetland
Laboratory Manager