Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Preliminary Geotechnical Investigation

Proposed Picton High School Redevelopment
Argyle Street, Picton, NSW

Prepared for
Billard Leece Partnership Pty Ltd

Project 34252.02
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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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

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

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, NSW. 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 50 mm 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 (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 equipment's 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 construction.

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 Pty Ltd (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

Introduction

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

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

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

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

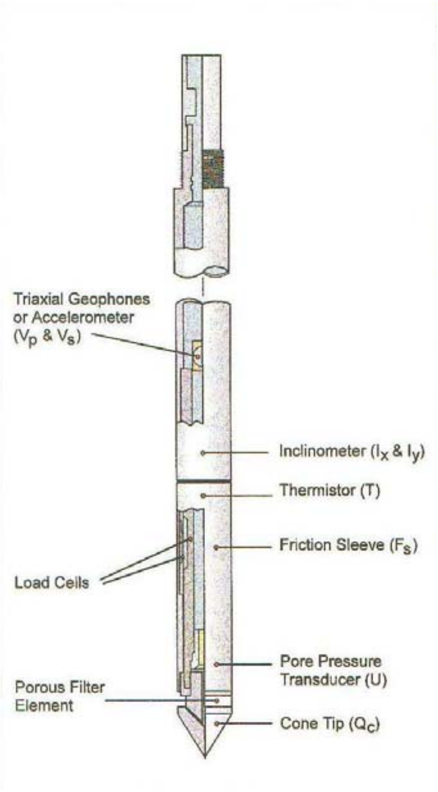

Figure 1: Cone Diagram

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)

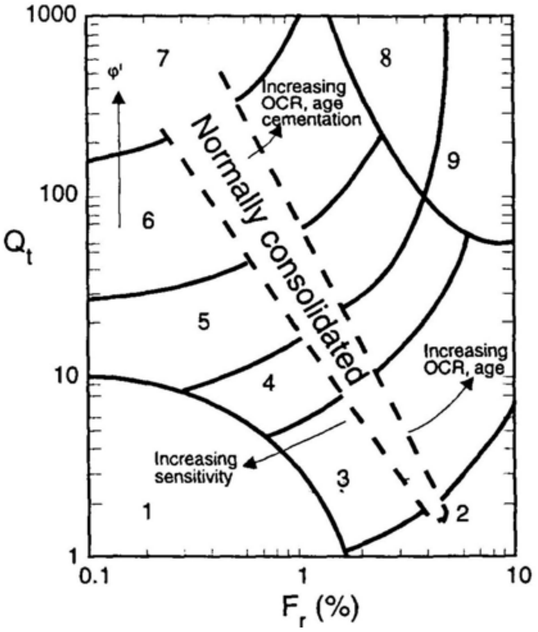

Figure 3: Soil Classification Chart

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.

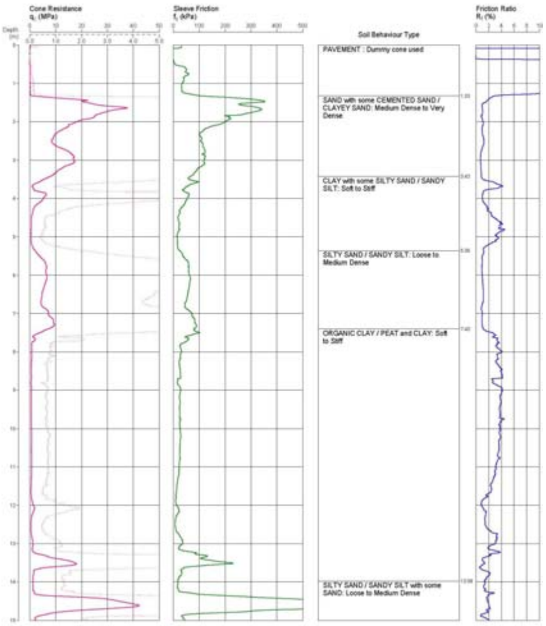

Figure 4: Sample Cone Plot

Rock 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

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.

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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: 1

Appendix B

Test Pit Logs (Pits 1 – 10)

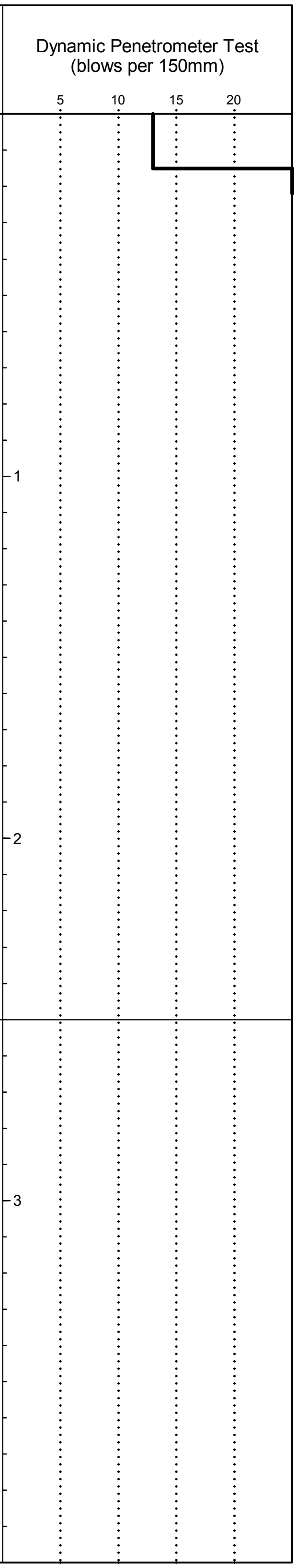
TEST PIT LOG

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	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
	0.0	FILLING - brown silty clay with some organics (topsoil)	D/B	0.0		PID<1	
				0.2			
	0.2	FILLING - brown and yellow gravel (ripped sandstone)					
	0.4	FILLING - brown silty clay with some gravel and rootlets, MC~PL	D	0.5		PID<1	
218			B				
				0.7			
	1.0	CLAY - red brown silty clay, MC~PL	D	1.0		PID<1	
217			D	1.5		PID<1	
			D	2.0		PID<1	
	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	D	2.5		PID<1	
215							



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)

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TEST PIT LOG

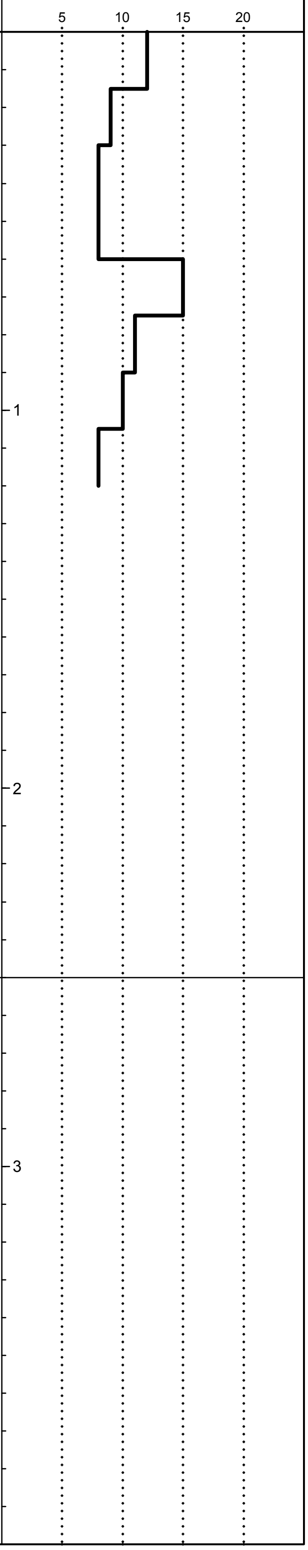
CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

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	Type	Depth	Sample	Results & Comments	Water
		FILLING - brown silty clay with some organics (topsoil)	D	0.0		PID<1	
				0.2			
	0.2	FILLING - brown silty clay, MC<PL					
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							

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)

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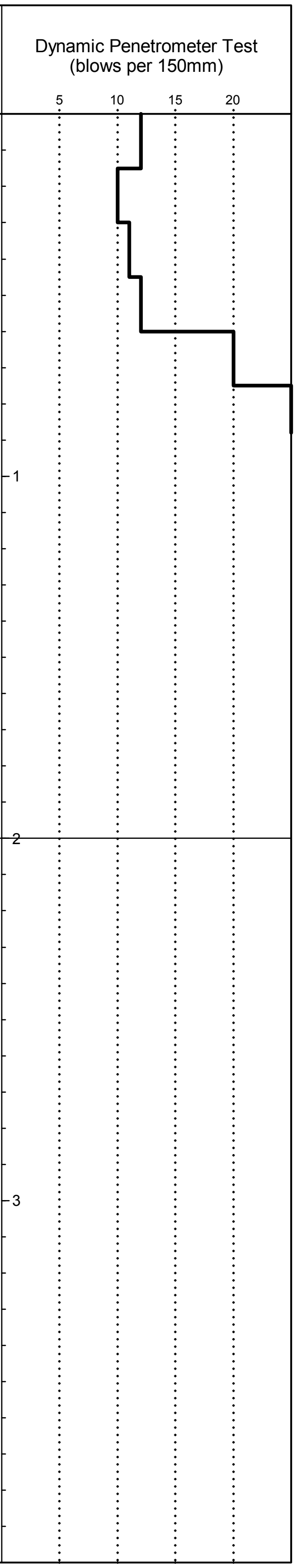
TEST PIT LOG

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	Sampling & In Situ Testing: Depth	Sampling & In Situ Testing: Sample	Sampling & In Situ Testing: 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			
	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)

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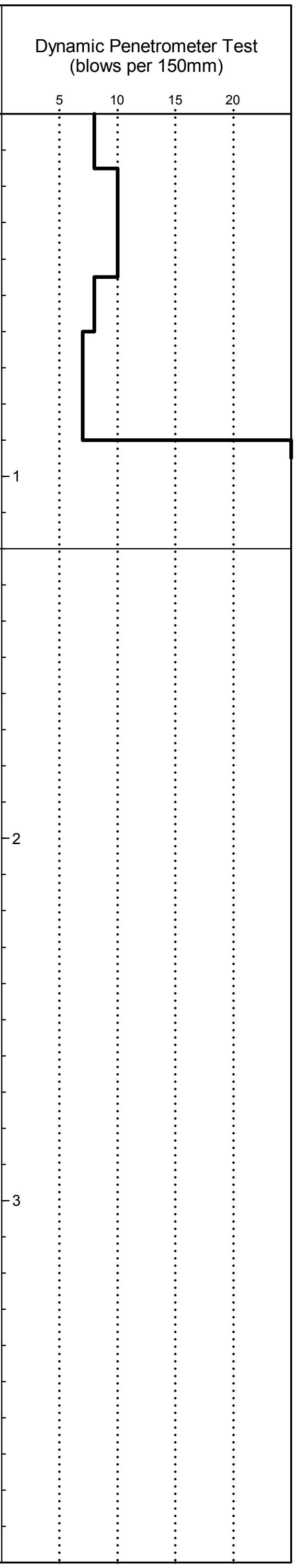
TEST PIT LOG

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			0.2			
				U_{50}				
	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						
	1			D	1.0		PID<1	
215	1.2	Pit discontinued at 1.2m - refusal on low to medium strength shale						
	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)

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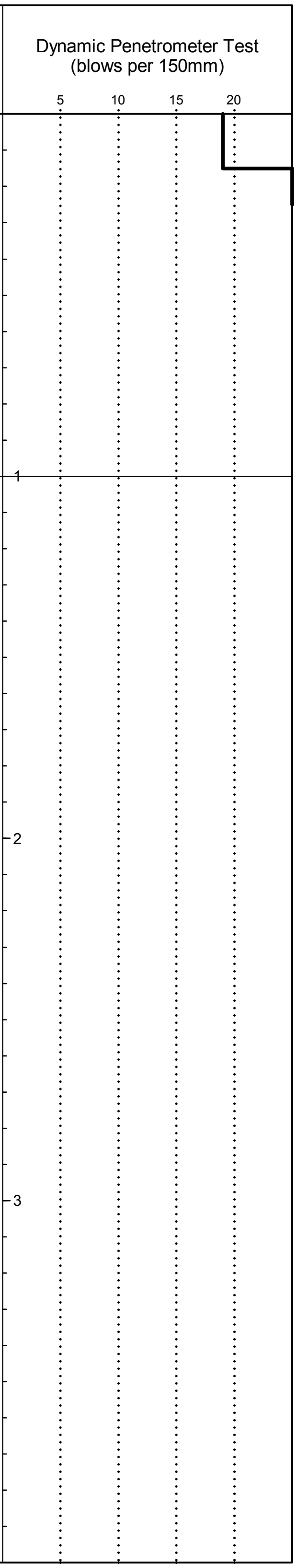
TEST PIT LOG

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	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)

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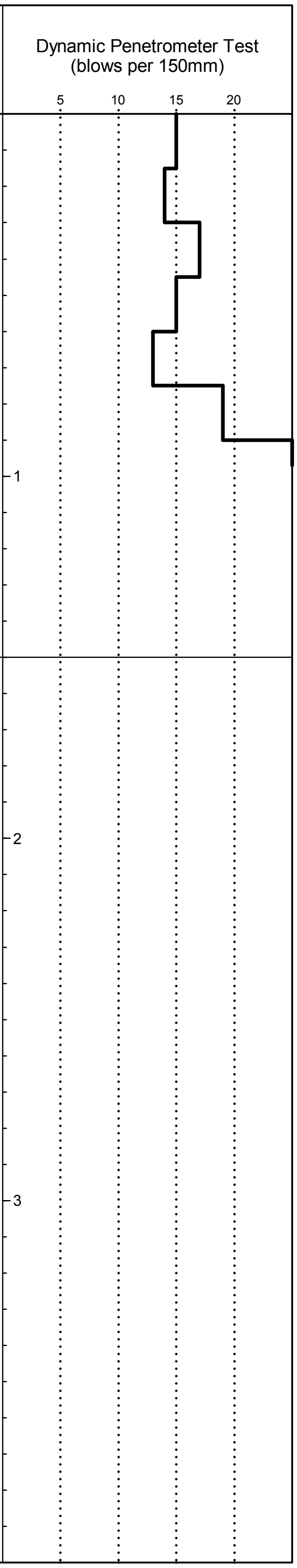
TEST PIT LOG

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	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
214		FILLING - brown and grey gravelly clay, MC<PL	D	0.0 0.2		PID<1	
			D	0.5		PID<1	
			U_{50}				
	0.9	SANDSTONE - very low to low strength, highly weathered, orange brown sandstone		0.9			
213	1		D	1.0		PID<1	
		- 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	
212	2						
211	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)

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Geotechnics / Environment / Groundwater

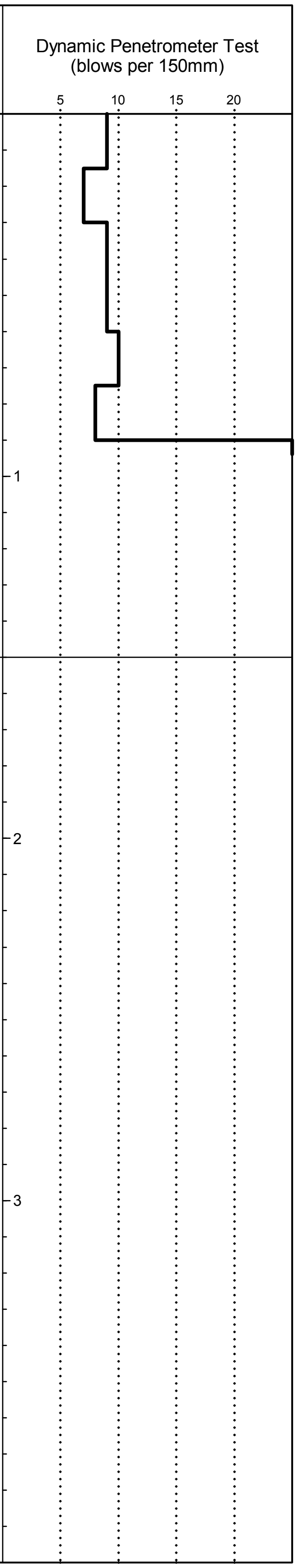
TEST PIT LOG

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
215		FILLING - brown silty clay with some organics (topsoil)		D	0.0 0.2		PID<1	
	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	1.0		PID<1	
214	1.5	Pit discontinued at 1.5m - refusal on low to medium strength sandstone		D	1.5		PID<1	
213	2							
212	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)

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TEST PIT LOG

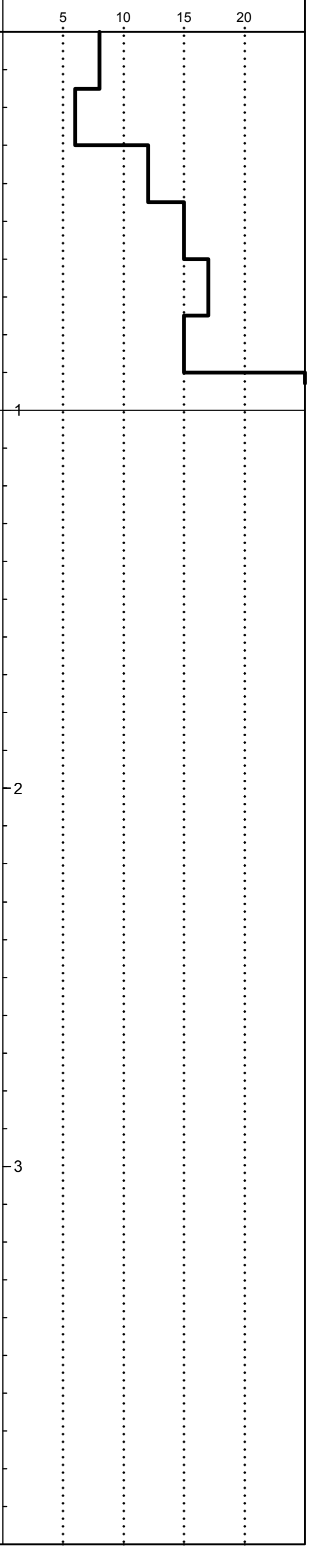
CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

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	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	
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						
	1 1.0	Pit discontinued at 1.0m - refusal on low to medium strength shale		D	1.0		PID<1	
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)

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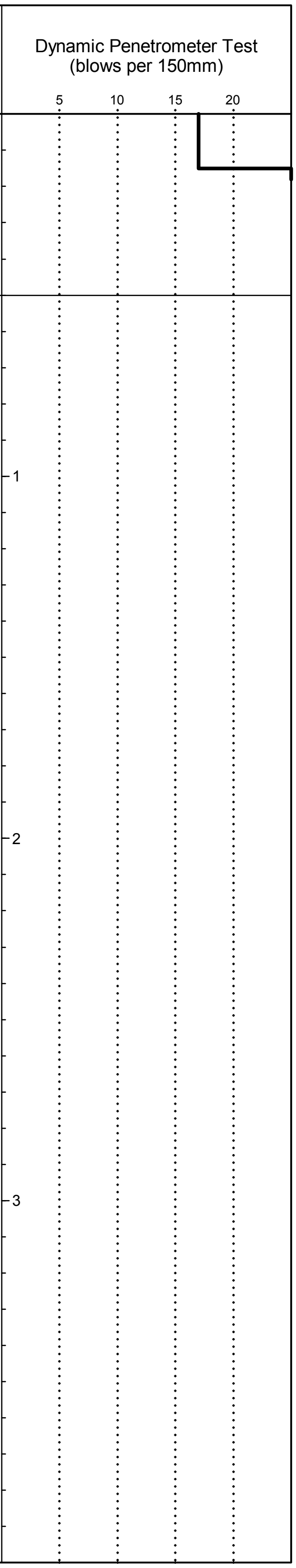
TEST PIT LOG

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)

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TEST PIT LOG

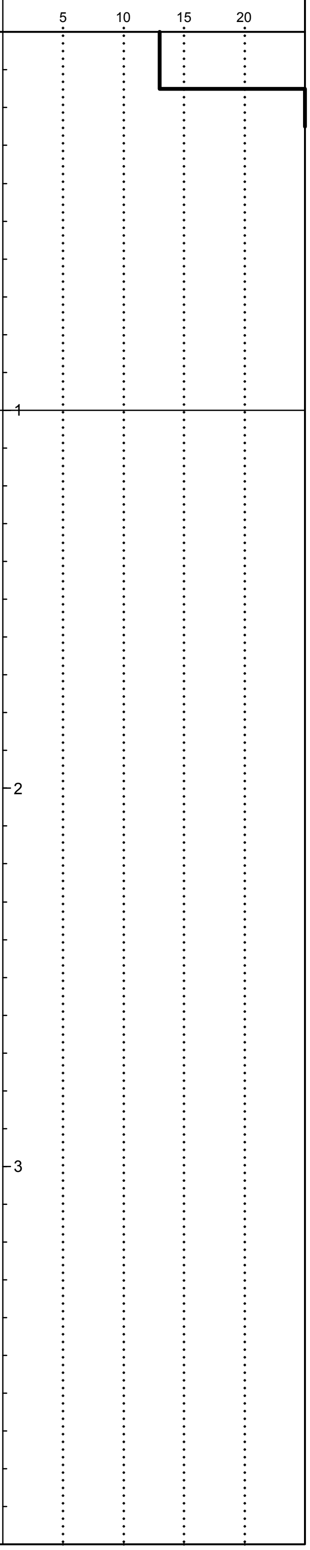
CLIENT: Billard Leece Partnership Pty Ltd
PROJECT: Proposed Picton High School Redevelopment
LOCATION: Argyle Street, Picton, NSW

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		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)

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Appendix C

Laboratory Test Results

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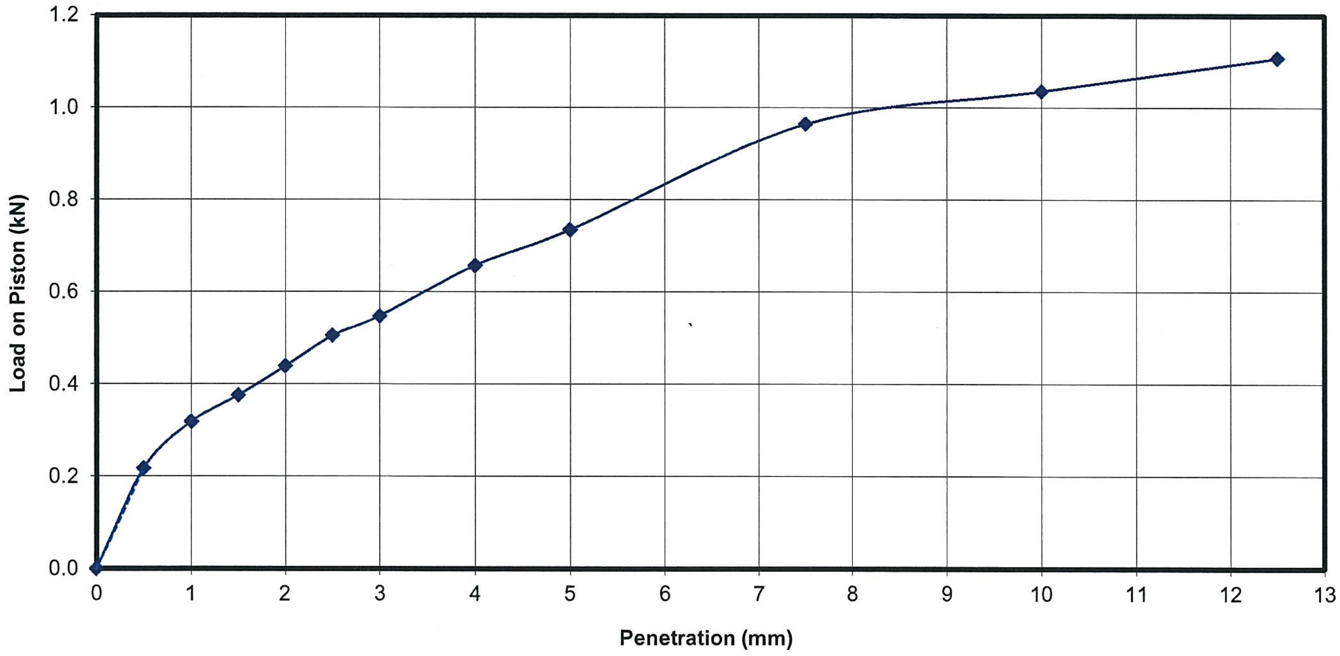
Smeaton Grange NSW 2567
Phone (02) 4647 0075
Fax (02) 4646 1886

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**

Tested: LL
Checked: AJS

A.J. Sweetland
Laboratory Manager

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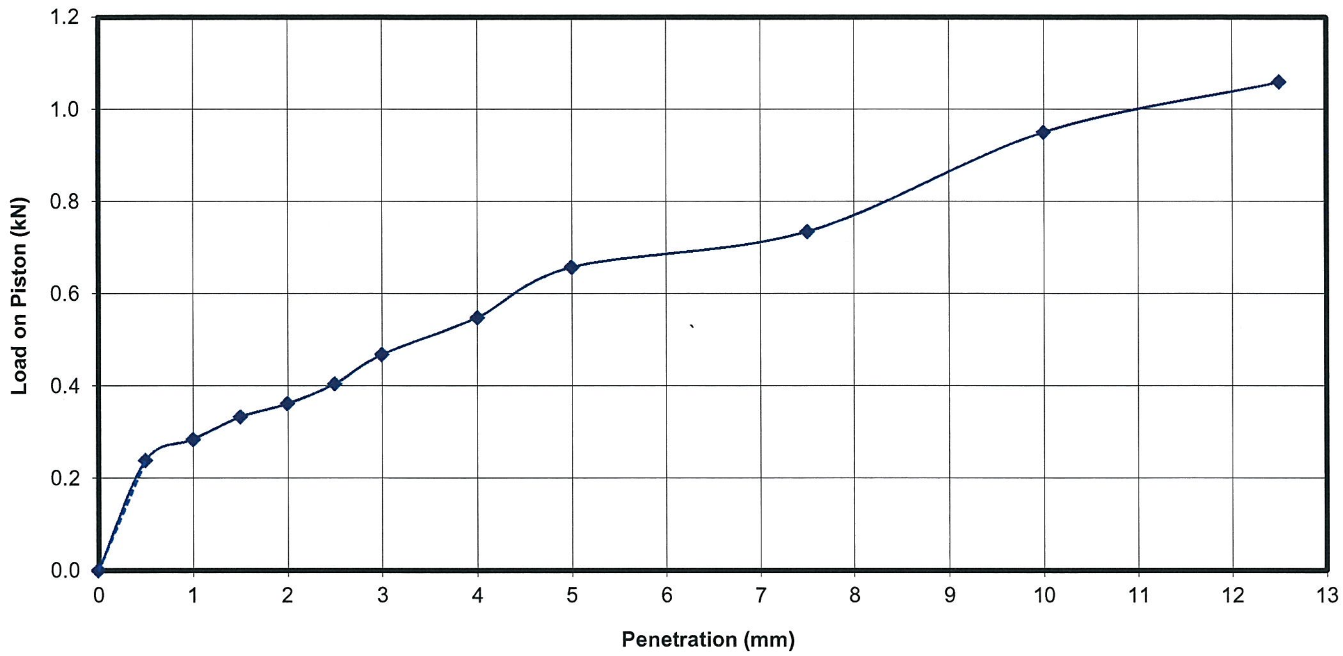
Results of California Bearing Ratio Test

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**

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Tested: LL
Checked: AJS

A.J. Sweetland
Laboratory Manager

Douglas Partners
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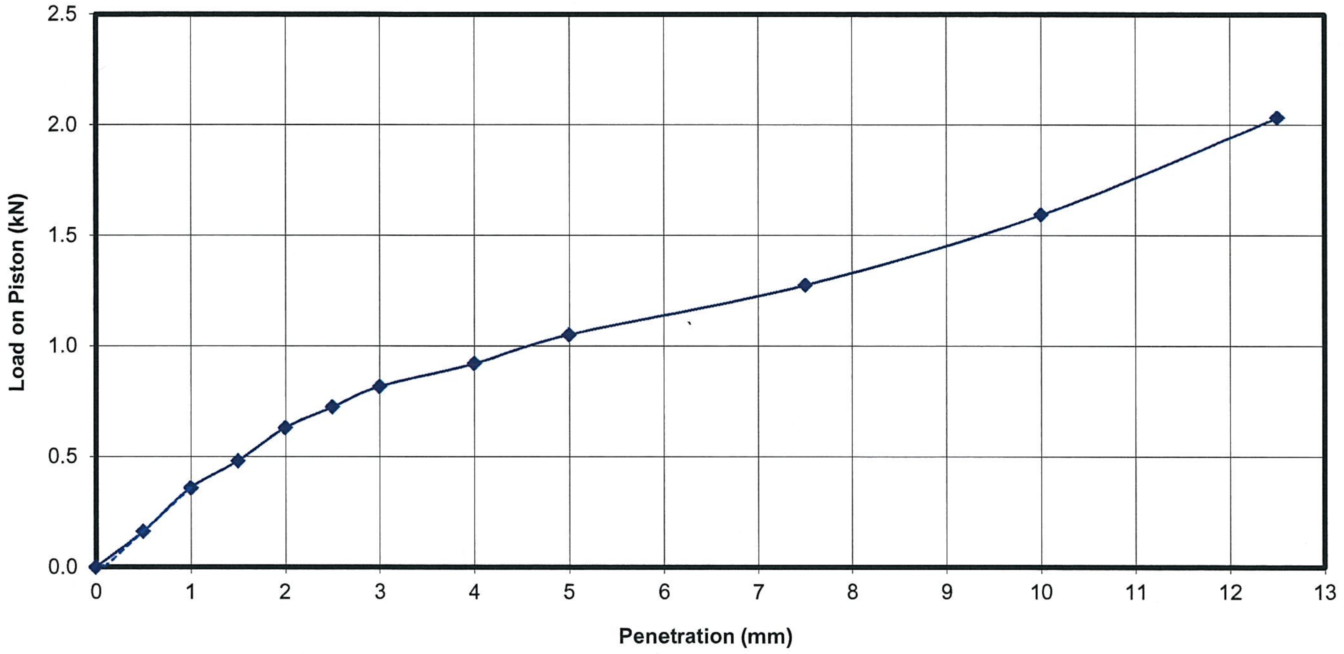
Results of California Bearing Ratio Test

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

Tested: LL
Checked: AJS

A.J. Sweetland
Laboratory Manager

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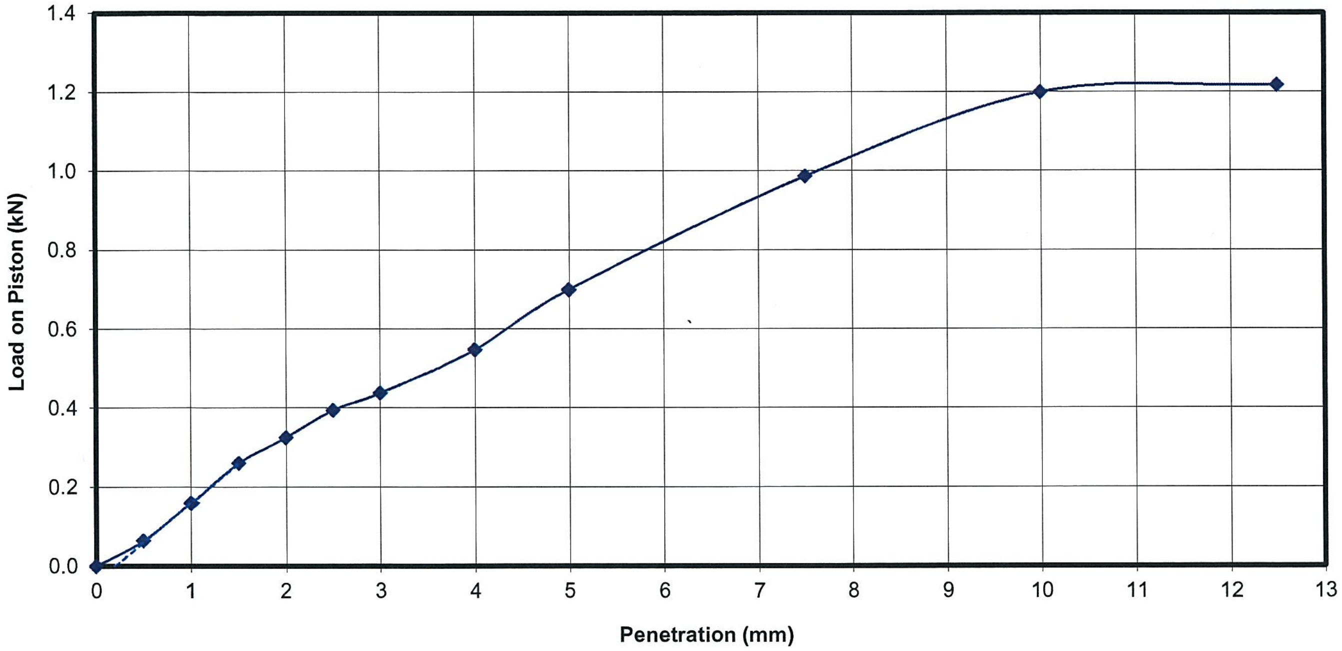
Results of California Bearing Ratio Test

Client :	Billard Leece Partnership Pty Ltd	**Project No. :**	34252.02
		Report No. :	MA17-037
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 :	TP7		
Depth / Layer :	0.3 - 0.5m	**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**

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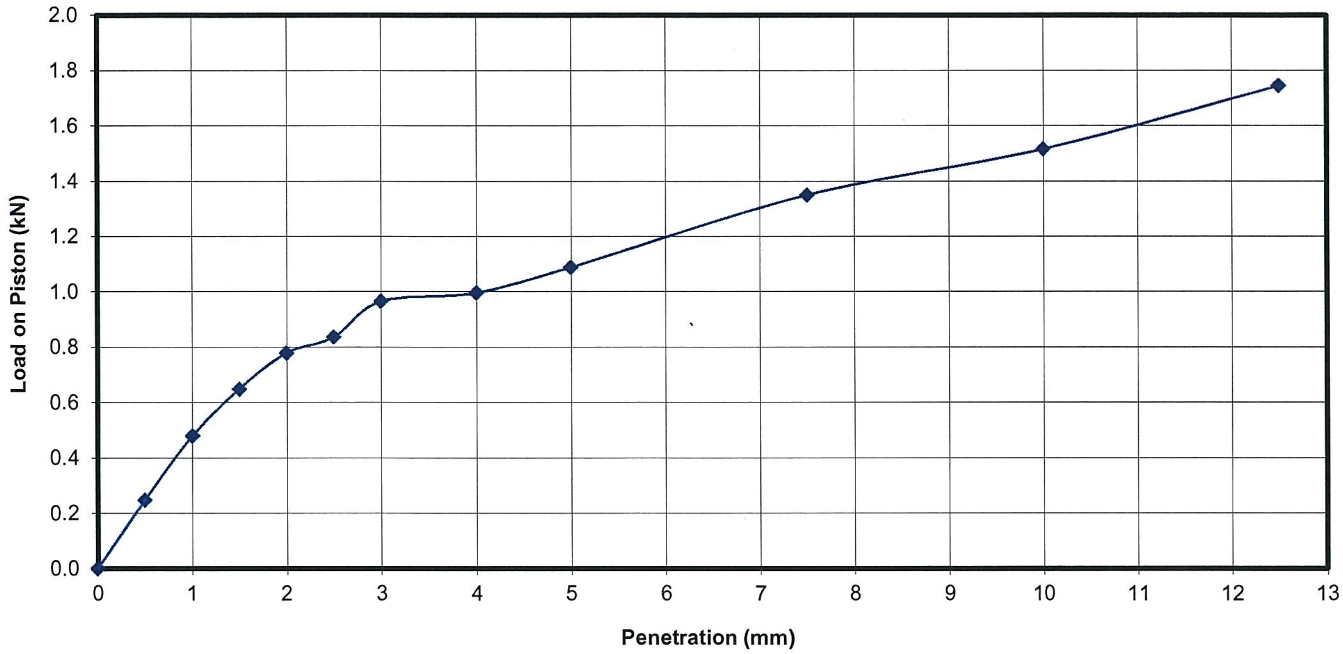
Results of California Bearing Ratio Test

Client :	Billard Leece Partnership Pty Ltd	**Project No. :**	34252.02
		Report No. :	MA17-038
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 :	TP9		
Depth / Layer :	0.5 - 0.7m	**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**

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Tested:	LL
Checked:	AJS

A.J. Sweetland
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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 (<50°C) oven dried
3. Oven (105°C) 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:

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Tested: JP
Checked: AJS

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Laboratory Manager