Douglas Partners
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Integrated Practical Solutions

Report on Geotechnical Investigation

Westmead Car Park
Institute Road
Westmead Hospital

Prepared for
Health Infrastructure

Project 73603.00
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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		15 January 2014
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NCS INTERNATIONAL CERTIFIED QUALITY MANAGEMENT SYSTEM

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

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Report on Geotechnical Investigation
Westmead Car Park
Institute Road, Westmead Hospital

1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for a proposed multi-storey car park at Institute Road, Westmead Hospital. The investigation was commissioned in a letter dated dated 9 August 2013 by Ms Amanda Bock of Health Infrastructure and was undertaken in accordance with DP's revised proposal dated 5 August 2013. Following the issuance of a draft report on 30 September 2013, comments were received from the client which are addressed in this report.

It is understood that the construction of a new multi-level car park superstructure is being considered at the location of the existing car park known as 'Car Park 7'. It is further understood that the proposed car park will have a short term capacity of 800+ spaces and the capacity to be increased to 1600 spaces in the future. The structural engineer advised of revised likely (ultimate) limit state compressive column loads between 3000 kN and 6600 kN and an (ultimate) limit state tension load of 1500 kN for one of the nominated pile categories.

The purpose of the geotechnical investigation was to assess and comment upon:

- the soil and bedrock profile in the vicinity of the proposed works;
- the existing pavement and subgrade profile within the works area;
- site preparation and earthworks;
- excavation conditions and excavation support;
- foundation types, levels and allowable bearing pressures;
- pavement design parameters; and
- other geotechnical constraints of the site, including soil aggressivity with respect to buried structural elements.

The investigation included the drilling of ten boreholes, the excavation of eight test pits and laboratory testing of selected samples. The details of the field work and laboratory testing are presented in this report, together with comments and recommendations on the issues listed above.

A contamination assessment was carried out concurrently with the geotechnical investigation and is reported separately (refer. report for DP Project No. 73603.01 dated 27 November 2013). The client has indicated that in circumstances where the subgrade is found to be contaminated, they would seek to minimise the excavation and off-site removal of subgrade materials.

This report should be read in conjunction with the notes 'About this Report' contained in Appendix A.

2. Site Description

The site is triangular in shape with a plan area of about 1.4 hectares. The site falls about 3 m in the north westerly direction. It is bounded on two sides by public and internal roads to the south west and north west, respectively, and by an existing multi-storey hospital building to the south east. The site is generally paved with asphaltic concrete and was in use as a car park. An existing, single storey building occupies the north east corner of the site. The existing car park and building were fringed by lawn and garden beds containing some native trees. The ground surface on which the lawns and garden beds were located had a hummocky appearance, suggesting the presence of filling.

A steeper batter of about 20° rose about 2 m from the south eastern site boundary towards the adjacent hospital building. Within the otherwise gently sloping site topography, this batter was suggestive of a fill platform between the site and the adjacent hospital building.

3. Regional Geology

The Geological Survey of NSW 1:100,000 Geological Series Sheet 9030 (Penrith) indicates that the site stradles the boundary of Ashfield Shale and the underlying Hawkesbury Sandstone. Ashfield Shale typically comprises black to dark grey shales and laminites. Hawkesbury Sandstone is generally a medium to coarse grained quartz sandstone with minor shale and laminite layers.

The corresponding Soil Conservation Service of NSW 1:100,000 Soil Landscape Series Sheet indicates that the bedrock is overlain by residual clay soils.

Field work for this investigation confirmed the presence of Ashfield Shale and the residual clay soils overlying the bedrock.

4. Field Work Methods

Field work was carried out between 28 August and 7 September 2013 and comprised:

- electromagnetic scanning for buried services at the borehole and test pit locations;
- exploratory boreholes, which entailed:
  - drilling of 10 boreholes to depths between 8 m and 8.5 m with a truck-mounted drill rig, using spiral flight augers through soil and NMLC diamond coring through rock;
  - Standard penetration tests (SPTs) through the soil profile in each borehole at regular depth intervals;
  - discrete disturbed and undisturbed sampling of soil throughout the soil profile;
  - retrieval of continuous rock core samples;
  - installation of standpipe wells in three boreholes;
- test pits, which entailed:
  - excavation of eight test pits to depths between 0.3 m and 3.1 m with a small, rubber-tracked excavator, using a bucket;

    - continuous in-situ strength testing of the soil profile using a dynamic cone penetrometer (DCP) at each test pit location to a maximum depth of 2.4 m or prior refusal;
    - discrete disturbed and bulk sampling of soil throughout the soil profile; and
- a survey of all borehole and test pit locations using differential GPS.

The locations of the boreholes and test pits are shown on Drawing 1 in Appendix B.

5. Field Work Results

The results of the field work, in the form of borehole and test pit logs, are given in Appendix C, together with notes defining classification methods and descriptive terms and photographs of the retrieved rock core samples.

The boreholes encountered the ground conditions summarised in Table 1.

Table 1: Summary of Ground Profile

Depth to Top of Unit (m)	Description
0	**Filling** – an existing flexible pavement comprising 30 – 50 mm thick asphaltic concrete (AC) overlying a 160 – 510 mm thick gravelly sand roadbase in the existing carpark areas. In unpaved parts of the site and for the south eastern half of the pavement subgrade, uncontrolled, but apparently well compacted, sandy, silty and gravelly clay filling with some building rubble, including masonry, concrete, wood and fibro cement fragments, was present down to depths between 0.5 m and 1.9 m. The filling in the southern corner of the site also contained a distinct layer of sand filling between 0.2 m and 1.1 m thick.
0.4 – 1.9	**Silty Clay** – stiff grading to hard in consistency with depth, with an increasing frequency of ironstone gravel bands present with depth, recovered as gravelly clay. Residual Soil.
1.0 – 3.8	**Shaly Clay** – extremely low strength, extremely weathered shale.
2.2 – 5.2	**Shale** – extremely low to very low strength, extremely weathered shale grading to very low to low strength, highly or moderately weathered and fragmented or fractured. Medium strength, slightly weathered and slightly fractured shale was encountered at depth, grading to high strength.

Groundwater was measured in the standpipe wells installed in boreholes BH2, BH4 and BH10. The groundwater levels provided in Table 2 were measured on 6 September 2013 during a return visit to the site.

Table 2: Measured Groundwater Levels (6/09/13)

Borehole	Depth (m)	Reduced Level (m AHD)
2	4.4	13.4
4	3.5	16.2
10	3.9	13.1

Comparison with the borehole records indicates that the phreatic surface intersected the extremely low strength, extremely weathered shaly clay or shale of the upper, weathered bedrock profile at the date of measurement.

6. Laboratory Testing

NATA-registered laboratories were used to carry out the following laboratory tests on soil and rock core samples obtained during the field work:

- 17 moisture content tests to assess the current moisture of the subgrade;
- four Atterberg limits tests for soil classification and reactivity assessment;
- four, 4-day soaked California bearing ratio (CBR) tests for pavement design;
- three conductivity, pH, sulphate and chloride ion tests for soil aggressivity assessment;
- 44 point load strength index tests of selected rock core samples; and

Except for the results of the point load strength index ($Is_{(50)}$) tests, the results of laboratory testing, reported in Appendix D, are summarised in Table 3. The results of the $Is_{(50)}$ tests of rock core samples are shown on the borehole logs contained in Appendix C, at the relevant depths.

Table 3: Summary of Laboratory Test Results

Borehole / Test Pit	Depth (m)	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	CBR (%)	pH	Electrical Conductivity (µS/cm)	Chloride (mg/kg)	Sulphate (mg/kg)
1	2.5	11.9	-	-	-	-	-	-	-	-
2	1.0 – 1.45	15.5	-	-	-	-	-	-	-	-
3	1.0	23.1	-	-	-	-	-	-	-	-
4	2.0	18.1	-	-	-	-	-	-	-	-
5	1.0	24.5	-	-	-	-	-	-	-	-
6	0.5	18.9	-	-	-	-	-	-	-	-
6	1.0	-	-	-	-	-	4.3	540	590	9
7	0.5	11.8	-	-	-	-	-	-	-	-
8	2.5	17.8	-	-	-	-	-	-	-	-
9	0.5	21.7	-	-	-	-	-	-	-	-
9	2.0	-	-	-	-	-	5.0	600	600	77
10	1.0	26.7	-	-	-	-	-	-	-	-
101	2.0	25.2	65	25	40	-	5.1	480	36	520
101	2.0 – 2.5	-	-	-	-	3.5	-	-	-	-
103	1.1	-	-	-	-	4	-	-	-	-
103	1.5	24.6	79	31	48	-	-	-	-	-
104	1.5	21.3	-	-	-	-	-	-	-	-
105	0.5 – 0.6	-	-	-	-	6	-	-	-	-
105	1.8	19.5	60	22	38	-	-	-	-	-
106	1.0	25.1	-	-	-	-	-	-	-	-
107	1.5	23.4					-	-	-	-
108	1.0 – 1.1	-	-	-	-	4	-	-	-	-
108	1.9 – 2.0	23.5	66	28	38		-	-	-	-

Atterberg limits test results indicate that the residual clay samples tested have high plasticity.

7. Geotechnical Model

The geotechnical model developed for the site, based on the borehole and test pit logs, is presented as inferred cross-sections A – A', B – B' and C – C', of Drawings 2, 3 and 4, respectively (refer to Appendix B). The cross-sections show the depths of soil overburden noted at the borehole and test pit locations together with interpreted geotechnical boundaries within the underlying rock. It should be noted that these interpreted boundaries are accurate at borehole locations only and that variations must be expected away from the boreholes. Thus, the strata or layers have been shown diagrammatically on the cross-sections by inferred strata boundaries only.

The bedrock profile is interpreted to be Ashfield Shale.

Classification of the rock encountered in the boreholes is based on the classification system of Pells et al (1998) in the paper entitled "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics, December 1998. The interpreted depth and RLs at the upper surface of the various bedrock classes is shown in Table 4.

Table 4: Summary of Geotechnical Model

Borehole	Surface RL at borehole (m AHD)	Depth (& RL)[1] to Top of Various Shale Classes[2] (m (& m AHD))			
		V	IV	III	II
1	18.08	2.1 (16.0)	-	6.35 (11.7)	7.5 (10.6)
2	17.75	1.0 (16.8)	3.6 (14.2)	5.1 (12.7)	7.8 (10.0)
3	19.88	1.3 (18.6)	4.4 (15.5)	2.4 (17.5)	6.7 (13.2)
4	19.71	3.4 (16.3)	-	4.5 (15.2)	-
5	18.66	3.5 (15.2)	-	-	6.0 (12.7)
6	19.21	2.5 (16.7)	3.8 (15.4)	6.4 (12.9)	7.9 (11.3)
7	18.05	2.3 (15.8)	4.3 (13.8)	-	6.3 (11.8)
8	18.98	3.8 (15.2)	4.3 (14.7)	-	6.7 (12.3)
9	18.20	2.8 (15.4)	4.2 (14.0)	-	7.6 (10.7)
10	17.00	3.8 (13.2)	-	-	5.2 (11.8)

Notes: 1 Numbers in parentheses are the Reduced Level (to AHD) for the top of the stratum.
2 Rock classification is based on References 1 and 2.

Rock contour drawings showing the interpreted top surface of Class III Shale and Class II Shale are presented as Drawings 5 and 6, respectively, in Appendix B. It should be noted that a 'pocket' of weaker, Class IV Shale was observed to be sandwiched between the upper Class III Shale and lower

Class II Shale at the location of borehole BH3 (refer. Drawing 4). It should be further noted that Class II Shale was not encountered in borehole BH4 down to the termination depth of 8.0 m.

8. Proposed Development

It is understood that the construction of a new multi-level car park superstructure is being considered at the location of the existing car park known as 'Car Park 7'. It is further understood that the proposed car park will have a short term capacity of 800+ spaces and the capacity to be increased to 1600 spaces in the future, for the parking of passenger vehicles and light commercial vehicles. With respect to the anticipated column loads, the revised ultimate limit state loads provided by the structural engineer are given in Table 5. By dividing the given ultimate limit state loads by a factor of 1.35 in accordance with the advice of the structural engineer, the approximate serviceability limit state column loads are also given in Table 5.

Table 5: Anticipated Column Loads

Bored Pile Mark	Ultimate Limit State		Serviceability Limit State	
	Compressive Load (kN)	Tensile Load (kN)	Compressive Load (kN)	Tensile Load (kN)
BP1	6600	-	4900	-
BP2	5400	-	4000	-
BP3	5400	1500	4000	1100
BP4	3000	-	2200	-

With reference to the feasibility study drawings provided by the structural engineer (Drawing Nos. NA80813269 SK3-Rev 2, SK4-Rev 2 and SK5-Rev 1 by Cardno dated 13 August 2013), it is understood that three options are under consideration to provide six to seven levels of car parking at the site:

- a ramped option, with a footprint limited to the south western half of the site, the longitudinal building axis parallel to Darcy Road, parking spaces aligned perpendicular to the Darcy Road alignment and no terracing of the ground floor level;
- a split level option; with an L-shaped footprint that does not occupy the western and north eastern apexes of the triangular-shaped site, the longitudinal building axis perpendicular to Darcy Road, parking spaces aligned parallel to Darcy Road and a terraced ground floor level; and
- a maximum site usage option, with a trapezium-shaped footprint that occupies all but the north eastern apex of the triangular-shaped site, the parking spaces are generally aligned parallel to Darcy Road except for a row along the Institute Road side of the building and a terraced ground floor level rising to the south east.

The existing coroner's court building and lawn at the north eastern apex of the site do not form part of the footprint for any of the proposed car park options and will therefore remain unchanged.

The client has indicated that in circumstances where the existing subgrade is found to be contaminated (refer to separate contamination assessment report by DP), they would seek to minimise the excavation and off-site removal of subgrade materials. The contamination of the filling identified by DP during the contamination assessment of the site (reported separately) would suggest that the reduction of off-site spoil disposal may therefore be advantageous for this project.

As such, those options that lend themselves to cut-to-fill operations, i.e. where a terraced ground floor level is proposed, will be governed by consideration of the findings of the contamination assessment. The placement of select filling over the existing subgrade to support the ground floor pavement is therefore the more likely approach to minimise excavation at the site. The client's desire to minimise spoil generation for off-site disposal in circumstances of subgrade contamination would also result in the need for displacement foundation pile types.

9. Comments

9.1 Site Preparation and Earthworks

Site preparation and earthworks should be carried out in accordance with the guidelines contained in AS 3798 – 2007. This includes clearing, to the minimum extent necessary for the works, of all trees, stumps and other materials unsuitable for incorporation into the works, as well as the removal of all roots and debris (such as old footings, buried pipes and the like) to sufficient depth so as to prevent obstruction of subsequent excavation and foundation works. For the proposed works, this will include:

- the demolition of the existing pavement surfaces, kerbs and gutters in the vicinity of the proposed works;
- relocation and/or decommissioning and removal of some existing buried services; and
- removal of trees and their roots.

In general, excavations that result from these activities should be backfilled. Prior to backfilling, the stripped surface should compacted to a minimum 98% of the maximum dry density for Standard compaction (AS 1289.5.1.1 – 2003) within 2% of the Standard optimum moisture content. Backfilling should then be carried out using clean, select material, with maximum 75 mm particle size and compacted in horizontal layers to a minimum 98% of the Standard maximum dry density. If site clays are used for backfilling, these will need to be placed under strict moisture control as there is a significant risk of shrink/swell movements from changes in soil moisture content due to the high plasticity of the site clays.

It is noted that particular care will be required when using compaction and earthworks machinery in close proximity to existing buildings and infrastructure so as to reduce the potential for damage due to vibrations.

Following the above site preparation activities, existing filling and natural clay will be exposed at the ground surface, with the existing filling thickening to the south east. Based on the DCP test and SPT results, the existing filling and the residual clay subgrade are apparently reasonably well compacted, and of at least stiff consistency, respectively, and can therefore be left in place provided the subgrade is treated as described in Section 9.4.

9.2 Excavations and Excavation Support

The contamination of the filling identified by DP during the contamination assessment of the site (reported separately) would suggest that the minimisation of excavation and off-site spoil disposal may be advantageous for this project. Nevertheless, should stripping of the site be required to attain the required formation level, the stripped material would likely comprise topsoil and clayey filling containing building rubble. These materials should be readily removed using conventional earthmoving equipment or hydraulic excavators, although due to the contamination of the filling, special measures would likely need to be implemented for worker health and safety reasons.

All excavated materials that are to be removed from site will need to be disposed in accordance with current OEH policies. Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from this site. Accordingly, environmental testing will need to be carried out to classify spoil prior to disposal. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site. The contamination assessment carried out by DP for this project addresses these matters in a separate report. For the purpose of waste disposal cost estimation, it can be assumed that the in situ wet density (ρ)of the filling is in the range 2.0 – 2.2 t/m^3.

Should excavations deeper than 1.2 m be required and space permits, it will be practical to batter the excavation slopes at maximum temporary batter slopes of 1.5H:1V in the filling and 1H:1V in the residual clay and shaly clay. If unsupported cut batters are not considered feasible, however, support would need to be provided to the excavations in this type of ground to avoid the risk of collapse of the sides for reasons of occupational health and safety. The design of retaining walls should be based on an average bulk unit weight of 20 kN/m^3 and on a triangular earth pressure distribution, utilising the applicable coefficient of active earth pressure shown in Table 6.

Table 6: Recommended Earth Pressure Coefficients

Unit	Active Earth Pressure Coefficient (Ka)	
	Short Term (Temporary)	Long Term (Permanent)
Filling & Silty Clay	0.25	0.3
Shaly Clay	0.2	0.25

9.3 Foundations

Consideration has initially been given to the use of shallow footings but their use is not recommended due to the magnitude of loads to be carried, and the depth of the uncontrolled filling encountered in the boreholes and test pits, as well as the contamination of the filling (refer to the separate contamination assessment report by DP).

Piled foundations are therefore recommended to transfer structural loads to the bedrock. The contamination of the filling, however, would suggest that the reduction of off-site spoil disposal may be advantageous for this project. Displacement piles, such as driven preformed or screwed cast-in-place piles, may be of benefit in these circumstances. These pile types are, however, limited in their ability to access the more competent rock strata beneath the Class V Shale and, as such, the geotechnical capacity of such piles would also be limited such that pile groups would need to be installed to support the individual columns applying the ultimate strength limit state loads anticipated in Table 5. The advantages and disadvantages of the various displacement and non-displacement pile types are identified in Table 7.

Table 7: Comparison of Pile Types

Pile Classification	Pile Type	Examples	Advantages	Disadvantages
Displacement	Driven Preformed	Concrete, steel, timber	• No spoil • Pile material can be inspected before installation • Not damaged by ground heave when driving adjacent piles • Installation unaffected by groundwater • Can be readily carried above ground level • Can be driven in long lengths	• Cannot be readily varied in length to suit varying level of bearing stratum • May break during hard driving • Uneconomical if material in pile is governed by handling and driving stresses rather than permanent loading stresses • Noise and vibration during driving • displacement of soil during pile driving in groups may damage adjacent structures or cause lifting by ground heave of adjacent piles • Diameter limited for driving, therefore limited in geotechnical capacity to the full structural capacity of the pile • End enlargement impractible
	Screwed Cast-in-place	Atlas, Omega, V	• Length can be readily adjusted to suit varying level of bearing stratum • Minor amount of spoil is produced • Noise and vibration significantly reduced	• Limited geotechnical capacity as cannot penetrate below shaly clay • Displacement of ground may damage "green" concrete of adjacent piles, or cause lifting by heave of adjacent piles • Better suited to cohesionless soils rather than cohesive soils

Pile Classification	Pile Type	Examples	Advantages	Disadvantages
Non-displacement	Supported	Continuous flight augers (CFA), cased or drilling fluid supported bored piles	• No risk of ground heave • Length can be varied to suit varying ground conditions • can be installed in large diameters • end enlargements are possible in clay soils • Pile material not dependent on handling or driving conditions • Can be installed in long lengths • no appreciable noise and vibration • In the case of CFA piles and drilling fluid supported bored piles, concrete can be placed underwater where groundwater is present within the socket • reduced risk of sidewall collapse during drilling • cased bored pile sockets and socket material can be inspected during drilling and prior to concrete placement	• For cased bored piles, difficulty with concrete placement in the presence of groundwater, unless the water is first pumped out of the pile socket • Concrete is not placed under ideal conditions and cannot be subsequently inspected • Cannot be readily extended above ground • Boring methods may loosen sandy or gravelly soils • For CFA piles, socket material cannot be inspected during installation • For CFA piles, sockets cannot be inspected prior to concrete placement • Significant amounts of spoil are generated during drilling requiring stockpiling and/or off-site disposal
	Unsupported	Bored piles	• time saving in cohesive soils and rock where side walls can stand unsupported for a short period of time • pile sockets and socket material can be inspected during drilling and prior to concrete placement	• As for cased bored piles above • Potential for sidewall collapse in cohesionless material or due to groundwater inflow

Table 8 provides pile design parameters for the foundation design for the proposed multi-storey car park. The parameters are based on non-displacement, cast-in-situ piles founded in shale bedrock. Note that displacement piles are likely to access Class V Shale only.

Table 8: Preliminary Pile Design Parameters

Shale Class	Young's Modulus (MPa)	Ultimate Parameters		Allowable (Serviceability) Parameters	
		Shaft Adhesion[3] (kPa)	End Bearing Pressure (kPa)	Shaft Adhesion[3] (kPa)	End Bearing Pressure (kPa)
V	50	100	2000	50	700
IV	100	150	3000	80	1000
III	200	350	6000	150	2500
II	700	600	30,000	300	6000

Note: 3 For pile foundations in compression only; reduce by 50% to obtain values for design against uplift. Assumes adequately cleaned and roughened pile sockets.

According to the Australian piling code AS 2159 – 2009, the design geotechnical strength of a pile ($R_{d,g}$) is the ultimate geotechnical strength ($R_{d,ug}$) multiplied by the geotechnical strength reduction factor (ϕ_g), such that:

$$R_{d,g} = \phi_g \, . \, R_{d,ug}$$

The calculated value $R_{d,g}$ must equal or exceed the structural design action effect E_d.

Selection of the geotechnical strength reduction factor (ϕ_g) is based on a series of individual risk ratings (IRR) which are weighted and lead to an average risk rating (ARR). The individual risk ratings and final value of ϕ_g depend on the following factors:

- site: the type, quantity and quality of testing;
- design: design methods and parameter selection;
- installation: construction control and monitoring;
- pile testing regime: testing benefit factor based on percentage of piles tested and the type of testing;
- redundancy - whether other piles can take up load if a given pile settles or fails.

Without details of the preferred pile type, design method, testing regime and other construction factors it is not possible for DP to calculate the appropriate ϕ_g value at this stage. A typical approach would be to adopt a conservative ϕ_g of, say, 0.4 for preliminary design purposes.

Higher values of ϕ_g could, however, be adopted by the pile designer if the individual risk ratings for each of the above-described factors are reduced by, for example, carrying out supplementary borehole investigations, adopting well-established and soundly based design methods, selecting design parameters for the founding stratum supported by previous pile load test data or laboratory testing of rock strength, and improving the level of construction control. AS 2159 – 2009 allows for the use of higher values of ϕ_g by specifying testing benefit factors that can be applied where pile testing is carried out to verify geotechnical strength and shaft integrity. Notwithstanding the above comments, where the basic geotechnical strength reduction factor (ϕ_{gb}) is greater than 0.4, the piling code requires pile testing to be carried out.

It is expected that under a design-and-construct (D & C) contract, most experienced piling contractors would incorporate pile testing with one or more of the above activities to increase the ϕ_g value to between 0.6 and 0.75 for this project.

Pile testing would be expected to primarily involve dynamic load testing (DLT), although static load testing may be appropriate, particularly where pile loads exceed that which can be practically undertaken using DLT methods. The value of testing would depend on a cost-benefit analysis that compares the cost of testing to the potential savings in pile installation.

By way of example and ignoring overlying rock classes, a bored pile of 600 mm diameter socketed 1.5 m into Class II Shale has a design geotechnical strength of 6600 kN in compression using a geotechnical strength reduction factor of 0.65. Settlements may be calculated using the elastic modulii given in Table 8.

For the above example, under a (compressive) serviceability loading of 4900 kN and assuming the elastic modulus of the concrete pile to be approximately 32,800 MPa (for f_c' = 40 MPa at 28 days), the settlement on the rock socket for the example pile is estimated to be of the order of 4 – 5 mm.

For piles proportioned on the basis of the recommended allowable (i.e. serviceability) parameters, total settlements of less than 1% of the pile diameter could be expected, with differential settlements between adjacent columns expected to be less than half this value.

If driven to refusal, the load capacity of driven piles would generally be equal to the structural capacity of the pile shaft, taking into consideration axial and lateral design loads, with allowance for load eccentricities. For most available pile hammers, maximum working load capacity is about 3000 kN for piles less than 10 m long. This would therefore require the installation of pile groups to support the anticipated column working load, if driven piles are used.

The capability of driven piles to develop adequate resistance should be demonstrated prior to construction, using pile driving prediction techniques based on wave equation analysis and the particular pile driving arrangement intended to be used by the contractor (e.g. hammer type and stroke, cushion, etc.).

Where non-displacement piles are to be installed adjacent to existing buried services, consideration should be given to the installation of temporary or permanent pile casing to a depth of, say, 2 m below the invert of the service to reduce the risk of undermining the existing buried services during drilling. If conventional bored piles are used, temporary casing will be required through filling in the southern part of the site to prevent the collapse of existing sandy filling materials into the pile hole. Should groundwater infiltration occur into bored pile sockets, then the water would need to be removed from the sockets prior to the placement of concrete.

All open foundation excavations should be clean and free of loose debris with pile sockets free of smear and adequately roughened immediately prior to concrete placement. Some softening of the socket material may occur after exposure, particularly in wet weather. Any softened or loose material within the excavation or socket length would need to be removed. Any surface water or groundwater inflow into footing excavations should be removed prior to pouring of concrete.

An experienced geotechnical professional should inspect all bored pile excavations prior to the placement of steel and concrete.

As displacement and CFA piles are 'blind' piling techniques (and hence not suited to geotechnical inspection), such piles will need to be certified by the piling contractor to confirm that the piles have reached a suitable depth, and that the material encountered is consistent with the design assumptions. Actual pile capacities for driven piles should be confirmed from driving data during construction.

9.4 Pavements

The pavement subgrade exposed following site preparation activities will partly comprise existing filling and partly residual soil. As the existing filling is contaminated (refer to separate contamination assessment report by DP), the client's preference is to leave the existing filling in place. Hence, the exposed subgrade should be rolled with a roller of 12 tonnes minimum deadweight for a minimum of six passes. A final proof roll of the compacted subgrade surface should be carried out in the presence of a geotechnical engineer to identify any 'soft spots', which if present should be treated in accordance with the advice of the inspecting geotechnical engineer. The treated surface should then be tyned and moisture conditioned (if necessary) to a depth of 150 mm and compacted using a minimum 12 tonne smooth drum roller to a minimum 98% of the maximum dry density obtained in the laboratory Standard compaction test (AS 1289.5.1.1 – 2003) within 2% of the Standard optimum moisture content.

Where filling is required to be placed over the existing subgrade to achieve the required formation level for the ground floor pavement, clean, select material with maximum 75 mm particle size should be used and compacted in horizontal layers to a minimum 98% of the Standard maximum dry density. If site clays are used for filling, these will need to be placed under strict moisture control within 2% of the Standard optimum moisture content, as there is a significant risk of shrink/swell movements from changes in soil moisture content due to the high plasticity of the site clays.

Assuming that the subgrade is prepared and compacted as suggested, and that appropriate subsurface drainage is installed to a depth of at least 0.5 m below subgrade level, then based on the laboratory CBR test results, a design CBR of 3% is suggested for pavement design purposes.

9.5 Soil Aggressivity

Based on the results of the chemical laboratory testing of natural soil samples collected from the site, and with reference to Table 6.4.2 (A) & (C) and 6.5.2 (A) & (C) in AS2159-2009, the exposure classification for piles and other buried structural elements is moderately aggressive for both concrete and steel piles in the residual clay soils.

10. Limitations

Douglas Partners (DP) has prepared this report for this project at Westmead Hospital in accordance with DP's revised proposal dated 5 August 2013 and acceptance received from Amanda Bock of Health Infrastructure dated 9 August 2013. This report is provided for the exclusive use of Health Infrastructure for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. 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 only at the specific sampling 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 anthropogenic 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 limited by undetected variations in ground conditions across the site between and beyond sampling 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 notes 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 given in this report.

This report, or sections of 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 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

References

1. Pells, P.J., Mostyn, G. and Walker, B.F. "Foundations on Sandstone and Shale in the Sydney Region". Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

2. Pells, P.J., Douglas, D.J., B., Thorne, C. and McMahon, B.K. "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region". Australian Geomechanics Journal, Vol. 3, 1978.

3. Walker, B.F. and Pells, P.J.N. "The Construction of Bored Piles Socketed into Shale and Sandstone". Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.

Appendix A

About this Report

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

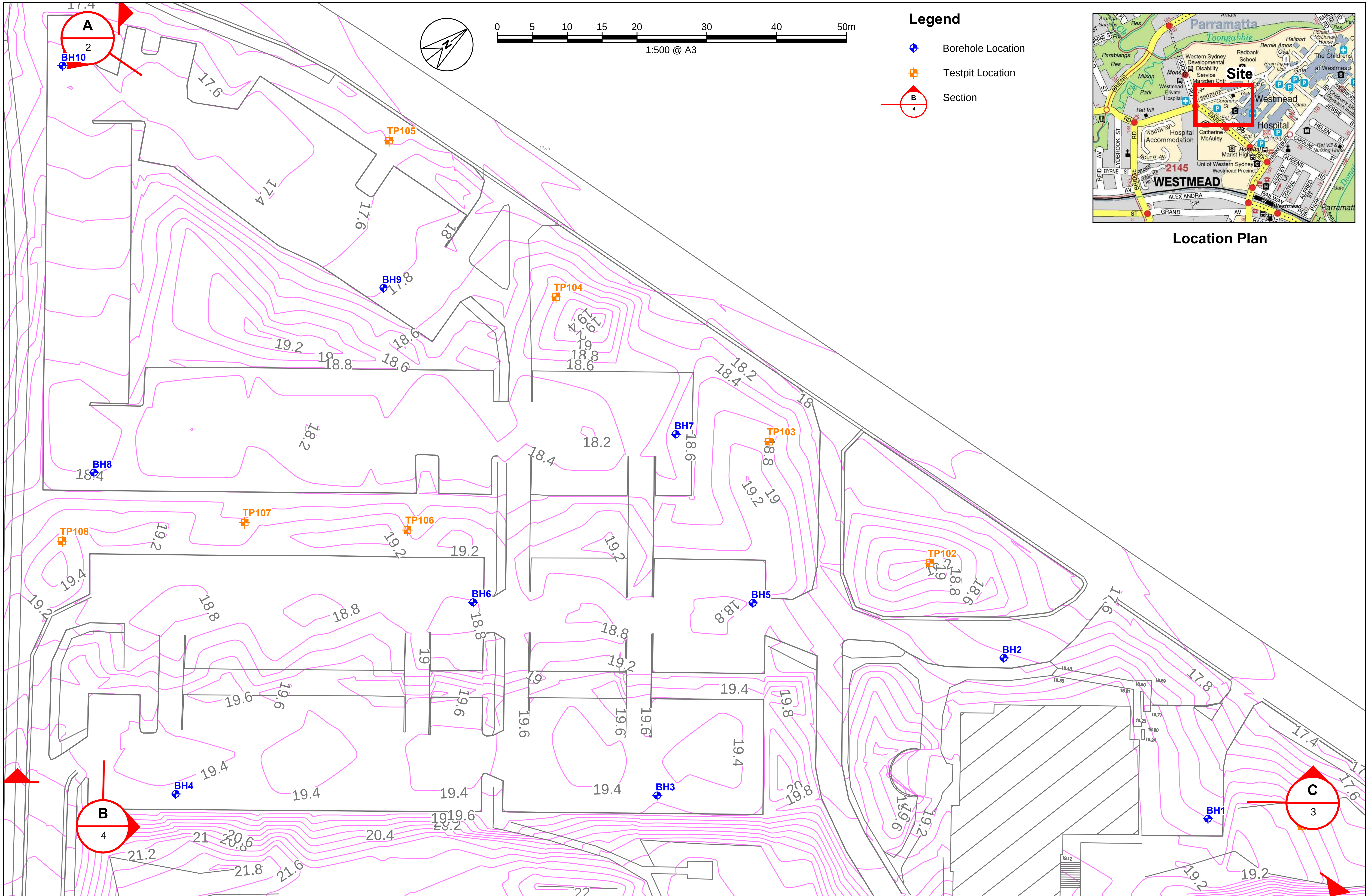
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

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

Appendix B

Drawings



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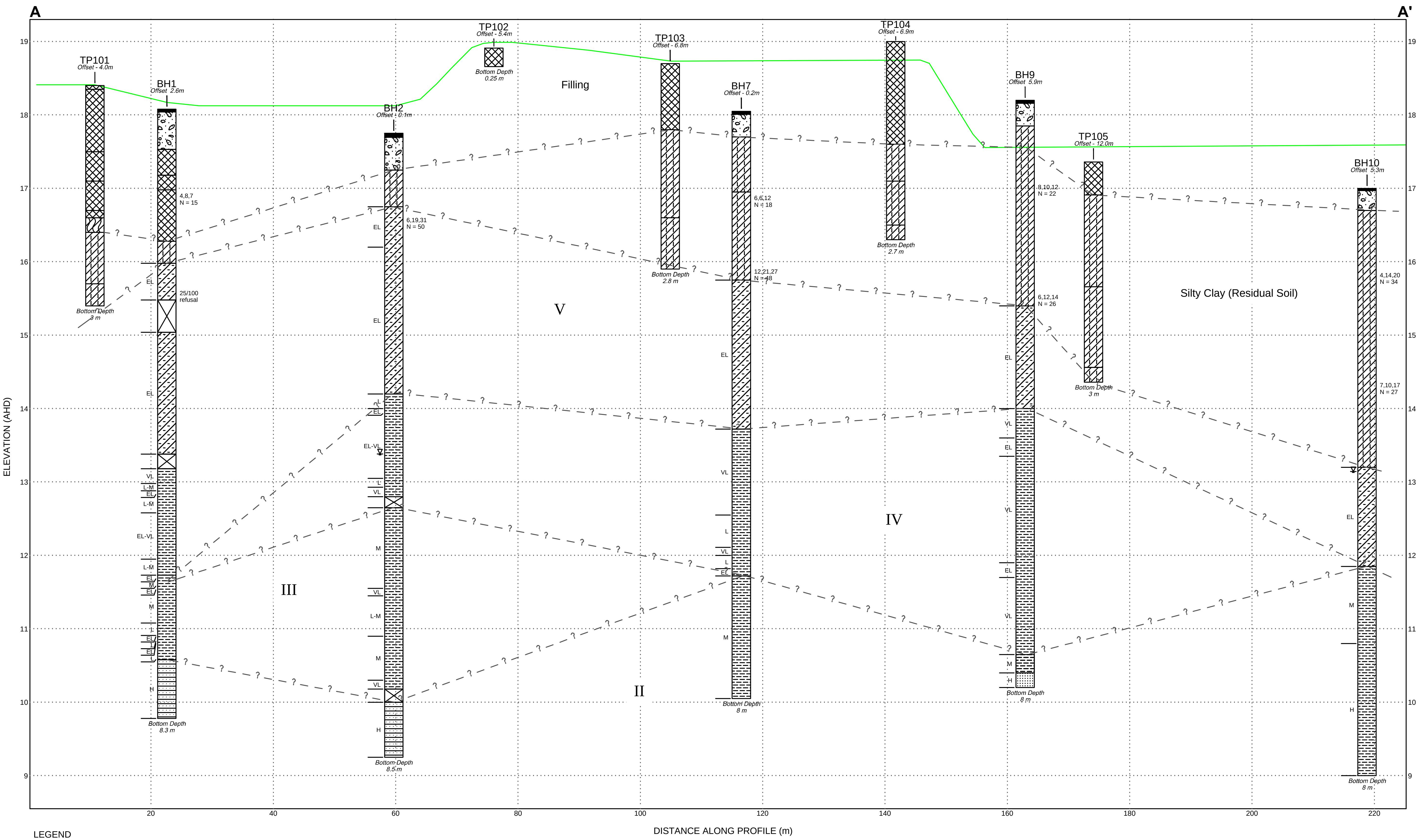
Date printed: 14 Jan 2014 - 04:21PM



LEGEND

- Core Loss
- Asphaltic Concrete
- Filling
- Laminite
- Roadbase
- Sandstone
- Shale
- Shaly Clay
- Silty Clay
- Topsoil

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

TESTS / OTHER
N - Standard penetration test value
▽ - Water level
— ? — — Inferred Geotechnical stratum boundary

ROCK CLASSIFICATION
V - Class V Shale
IV - Class IV Shale
III - Class III Shale
II - Class II Shale

NOTE:
1. Summary logs only. Should be read in conjunction with detailed logs.
2. Rock classification in accordance with Pells et al (1998)

0 12
Horizontal Scale (metres)

Vertical Exaggeration = 12.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Health Infrastructure		TITLE: **Cross-section A-A'**	PROJECT No: 73603.00
OFFICE: Sydney	DRAWN BY: JCP	**Westmead Car Park**	DRAWING No: 2
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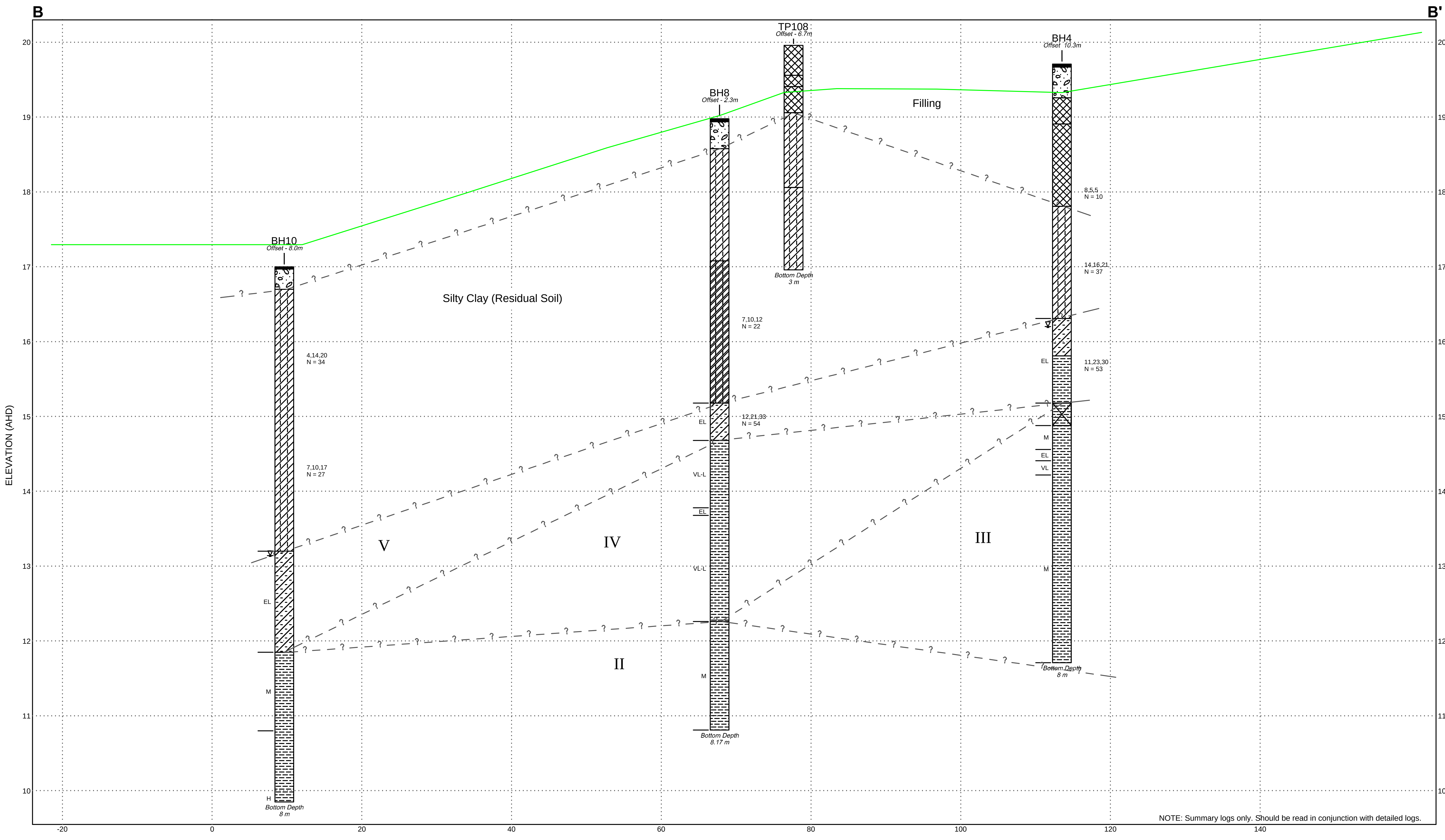

LEGEND

- Core Loss
- Asphaltic Concrete
- Filling
- Roadbase
- Shale
- Shaly Clay
- Silty Clay

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

TESTS / OTHER
N - Standard penetration test value
- Water level
— ? — — Inferred Geotechnical stratum boundary

ROCK CLASSIFICATION
V - Class V Shale
IV - Class IV Shale
III - Class III Shale
II - Class II Shale

NOTE:
1. Summary logs only. Should be read in conjunction with detailed logs.
2. Rock classification in accordance with Pells et al (1998)

0 10
Horizontal Scale (metres)

Vertical Exaggeration = 10.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Health Infrastructure		TITLE: **Cross-section B-B'**	PROJECT No: 73603.00
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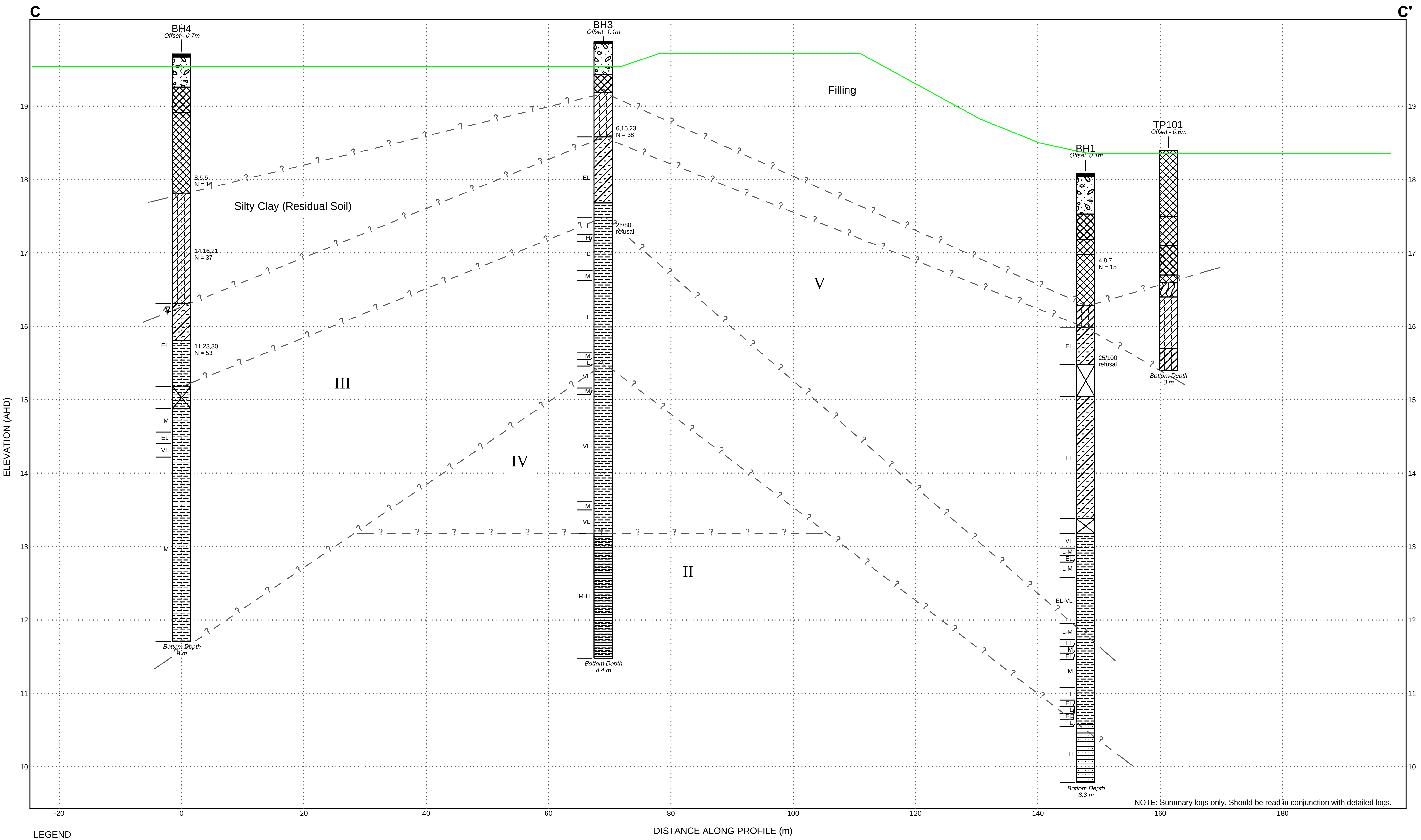

LEGEND

Core Loss	Roadbase
Asphaltic Concrete	Shale
Filling	Shaly Clay
Laminite	Silty Clay
Topsoil	

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

TESTS / OTHER
N - Standard penetration test value
Water level
— ? — — Inferred Geotechnical stratum boundary

ROCK CLASSIFICATION
V - Class V Shale
IV - Class IV Shale
III - Class III Shale
II - Class II Shale

NOTE:
1. Summary logs only. Should be read in conjunction with detailed logs.
2. Rock classification in accordance with Pells et al (1998)

0 12
Horizontal Scale (metres)
Vertical Exaggeration = 12.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Health Infrastructure		TITLE: **Cross-section C-C'**	PROJECT No: 73603.00
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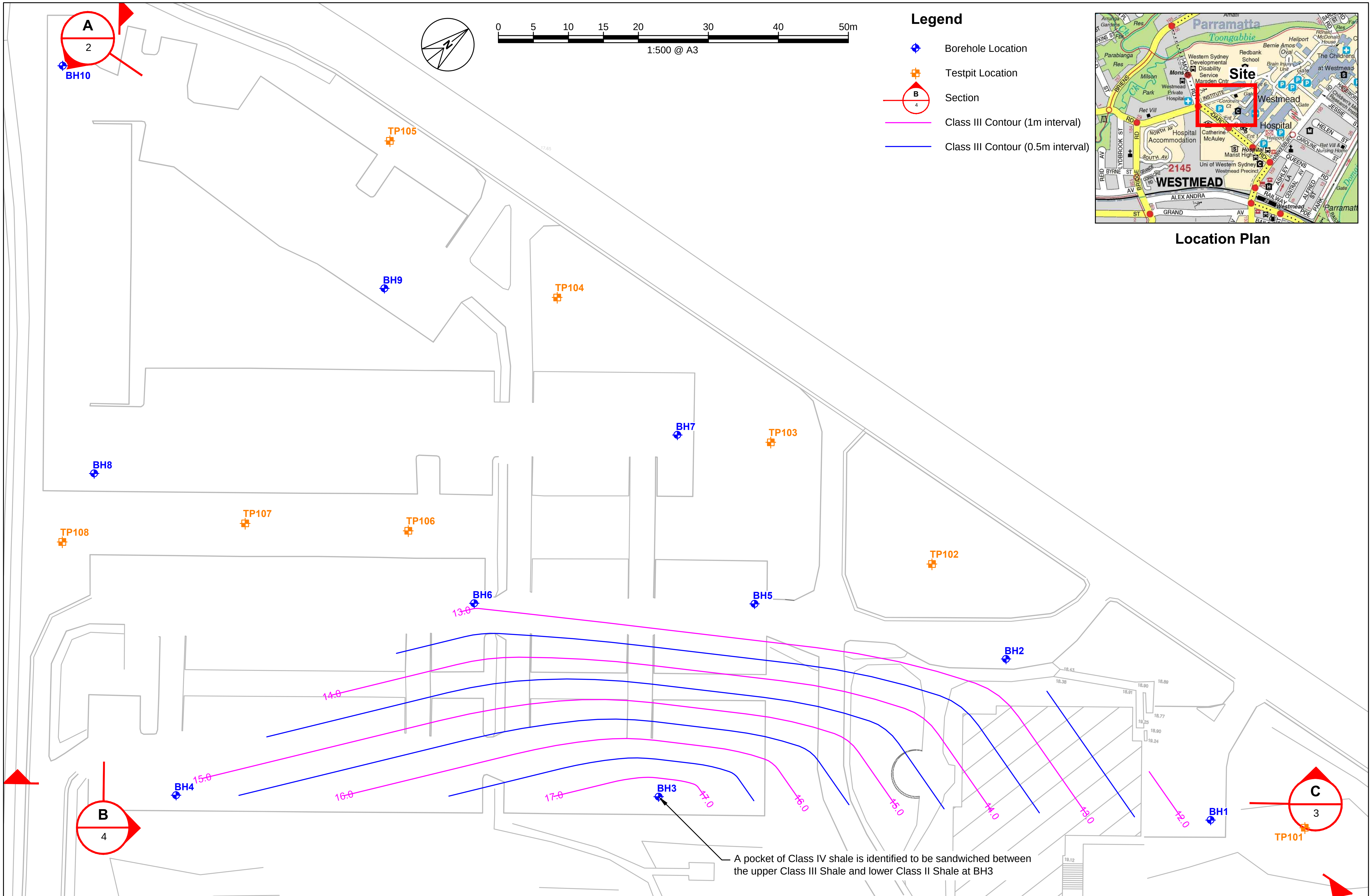

Douglas Partners — Geotechnics / Environment / Groundwater	CLIENT: Health Infrastructure		TITLE: Rock Contour Plan - Interpreted Top of Class III Shale	PROJECT No: 73603.00
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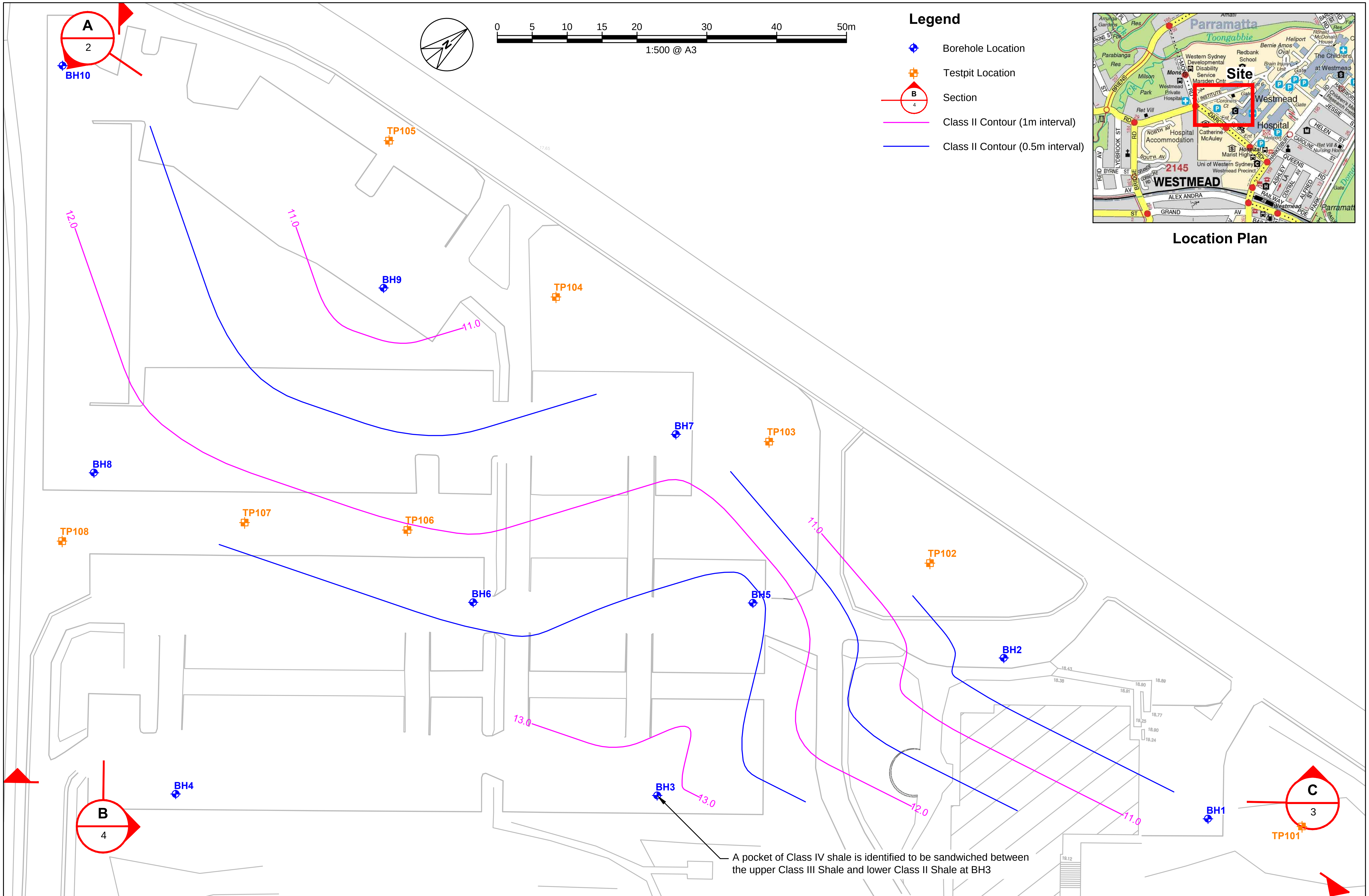

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Health Infrastructure	TITLE: **Rock Contour Plan - Interpreted Top of Class II Shale**	PROJECT No: 73603.00
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Appendix C

Field Work Results

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.

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

 Asphalt

 Road base

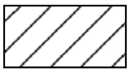
 Concrete

 Filling

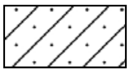
Soils

 Topsoil

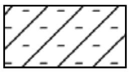
 Peat

 Clay

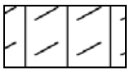
 Silty clay

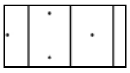
 Sandy clay

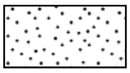
 Gravelly clay

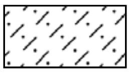
 Shaly clay

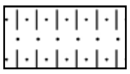
 Silt

 Clayey silt

 Sandy silt

 Sand

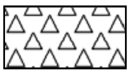
 Clayey sand

 Silty sand

 Gravel

 Sandy gravel

 Cobbles, boulders

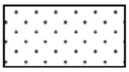
 Talus

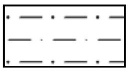
Sedimentary Rocks

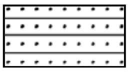
 Boulder conglomerate

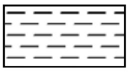
 Conglomerate

 Conglomeratic sandstone

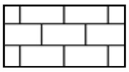
 Sandstone

 Siltstone

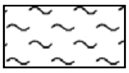
 Laminite

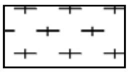
 Mudstone, claystone, shale

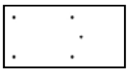
 Coal

 Limestone

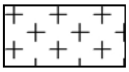
Metamorphic Rocks

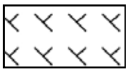
 Slate, phyllite, schist

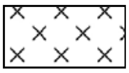
 Gneiss

 Quartzite

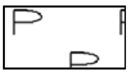
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

 Dacite, epidote

 Tuff, breccia

 Porphyry