Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Development,
Hunter Sports High School
Pacific Highway, Gateshead

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Table of Contents

Appendix A:	CSIRO-BTF 18
	About this Report
	Sampling Methods
	Soil Descriptions
	Rock Descriptions
	Symbols and Abbreviations
Appendix B:	Borehole Logs (Bores 101 to 107)
	Core Photoplates (Bores 101 to 104)
	Results of Dynamic Penetrometer Tests
Appendix C:	Laboratory Test Results
Appendix D:	Pile Capacity Estimate Charts
Appendix E:	Drawing 1 – Test Location Plan
	Drawing 2 – Section A-A’

Report on Geotechnical Investigation
Proposed Development, Hunter Sports High School
Pacific Highway, Gateshead

1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for a proposed redevelopment at Hunter Sports High School, Pacific Highway, Gateshead. The assessment was commissioned on 8 September 2015 by the NSW Department of Finance, Services and Innovation (Purchase Order 3000156955) and was undertaken with reference to DP proposal NCL150571 dated 2 September 2015.

It is understood that a redevelopment of the site is proposed, comprising construction of multiple two and three storey buildings, including a gymnasium / hall, workshops and food tech area. Existing buildings (including the 'Bini shell' structure / "G" Block and the Library / "L" Block) as well as the existing carpark will be demolished prior to construction.

A geotechnical investigation was required to provide information on subsurface conditions and comments on the following:

- Suitable footing types and indicative soil and rock parameters for footing design;
- Indicative geotechnical parameters for retaining wall design;
- Flexible pavement thickness design for the proposed carpark;
- Mine Subsidence Desktop Study Risk Assessment (which is reported separately).

The investigation comprised the drilling of seven bores, laboratory testing, engineering analysis and reporting.

DP has previously undertaken a geotechnical investigation, Mine Subsidence Desktop Study Risk Assessment and Preliminary Site Investigation (Contamination) relating to development previously proposed to the south of the current investigation area, within the Hunter Sports High School site (Project 81598, September 2014). It is understood that the development proposed at that time has been superseded by the development in the current investigation area.

2. Site Description and Regional Geology

The site is located at Hunter Sports High School, Pacific Highway, Gateshead as shown on Drawing 1 in Appendix E. At the time of the investigation the school site contained various school buildings, including classrooms and sporting facilities, car parking areas, open playing fields and numerous semi - mature to mature trees. The school site generally slopes to the south west at about 3° to 5° with localised steeper areas (particularly between "B" Block, and "L" and "G" Blocks).

The site of proposed development is within the central part of the school site and is currently occupied by staff car parking, “G” Block, “L” Block, “A – E” Block and a quadrangle. The site is bound to the east by the Pacific Highway, to the south by existing school features including basketball / netball courts and a gymnasium; to the west by an existing service road and grassed sports fields; and to the north by existing school buildings.

Existing site features are shown on Figures 1 to 3.

Figure 1: Looking south-west towards Bore 104 (proposed three storey building).

Figure 2: Looking south-west to Bore 106 (proposed gymnasium / hall to the left).

Figure 3: Looking north toward the existing Admin Block (proposed building over COLA); rig on Bore 105

Reference to the 1:31,680 Surface Geology of the Newcastle Coalfield geological map indicates that the majority of the school site is underlain by the Permian aged Kahibah Formation of the Adamstown subgroup of the Newcastle Coal Measures. The Kahibah Formation typically comprises conglomerate, sandstone, siltstone, coal and tuff.

The Montrose Coal Seam is shown to outcrop to the north-west of the site. Previous work in the Gateshead area indicates that the Montrose Coal Seam is sometimes associated with shallow groundwater which can also be under artesian pressure.

The conditions encountered in the geotechnical investigation were generally consistent with the geotechnical mapping.

Reference to the Acid Sulphate Soil Risk Map for Wallsend prepared by the Department of Land & Water Conservation indicates that the site is in an area of no known occurrence of acid sulphate soils.

3. Field Work Methods

The field work was carried out in the period of 28 September 2015 to 30 September 2015 and comprised the drilling of seven bores (Bores 101 to 107).

The bores were drilled with a four wheel drive mounted rotary drilling rig equipped with solid flight augers and wash boring equipment for drilling in soil and weathered rock, as well as NMLC diamond coring equipment for coring bedrock.

Bores 101 and 106 were drilled to 7.1 m and 2.5 m depth in the vicinity of the proposed new gymnasium / hall and were taken to the limit of investigation and tungsten carbide (TC) bit refusal respectively.

Bores 102 and 105 were drilled to 6.2 m and 2.2 m depth in the vicinity of the proposed building over cola and were taken to the limit of investigation and TC-bit refusal respectively.

Bores 103, 104 and 107 were drilled to depths ranging from 1.1 m to 13.2 m in the vicinity of the proposed TAS Workshops and were taken to the limit of investigation (Bores 103 and 104) / TC-bit refusal (Bore 107).

Standard penetration tests (SPTs) were performed at selected depths within the soil / weathered rock at each bore location.

Rock coring was performed in Bores 101 to 104. However, some rock core was not recovered in Bore 101, probably due to the rock material being of extremely low strength and the rock structure breaking down / disintegrating during the coring process.

The test locations were set out by a geotechnical engineer from DP. The engineer also logged the subsurface conditions encountered in the bores and collected samples for subsequent laboratory testing and identification purposes. The engineer boxed and photographed the rock core and carried out point load strength index tests on the core. Dynamic penetrometer tests were carried out to about 1 m depth in accordance with AS1289.6.3.2 at each bore location.

Reduced levels at the bore locations were interpolated from a client supplied plan. The MGA coordinates at each bore location were recorded using a hand held GPS unit which is normally accurate to within about ±5 to 10 m depending on satellite coverage.

The approximate locations of the bores are shown on Drawing 1 in Appendix E.

4. Field Work Results

The subsurface conditions encountered in the bores are presented in detail in the borehole logs in Appendix B along with the core photoplates and results of the dynamic penetrometer tests. The borehole logs should be read in conjunction with the accompanying notes in Appendix A which explain the descriptive terms and classification methods used in the logs. The subsurface conditions encountered are summarised in Tables 1 to 3.

Table 1: Summary of Subsurface Conditions – Gymnasium / Hall (Bores 101 and 106)

Depth (m)		Strata	Description
From	To		
0	0.3 / 0.4	Filling	Spray seal wearing surface, overlying sandy gravel road base filling
0.3 / 0.4	1.0 / 1.2	Clayey Silt / Silty Clay	Typically very stiff to hard
1.0 / 1.2	2.1 / 2.3	Siltstone	Typically extremely low to medium strength
2.3	7.1	Sandstone / Pebbly Sandstone	Typically low to medium strength (Bore 101 only)

Table 2: Summary of Subsurface Conditions – Three Storey Building over COLA (Bores 102 and 105)

Depth (m)		Strata	Description
From	To		
0.0	0.2 / 0.3	Filling	Spray seal wearing surface, overlying sandy gravel (slag) filling
0.3	1.6	Filling	Brown silty clay filling (Bore 105 only)
0.25 / 1.6	1.9 / 2.3	Clayey Silt / Silty Clay / Silt	Typically stiff to very
2.3	4.9	Carbonaceous Silty Clay / Weathered Coal	Stiff to very stiff (Bore 102 only)
4.8	5.2	Coal	Typically very low strength (Bore 102 only)
1.9 / 5.2	2.2 / 5.4	Siltstone	Typically extremely low strength
5.4	6.2	Laminite	Typically low strength (Bore 102 only)

Table 3: Summary of Subsurface Conditions – TAS Workshops (Bores 103, 104, and 107)

Depth (m)		Strata	Description
From	To		
0	0.2	Filling	Clayey sand filling (Bore107 only)
0	0.2 / 0.3	Sand	Typically medium dense (Bores 103 and 104)
0.2 / 0.3	0.6 / 5.2	Clayey Silt / Clay / Silt	Typically firm to very stiff
3.8	5.5	Carbonaceous Silty Clay	Typically stiff to very stiff (Bore104 only)
5.2	6.4	Sandstone / Siltstone	Typically extremely low to very low strength (Bore 103 only)
5.5 / 7.6	6.2 / 11.2	Coal	Typically extremely low to medium strength (Bores 103 and 104 only)
0.9 / 11.2	1.1 / 13.2	Siltstone / Laminite	Typically extremely low to medium strength.

The following table summarises the approximate depth to bedrock, including the depth to V-bit and TC-bit refusal (where encountered), in the bores.

Table 4: Depth to Rock

Bore No.	Surface RL (AHD)	Top of Rock		V-bit Refusal		TC-bit Refusal		Medium Strength Bedrock	
		Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)	Depth (m)	RL (AHD)
101	27.2	1.1	26.2	1.1	26.2	-	-	5.7	21.5
102	28.8	4.8	24.0	4.8	24.0	-	-	5.9	22.9
103	25.5	5.2	20.3	5.5	20.0	-	-	11.7*	13.8
104	25.5	5.5	20.0	5.8	19.7	-	-	7.3	18.2
105	28.6	1.9	26.7	2.2	26.4	2.2	26.4	-	-
106	25.8	1.3	24.6	1.4	24.5	2.5	23.3	-	-
107	25.9	0.6	25.3	0.7	25.2	1.1	24.8	-	-

Notes to Table 4: *Bore 103 encountered medium strength coal at 8.5 m depth.

Free groundwater was observed in Bores 101 and 103 at depths of 1.2 m and 2.9 m respectively (approximately RL 26.1 m to 22.6 m AHD, respectively). No free groundwater was observed in Bores 105 to 107 whilst augering. The use of drilling fluids below the augered depths at Bores 102 and 104 precluded the observation of groundwater. It should be noted that groundwater levels are affected by factors such as recent weather conditions and soil permeability and will vary with time.

5. Laboratory Testing

Laboratory testing comprised two shrink-swell tests.

Detailed laboratory test result sheets are included in Appendix C and are summarised in Table 5 below.

Table 5: Results of Laboratory Testing

Bore	Depth (m)	Description	FMC (%)	Iss (% per ∆pF)
105	1.7 – 1.81	Clayey Silt / Silty Clay – Grey mottled orange	28.1	3.7
103	0.35 – 0.8	Sandy Clay	22.1	1.6

Notes to Table 5:

FMC	- Field moisture content	Iss	- Shrink-Swell Index

Axial and diametral point load testing was carried out on selected rock core samples taken from Bores 101 to 104. The results of the testing indicates Point Load Strength Index ($I_{s(50)}$) values within the range of extremely low strength rock to high strength rock and the $I_{s(50)}$ results are shown on the borehole logs in Appendix B.

6. Proposed Development

It is understood that it is proposed to demolish a number of the existing school buildings to make way for the construction of multiple two and three storey buildings including a gymnasium / hall (movement complex), workshops and food tech area at Hunter Sports High School, Pacific Highway, Gateshead. Proposed excavation depths, retaining wall heights and structural loads were unknown at the time of writing this report.

7. Comments

7.1 General

Based on the current and previous (Ref 1) geotechnical investigations at the site, and in the immediate surrounds, the geotechnical conditions pertinent to design and construction at the site are as follows:

- Variable depth to rock;
- The presence of coal seams;
- The presence of carbonaceous clay layers;
- Groundwater.

The following sections provide comment on the items listed above and in Section 1.

7.2 Site Classification

Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with normal seasonal variation in moisture. The site classification is based on procedures presented in AS 2870-2011 (Ref 2), the typical soil profiles revealed in the bores and the results of laboratory testing.

The site is underlain by carbonaceous clays (completely weathered coal), the shear strength of which is highly sensitive to changes in moisture, and while stiff to very stiff in their current state, they will soften appreciably if subjected to increased moisture.

Therefore, the design and construction of the footings must account for the moisture-sensitive ground conditions. Associated risks include a loss of bearing capacity if the soils become saturated for any reason, and differential movement of footings with variations in soil moisture.

Therefore, while geotechnical design parameters for shallow footings are provided herein, it is strongly recommended that consideration be given to supporting column loads on deep foundations (piles) founded within the underlying bedrock.

The site classification in the area of the proposed two and three storey buildings is generally considered to be commensurate with a Class M classification, with the exception of the area of the proposed building over cola which is Class P due to 2 m depth of filling encountered in Bore 105. Provided all the footings are founded in natural material below the filling, it is suggested that reactive soil movements commensurate with a Class M site should be accommodated in design.

The site classification given above is for normal seasonal moisture fluctuations without the influence of trees. It is noted that there were some trees on parts of the school site. The presence of the trees can increase the soil suction and therefore increase reactive clay movement. Removal of the trees prior to construction and the associated suction change is expected to result in swell movements that are additional to the characteristic surface movements. Reference should be made to AS2870-2011, Appendix CH for guidance on design of footings to take into account the presence of existing or proposed trees.

Site classification, as above, is based on the information obtained from the bores and on the results of laboratory testing, and has involved some interpolation between data points. In the event that the conditions encountered during construction are different to those presented in this report, it is recommended that advice be obtained from this office.

It is noted that site classification applies to residential development, as per AS2870-2011, however the principles of design, construction and maintenance can be applied to other developments.

It should be noted that this classification is dependent on proper site maintenance, which should be carried out in accordance with the CSIRO Sheet BTF-18, “Foundation Maintenance and Footing Performance: A Homeowners Guide” in Appendix A and with AS 2870-2011 (Ref 2).

Design, construction and maintenance should take into account the need to achieve and preserve an equilibrium soil moisture regime beneath and around buildings. Such measures include designing paved areas around buildings to fall away from the building, flexible plumbing connections and service trenches to be backfilled with compacted clay. These and other measures are described in AS 2870-2011 (Ref 2) and the CSIRO-BTF18 publication in Appendix A.

Masonry walls should be articulated in accordance with TN61 (Ref 3) to minimise the effects of differential movement.

7.3 Shallow Footings

Founding conditions are expected to range across the site from bedrock (extremely low to medium strength), to generally stiff to very stiff clay and silty clay. To minimise the risk of differential settlement from founding on materials of differing stiffness, it is recommended that footings supporting multi-level structures should be supported on rock in all areas.

The proposed movement complex will be single storey. Relatively shallow bedrock was encountered within Bores 101 and 106 (depths of 1.05 m and 1.25 m respectively). Shallow pad or strip footings, should be founded beneath the filling in very stiff or better clay or weathered rock.

Shallow footings founded in very stiff or better clay at a depth of at least 0.5 m and up to 1 m wide should be proportioned for a maximum allowable bearing pressure of 150 kPa.

However, due to the potential presence of carbonaceous clay layers, if pad footings are to be founded in clay, the following should be undertaken as a minimum:

- Footing excavations should be inspected and proved by a geotechnical engineer;
- A blinding layer of concrete should be placed following excavation and inspection of footings, to protect the base of the footing from exposure to the elements and potential softening;

Drawing 2 in Appendix E shows the interpreted top of rock levels (AHD), based on the results of the bores. The interpolated rock level should be treated with caution, because it is based on interpolation between a small number of data points, however Drawing 2 provides a guide of approximate potential founding levels.

The depth to rock encountered in the bores would suggest that that a combination of pad footings and piles may be suitable for support of building loads.

Shallow footings founded in extremely low strength or better rock at a depth of at least 0.5 m and up to 1 m wide should be proportioned for a maximum allowable bearing pressure of 600 kPa.

The base of the footings should be founded below a line of 45° subtended from the toe of any cuttings or base of service trenches.

7.4 Piled Footings

Piles could be used for support of structural loads. Based on the results of the field work, including the presence of relatively shallow groundwater at the location of Bores 101 and 103, it is considered that suitable pile types include:

CFA Piles – Continuous Flight Auger (CFA) piles are drilled to their nominated depth, after which the augers are withdrawn at a controlled rate and grout is pumped into the hole. There is a tendency to produce a softened remoulded skin around the pile, leading to relatively low shaft adhesion, however in this application the piles would primarily be end-bearing.

Bored Piles – Traditional bored piles are expected to be suitable, where founded on the underlying bedrock. Temporary liners may be required to control groundwater inflow where groundwater is encountered above the base of the pile. The base of the pile hole should be cleaned of debris and water prior to placement of concrete. Conventional uncased bored piles could also be considered for use at the movement complex site.

Where coal was encountered it is recommended that piles be used to transfer the column loads to the rock underlying the coal. Driven piles are not expected to be suitable due to the strength of the coal encountered in Bore 103 as well as the risk that the noise and vibration associated with installation of driven piles could possibly cause distress to some components of the existing school buildings and possibly nearby residents / businesses. This risk should be assessed by the piling contractor, based on the type of equipment proposed.

Based on the subsurface conditions encountered in the bores and the results of point load testing, the suggested geotechnical parameters for the design of CFA and bored piles are presented in Table 6.

Table 6: Allowable End Bearing Pressure and Shaft Adhesion for Bored and CFA Piles

Material	Top RL of founding material (m AHD)							Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion (kPa)
	Bore								
	101	102	103	104	105	106	107		
Stiff to very stiff clayey silt / silty clay	26.9	28.6	25.5	25.2	27.0	25.4	25.7	300	25
Extremely low strength rock	26.2	23.9	19.7	20.0	26.7	24.6	25.3	600	60
Very low strength	NE	NE	14.3	NE	26.5	NE	NE	1000	100
Low strength	23.3	23.4	NE	18.9	NE	24.5	25.0	1500	150
Medium strength	21.5	22.9	13.7	18.2	NE	NE	NE	3500*	350*

Notes to Table 6:

1. Allowable (working) capacity is approximately 75% of $R_{d,g}$ (where $R_{d,g}$ is the Design Geotechnical Strength) as defined in AS2159-2009.
2. NE: Not encountered
3. *The use of these design parameters would require 'proving' of the bedrock to at least 1.5 pile diameters below the base of the pile.

It is noted that limited data was obtained within the medium strength bedrock during the current investigation. If design is to be based on founding within medium strength bedrock using the parameters provided in Table 6, additional coring of the bedrock should be undertaken to prove the continuity of this stratum, to depths of at least 1.5 pile diameters below proposed pile founding depth.

The shaft adhesion should only be calculated for that part of the socket length which is the greater of 1.5 times the pile diameter or 1 m below the ground surface (relative to the top of the pile).

The estimated allowable axial capacity (in compression only) as a function of RL at Bores 101 to 104 has been estimated based on the parameters given in Table 6, and is displayed on the pile capacity charts in Appendix D. The estimated capacities are given for several pile diameters, and are relevant only for CFA or bored piles.

Total settlement of up to about 1% of the pile diameter is expected for piles in axial compression proportioned as above.

Bored pile excavations should be cleaned of all loose material and if water is present in the bore this should be removed or the concrete should be added to the base of the bore using tremmie techniques to displace the water above the concrete. Accordingly, it is recommended that DP be engaged during pile excavation to undertake pile hole inspections to confirm the design parameters provided in Table 6. In this regard, it is noted that free groundwater was encountered at depths of 1.2 m (Bore 101) and 2.9 m (Bore 103) while the bores remained open.

Prospective piling contractors should confirm the expected penetration and pile capacities achievable with their equipment.

The chemical aggressiveness of soil or groundwater towards buried structures was not assessed as part of this investigation. A review of the proposed design should be undertaken before construction commences to determine whether additional soil testing is required and the need for corrosion protection measures.

The parameters provided within Table 6 are considered appropriate for piles which are subject to geotechnical inspection during construction. If inspection is not possible, for example due to water, or piling techniques (e.g. CFA), it is recommended that pile capacities should be downgraded by 25%.

7.5 Excavations and Batters

The proposed bulk excavation depths were not known at the time of preparation of this report, however are expected to be less than about 1 m depth. The bores typically encountered stiff to very stiff clay and weathered rock in the upper 1 m. It is expected that conventional equipment such as hydraulic excavators will be adequate for the majority of bulk excavations in clay and weathered rock.

However, deeper excavation into rock may be required for deepened pad / strip footings or service trenches in which case excavation of low to medium strength or better rock, if encountered, could, possibly require the use of rock hammers for detailed excavations such as footings and service trenches.

The selection of methods and equipment for rock excavation should be undertaken by the contractor who should take into account the factors described above, together with economical production rates.

Permanent cut slopes in very stiff clay or weathered rock up to a maximum of 2 m vertical height should be battered at 1V:2H or flatter. However, flatter slopes (at least 1V:3H) are suggested if maintenance and machinery access is required. If it is proposed to excavate more than 2 m vertical height, further advice on batter slopes and stability should be obtained from this office.

Fill batters up to 2 m height should be 1V:3H or flatter or supported by a retaining wall. It is recommended that measures be taken to protect batter slopes against erosion by methods such as topsoiling and grassing.

7.6 Retaining Walls

7.6.1 Temporary Excavation

The following geotechnical matters should be considered in design and construction for retaining walls on the site, as well as for adjacent existing structures which are not proposed for demolition:

- Short term stability of the soil and rock profile. The soils are generally of stiff or better consistency and the bedrock is typically extremely low strength or better and would be expected to stand unsupported in the short term. However, there would be the possibility of localised dry friable lumps dislodging. This may be exacerbated by prolonged exposure and adverse weather. The risk could be reduced by ensuring a short exposure period, and undertaking the construction in sections, if feasible;
- Temporary batter slopes: stiff clay should be battered no steeper than 1.5H:1V and extremely low strength rock 1H:1V in the short term for cuts up to about 2 m height;
- The presence of groundwater seepage, encountered at different levels in the bores, could adversely affect better stability which should be assessed during excavation.

7.6.2 Design Parameters

For permanent retaining walls, where the wall will be free to deflect, design may be based on “active” (K_a) earth pressure coefficients, assuming a triangular earth pressure distribution. This would comprise any non-propped or laterally unrestrained walls (e.g. cantilever type walls).
The suggested long term (permanent) design soil and rock parameters are shown in Table 7 below. Any additional surcharge loads, including those imposed by adjacent land use and inclined slopes, during or after construction, should be accounted for in design.

Table 7: Geotechnical Parameters for Retaining Structures

Parameter	**Symbol**	**Clay and Carbonaceous Clay**	**Weathered Rock**
Bulk Density	γ	18 kN / m^3	20 kN / m^3
Effective Cohesion	c'	0 kPa	5 kPa
Angle of Friction	ϕ'	25°	32°
Active Earth Pressure Coefficient	K_a	0.4	0.3

Backfill placed behind the wall should be free-draining (20 mm single size gravel or coarser) and connected to the wall drainage system. A slotted drainage pipe should be placed at the base of the backfill which should all be encapsulated in a geotextile fabric. Alternatively, the retaining wall should be designed for full hydrostatic pressure.

A clay lining, a dish drain or impermeable surface should be formed at the top of the wall backfill to prevent stormwater overland flow surcharging the retaining wall.

Cantilever walls should not be used to support any adjacent building foundations or underground services. The wall should be designed for an at rest earth pressure coefficient (K_o) of 0.6, plus any surcharge from the footings if support of adjacent footings is required.

The stiff or better clay or rock would be a suitable bearing stratum for retaining wall footings which should be proportioned for a maximum allowable bearing pressure of 100 kPa in clay and 600 kPa in extremely low strength or better rock.

7.7 Pavement Design

The following pavement thickness design is in accordance with Austroads – Guide to Pavement Technology (Ref 4).

7.7.1 Design Traffic

It is understood that the carpark pavement will be trafficked by predominantly light vehicles with the occasional garbage and delivery trucks. Austroads (Ref 4) provides indicative design traffic values for lightly trafficked roads. With regard to design traffic for the new pavement the traffic load is based on a minor street with two lane traffic, 3% heavy vehicles, a design life of 20 years for flexible pavement.

Table 8 indicates the design traffic loading on which the pavement thickness design is based.

Table 8: Design Traffic

Pavement Type	Design Traffic
Flexible Pavement	4×10^3 ESA

If the traffic loading is to be significantly different from the above, the pavement thickness should be reviewed.

7.7.2 Subgrade CBR

The results of previous laboratory testing (Ref 1) on clay subgrade soil from the investigation indicated a four day soaked CBR value of 6%. Dynamic penetrometer testing at the current bore locations, however, indicated an in situ CBR of about 4% to 6% in the upper clay soils.

Based on the results of the field testing and experience with similar material a subgrade CBR of 4% was adopted for design purposes.

7.8 Flexible Pavement Thickness Design

The flexible pavement thickness design for the proposed carpark pavement is presented in Table 9, below.

Table 9: Flexible Pavement Thickness

Pavement Layer	Thickness (mm)
Wearing Course	30(1)
Basecourse	100
Subbase	160
Total	290

Notes to Table 9:

1 30 mm thickness of AC10. A 7 mm prime seal should be placed over the basecourse prior to placement of the AC.

The pavement thickness presented above is dependent on the provision and maintenance of adequate surface and subsurface drainage. Surface grades should be sufficient to prevent ponding of stormwater.

It is expected that there may be an increased maintenance requirement in areas of tightly turning trucks due to the high shear / torsional stresses applied to the pavement surface. The use of a stiffer binder (i.e. Class 600 bitumen) in the asphalt and a wearing course of 40 mm AC14 would be expected to reduce the damage to the asphalt surface in areas of tightly turning heavy vehicles. Alternatively, a concrete pavement would be expected to provide increased durability in this regard.

The recommended material quality and compaction requirements for sealed flexible pavement are presented in Table 10, below.

Table 10: Material Quality and Compaction Requirements – Sealed Flexible Pavement

Pavement Layer	Material Quality	Compaction Requirements
Basecourse	CBR ≥ 80%, PI ≤ 6%. Grading in accordance with SR41 (Ref 5)	Compact to at least 98% dry density ratio Modified (AS 1289.5.2.1, Ref 6)
Subbase	CBR ≥ 30%, PI ≤ 12%. Grading in accordance with SR41 (Ref 5)	Compact to at least 95% dry density ratio Modified (AS 1289.5.2.1, Ref 6)
Select Subgrade*	Soaked CBR ≥ 15%	Compact to 100% dry density ratio Standard (AS 1289.5.1.1, Ref 7)
Subgrade	CBR ≥ 4%	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1, Ref 7)

Notes to Table 10:

CBR – California bearing ratio (4 day soaked)

PI – Plasticity Index

* If required, refer Section 7.9

7.9 Subgrade Preparation

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

- Excavate to design subgrade level;
- Remove any additional topsoil, uncontrolled filling or deleterious materials. Tree stumps / tree roots should be removed and backfilled with approved select subgrade material;
- Proof roll the excavated surface to assess moisture content and soft zones. Remove soft zones and replace with compacted approved filling. Moisture contents should be in the range -4% (dry) to -1% (dry) OMC, for pavements where OMC is the optimum moisture content at standard compaction. If wet subgrade conditions are encountered, the material should either be tyned and allowed to dry or removed and replaced with a select subgrade (CBR>15%). The depth of any excavation should be confirmed by geotechnical inspection;
- Compact the natural subgrade to a minimum dry density ratio of 100% Standard (AS 1289.5.1.1). The compacted clay subgrade should be left exposed for a minimum amount of time prior to placement of pavement layers to minimise the occurrence of desiccation cracking in dry weather, or softening in wet weather;
- If raising of the subgrade level is required, all deleterious materials should be removed. Approved filling should then be placed in layers not exceeding 250 mm loose thickness and compacted to a minimum dry density ratio of 100% Standard at the moisture content described above.

Geotechnical inspections and testing should be undertaken during construction in accordance with AS 3798-2007 (Ref 8).

8. References

1. Douglas Partners Pty Ltd, “Report on Geotechnical Investigation, Hunter Sports High School Upgrade, Pacific Highway, Gateshead”, Report 81598.00.R.002, dated October 2014.
2. Australian Standard AS 2870-2011 "Residential Slabs and Footings", 2011, Standards Australia.
3. Cement Concrete Aggregates Australia, Technical Note 61 “Articulated Walling”.
4. Pells, Douglas, et al “Design Values for Foundations on Sydney Sandstone and Shale”, Australian Geomechanics Society, 1978.
5. Austroads, “Guide to Pavement Technology, Part 2: Pavement Structural Design”, 2012.
6. Australian Road Research Board, Special Report No.41, “Into a New Age of Pavement Design, A Structural Design Guide for Flexible Residential Street Pavements”, dated 1989.
7. Australian Standard AS 1289.5.2.1-2003, “Methods of testing soils for engineering purposes”, Standards Australia.
8. Australian Standard AS 1289.5.1.1-2003, “Methods of testing soils for engineering purposes”, Standards Australia.
9. Australian Standard AS 3798-2007, “Guidelines on Earthworks for Commercial and Residential Developments”, Standards Australia.

9. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for the redevelopment of Hunter Sports High School, located at Pacific Highway, Gateshead in accordance with DP proposal NCL150571 dated 2 September 2015, and acceptance received from Ms Jennifer Bates of NSW Public Works - Department of Finance, Services and Innovation dated 8 September 2015 (Purchase Order 3000156955). The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of NSW Public Works 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 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.

The scope for work for this investigation / report did not include the assessment of surface or subsurface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

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

Appendix A

CSIRO-BTF 18
About this Report
Sampling Methods
Soil Descriptions
Rock Descriptions
Symbols and Abbreviations

Foundation Maintenance and Footing Performance: A Homeowner's Guide

PUBLISHING
BTF 18-2011
replaces
Information
Sheet 10/91

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

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

Soil Types

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

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870-2011, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.

In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

Notes

1. Where controlled fill has been used, the site may be classified A to E according to the type of fill used.
2. Filled sites. Class P is used for sites which include soft fills, such as clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soil subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise.
3. Where deep-seated moisture changes exist on sites at depths of 3 m or greater, further classification is needed for Classes M to E (M-D, H1-D, H2-D and E-D).

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

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

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

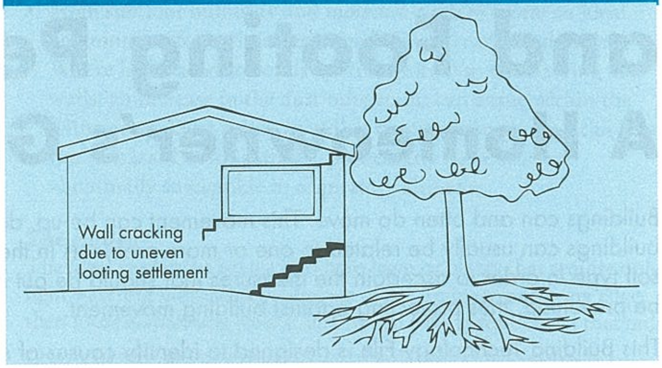
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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

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

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

Trees can cause shrinkage and damage



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

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

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

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

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

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

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

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

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

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

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

Effects on framed structures

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

Effects on brick veneer structures

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870-2011.

AS 2870-2011 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing

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

Ground drainage

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

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

Protection of the building perimeter

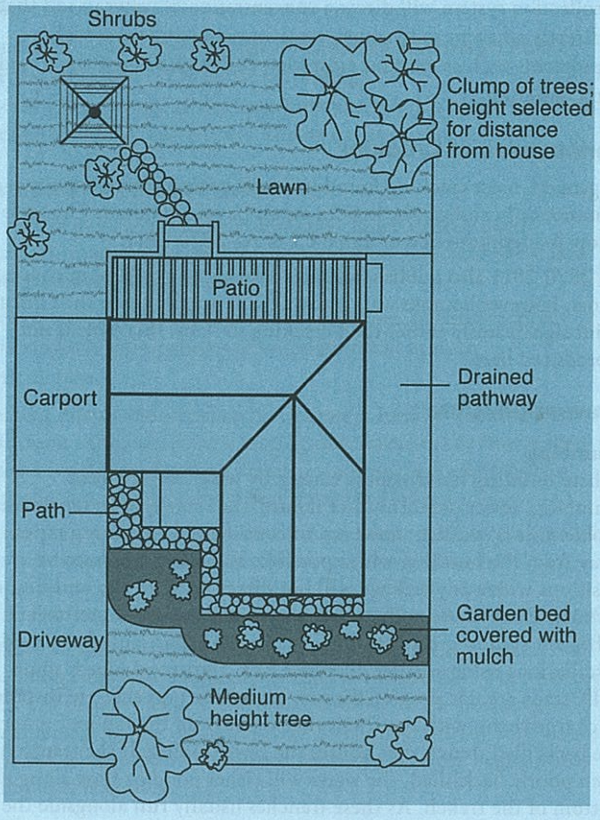
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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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.

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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.

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.

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

July 2010

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

■	Asphalt
░	Road base
░	Concrete
▩	Filling

Soils

▨	Topsoil
▒	Peat
▨	Clay
▨	Silty clay
▨	Sandy clay
▨	Gravelly clay
▨	Shaly clay
▥	Silt
▥	Clayey silt
▥	Sandy silt
░	Sand
░	Clayey sand
░	Silty sand
░	Gravel
░	Sandy gravel
░	Cobbles, boulders
░	Talus

Sedimentary Rocks

░	Boulder conglomerate
░	Conglomerate
░	Conglomeratic sandstone
░	Sandstone
▤	Siltstone
▤	Laminite
▤	Mudstone, claystone, shale
■	Coal
▦	Limestone

Metamorphic Rocks

░	Slate, phyllite, schist
░	Gneiss
░	Quartzite

Igneous Rocks

░	Granite
░	Dolerite, basalt, andesite
░	Dacite, epidote
░	Tuff, breccia
░	Porphyry

Appendix B

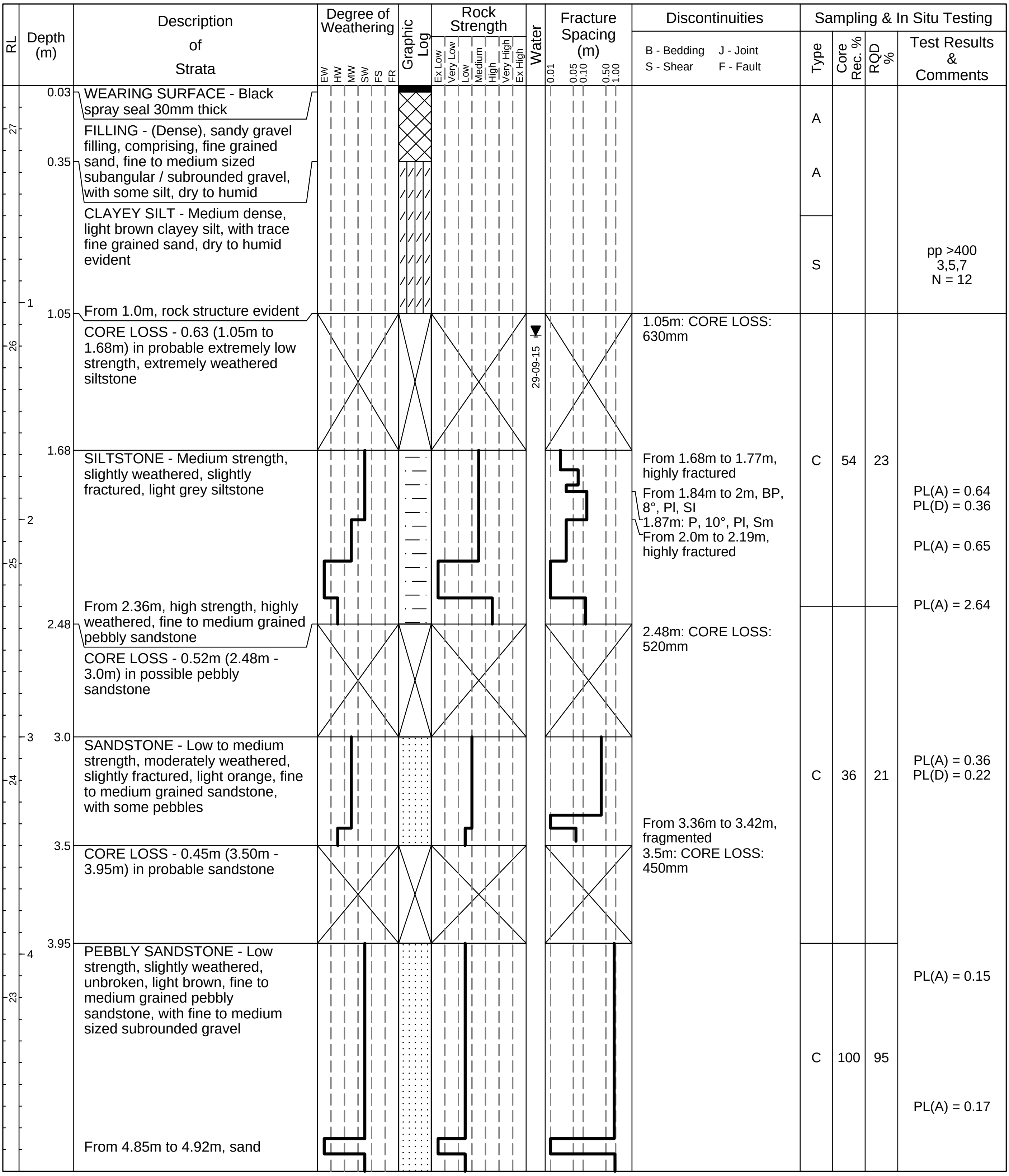
Borehole Logs (Bores 101 to 107)
Core Photoplates (Bores 101 to 104)
Dynamic Penetrometer Test Results

BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 27.2 AHD
EASTING: 377728
NORTHING: 6349519
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 81598.01
DATE: 28/9/2015
SHEET 1 OF 2



Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.03	WEARING SURFACE - Black spray seal 30mm thick			A			
0.35	FILLING - (Dense), sandy gravel filling, comprising, fine grained sand, fine to medium sized subangular / subrounded gravel, with some silt, dry to humid			A			
	CLAYEY SILT - Medium dense, light brown clayey silt, with trace fine grained sand, dry to humid evident			S			pp >400 3,5,7 N = 12
1.05	From 1.0m, rock structure evident						
	CORE LOSS - 0.63 (1.05m to 1.68m) in probable extremely low strength, extremely weathered siltstone	29-09-15	1.05m: CORE LOSS: 630mm	C	54	23	
1.68	SILTSTONE - Medium strength, slightly weathered, slightly fractured, light grey siltstone		From 1.68m to 1.77m, highly fractured From 1.84m to 2m, BP, 8°, Pl, Sl 1.87m: P, 10°, Pl, Sm From 2.0m to 2.19m, highly fractured				PL(A) = 0.64 PL(D) = 0.36 PL(A) = 0.65 PL(A) = 2.64
2.48	From 2.36m, high strength, highly weathered, fine to medium grained pebbly sandstone						
	CORE LOSS - 0.52m (2.48m - 3.0m) in possible pebbly sandstone		2.48m: CORE LOSS: 520mm	C	36	21	
3.0	SANDSTONE - Low to medium strength, moderately weathered, slightly fractured, light orange, fine to medium grained sandstone, with some pebbles		From 3.36m to 3.42m, fragmented				PL(A) = 0.36 PL(D) = 0.22
3.5	CORE LOSS - 0.45m (3.50m - 3.95m) in probable sandstone		3.5m: CORE LOSS: 450mm				
3.95	PEBBLY SANDSTONE - Low strength, slightly weathered, unbroken, light brown, fine to medium grained pebbly sandstone, with fine to medium sized subrounded gravel			C	100	95	PL(A) = 0.15 PL(A) = 0.17
	From 4.85m to 4.92m, sand						

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 1.05m

TYPE OF BORING: Solid flight augering (v-bit) to 1.05m, NMLC coring to 7.10m

WATER OBSERVATIONS: Free groundwater measured at 1.15m, 29/09/2015

REMARKS: 2.70m south-west and 4.80m north-west of kerb of carpark. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

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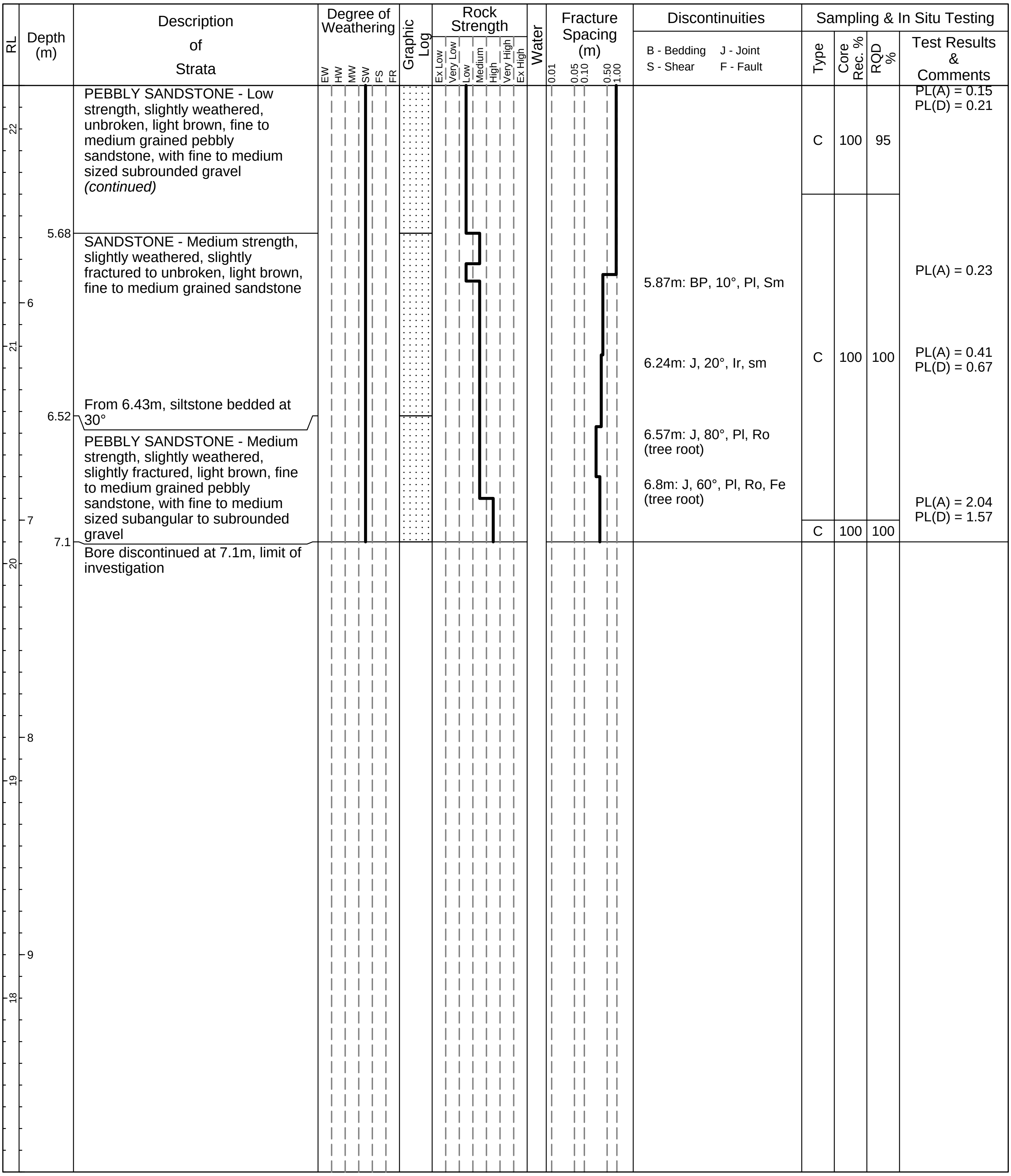
BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 27.2 AHD
EASTING: 377728
NORTHING: 6349519
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 81598.01
DATE: 28/9/2015
SHEET 2 OF 2



Discontinuities: B - Bedding J - Joint S - Shear F - Fault

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	PEBBLY SANDSTONE - Low strength, slightly weathered, unbroken, light brown, fine to medium grained pebbly sandstone, with fine to medium sized subrounded gravel *(continued)*		C	100	95	PL(A) = 0.15 PL(D) = 0.21
5.68	SANDSTONE - Medium strength, slightly weathered, slightly fractured to unbroken, light brown, fine to medium grained sandstone From 6.43m, siltstone bedded at 30°	5.87m: BP, 10°, Pl, Sm 6.24m: J, 20°, Ir, sm	C	100	100	PL(A) = 0.23 PL(A) = 0.41 PL(D) = 0.67
6.52	PEBBLY SANDSTONE - Medium strength, slightly weathered, slightly fractured, light brown, fine to medium grained pebbly sandstone, with fine to medium sized subangular to subrounded gravel	6.57m: J, 80°, Pl, Ro (tree root) 6.8m: J, 60°, Pl, Ro, Fe (tree root)	C	100	100	PL(A) = 2.04 PL(D) = 1.57
7.1	Bore discontinued at 7.1m, limit of investigation					

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 1.05m

TYPE OF BORING: Solid flight augering (v-bit) to 1.05m, NMLC coring to 7.10m

WATER OBSERVATIONS: Free groundwater measured at 1.15m, 29/09/2015

REMARKS: 2.70m south-west and 4.80m north-west of kerb of carpark. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 28.8 AHD
EASTING: 377694
NORTHING: 6349567
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 81598.01
DATE: 29/9/2015
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.03	WEARING SURFACE - Black spray seal, 25mm thick		A			
0.25	FILLING - (very dense), dark grey sandy gravel filling comprising, fine to medium grained sand and medium sized subangular gravel (slag), humid					
	CLAYEY SILT / SILTY CLAY - Stiff to very stiff, light brown silty clay / clayey silt, with some organics (wood), M>Wp					
1	From 1.0m, very stiff		S			pp = 350-400 2,4,5 N = 9
			U			pp = 400
	From 1.70m, some rock structure evident		A			
2.3	CARBONACEOUS SILTY CLAY - Stiff to very stiff, dark brown, carbonaceous silty clay, M>Wp		S			pp = 400 3,4,6 N = 10
	From 3.20m, Very stiff to hard, black, with some rock structure evident (weathered coal)					
4			S			pp = 400-420 3,7,15 N = 22
	From 4.80m, increased drilling resistance					
4.9		From 4.90m to 5.22m,	C	100	97	PL(A) = 0.04

RL: 28, 27, 26, 25, 24

Degree of Weathering: EW HW MW SW FS FR

Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High

Fracture Spacing (m): 0.01, 0.05, 0.10, 0.50, 1.00

B - Bedding J - Joint
S - Shear F - Fault

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 4.90m
TYPE OF BORING: Solid flight augering (tc-bit) to 0.25m, solid flight auger (v-bit) to 4.80m, NMLC coring to 6.20m
WATER OBSERVATIONS: Free groundwater obscured due to drilling methods
REMARKS: 2.5m south-west of B Block. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 28.8 AHD
EASTING: 377694
NORTHING: 6349567
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 81598.01
DATE: 29/9/2015
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing: Type	Core Rec. %	RQD %	Test Results & Comments
		COAL - Very low strength, fresh, fractured, black coal *(continued)*						fractured coal (ti)				
	5.22	SILTSTONE - Extremely low strength, fresh, slightly fractured, grey siltstone interbedded, with lenses of coal up to 5mm thick						5.28m: P, 5°, Un, Sm				PL(A) = 0.03
23	5.44 6	LAMINITE - Low strength, fresh, slightly fractured, light grey, fine grained sandstone interbedded, with siltstone bands up to 20mm thick, with trace lenses of coal From 5.90m, medium strength						5.76m: P, 2°, Pl, Si, Cn	C	100	97	PL(A) = 0.12 PL(D) = 0.22 PL(A) = 0.55 PL(D) = 0.61
	6.2	Bore discontinued at 6.2m, limit of investigation										
22												
	7											
21												
	8											
20												
	9											
19												

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 4.90m

TYPE OF BORING: Solid flight augering (tc-bit) to 0.25m, solid flight auger (v-bit) to 4.80m, NMLC coring to 6.20m

WATER OBSERVATIONS: Free groundwater obscured due to drilling methods

REMARKS: 2.5m south-west of B Block. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.5 AHD
EASTING: 377610
NORTHING: 6349577
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 81598.01
DATE: 28 - 30/9/2015
SHEET 1 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing Type	Core Rec. %	RQD %	Test Results & Comments
		SAND - Medium dense, brown, fine to medium grained sand, with some clay, damp							A			
	0.2	CLAY - Very stiff, brown clay, with some fine grained sand, M>Wp										
25									U_{50}			pp >380-400
		From 0.80m, orange, no sand							A			
	1											
									S			pp = 370 2,4,5 N = 9
24												
	2											
23												
									S			pp >400 4,9,14 N = 23
						29-09-15						
	3											
22	3.5	SILT - Hard, pale grey mottled yellow / light orange silt, with some fine to medium grained sand and trace clay, with some rock structure evident										
	4											
									S			pp = 550 10,20/130mm refusal
21												
		From 4.60m, wet							A			

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 5.80m
TYPE OF BORING: Solid flight augering (v-bit) to 5.50m, solid flight auger (v-bit) to 5.80m, NMLC coring to 13.20m
WATER OBSERVATIONS: Free groundwater measured at 2.90m on 29/09/2015
REMARKS: 7.30m north-west and 4.50m south-west of L Block. Surface level interpolated from plan provided by client.

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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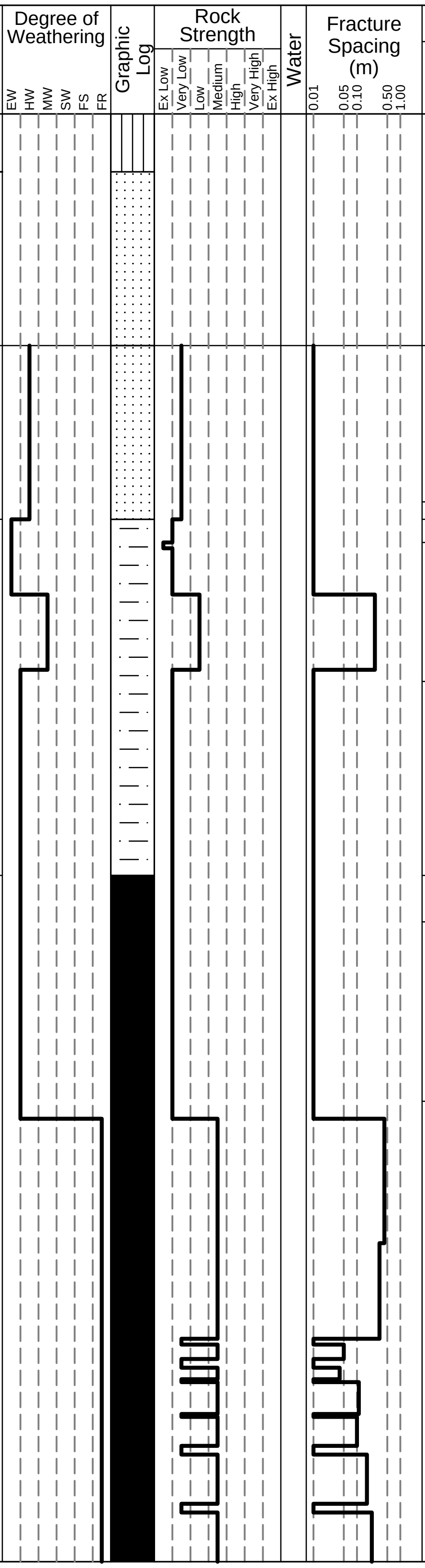
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.5 AHD
EASTING: 377610
NORTHING: 6349577
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 81598.01
DATE: 28 - 30/9/2015
SHEET 2 OF 3



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
20	5.2	SANDSTONE - Extremely low strength, extremely weathered, friable, red, fine to medium grained sandstone, with soil like properties		S			29/130mm refusal
	5.8 6	SANDSTONE - Very low strength, highly weathered, friable, light grey and orange, fine to medium grained sandstone	6.2m: J, SV, un, S, Cn 6.28m: P, 3°, Un, Sm, clay infill, 3mm thick 6.34m: P, 5°, cu, Sm clay infill 3mm thick				PL(A) = 0.06 PL(D) = 0.03
19 18	6.4 7	SILTSTONE - Extremely low to very low strength, extremely weathered, friable, light grey and orange siltstone From 6.66m to 6.90m, low strength, moderately weathered, orange siltstone, with ironstained,healed micro-fracturing From 6.92m, extremely low to very low strength, highly weathered to extremely weathered	From 6.40m to 6.66m, J, SV, Un, Sm, cn 6.48m: P, 2°, Pl SI, clay infill 2m - 20mm thick From 6.92m to 7.30m, J, SV, Pl, Sm, clay infill 40mm thick 6.96m: J, 30°, Pl, SI, Cn 7.35m: P, 10°, Pl, Sm, Cn 7.53m: J, 50°, pl, Sm, Cn	C	100	19	PL(A) = 0.13 PL(A) = 0.02 PL(A) = 0.01
17 16	7.63 8 9	COAL - Extremely low to very low strength, highly weathered, black coal, with bands of siltstone up to 70mm thick From 7.81m to 7.84m, siltstone band From 7.99m to 8.02m, siltstone band From 8.40m to 8.47m, siltstone band From 8.47m, medium strength, fresh From 9.23m, very low strength, siltstone bands at 50mm to 200mm spacings	7.63m: BP, 2°, Pl, Sm, Cn 7.79m: P, SH, Pl, Sm 8.08m: P, SH, Pl, Sm 8.37m: J, 35°, Pl, Sm 8.41m: P, 3°, Un, Sm 8.9m: J, SV, Pl, Sm, Cn 9.81m: P, 20°, Pl, Sm, clay infill 30mm thick	C C	100 100	18 83	PL(A) = 0.04 PL(D) = 0.01 PL(A) = 0.53 PL(A) = 0.65

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 5.80m

TYPE OF BORING: Solid flight augering (v-bit) to 5.50m, solid flight auger (v-bit) to 5.80m, NMLC coring to 13.20m

WATER OBSERVATIONS: Free groundwater measured at 2.90m on 29/09/2015

REMARKS: 7.30m north-west and 4.50m south-west of L Block. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

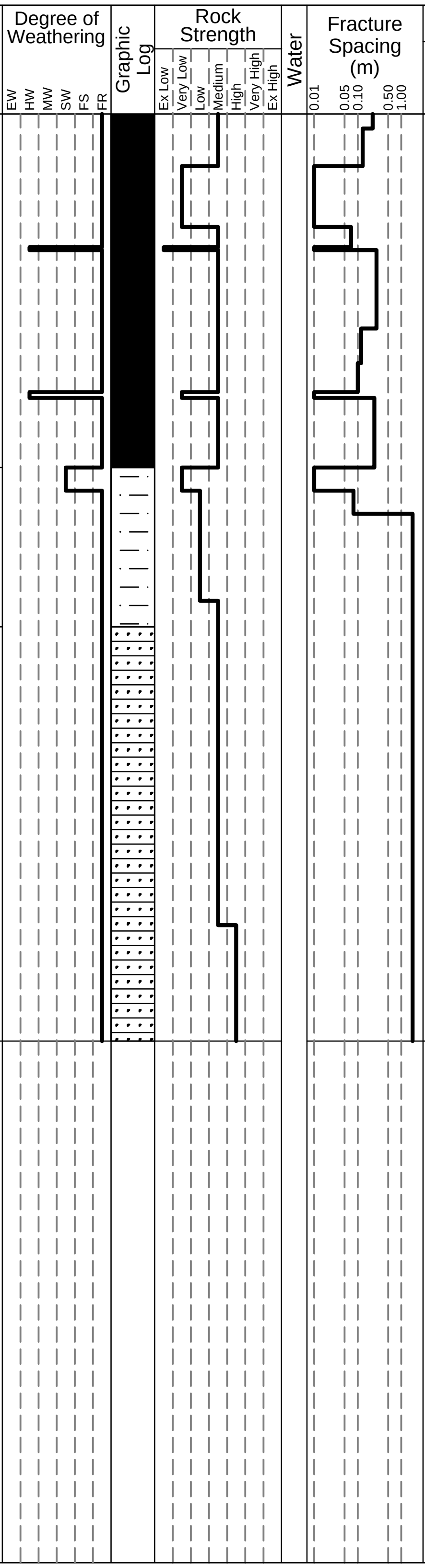
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.5 AHD
EASTING: 377610
NORTHING: 6349577
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 81598.01
DATE: 28 - 30/9/2015
SHEET 3 OF 3



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		COAL - Extremely low to very low strength, highly weathered, black coal, with bands of siltstone up to 70mm thick *(continued)* From 10.18m to 10.39m, very low strength, tuff	10.05m: J, 70° - 90°, Ir, Ro	C	100	83	
15			10.24m: P, 5°, Pl, Sl, Cn From 10.46m to 10.74m, J, 85°, Pl, Sm, he	C	100	68	PL(A) = 0.03
	11	From 10.89m to 10.94m, medium strength siltstone	10.86m: P, 3°, Pl, Sm, Cn				PL(A) = 0.61
14	11.22	SILTSTONE - Very low strength, slightly weathered, slightly fractured, grey siltstone, with bands of coal up to 10mm thick From 11.30m, low strength, fresh From 11.68m, medium strength	11.23m: P, 5°, Pl, Sm, Cn 11.38m: J, 40°, Un, Sm				PL(A) = 0.11
	11.77	LAMINITE - Medium to high strength, fresh, unbroken, grey, fine to medium grained sandstone, interbedded with bands of siltstone up to 100mm thick		C	100	100	PL(A) = 0.78 PL(D) = 1.03
	12						PL(A) = 0.84 PL(D) = 1
13							
	13						PL(D) = 2.42
							PL(A) = 1.25 PL(D) = 1.15
	13.2	Bore discontinued at 13.2m, limit of investigation					
12							
	14						
11							

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 5.80m

TYPE OF BORING: Solid flight augering (v-bit) to 5.50m, solid flight auger (v-bit) to 5.80m, NMLC coring to 13.20m

WATER OBSERVATIONS: Free groundwater measured at 2.90m on 29/09/2015

REMARKS: 7.30m north-west and 4.50m south-west of L Block. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.5 AHD
EASTING: 377648
NORTHING: 6349532
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 81598.01
DATE: 29/9/2015
SHEET 1 OF 2

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
Fracture Spacing (m): 0.01, 0.05, 0.10, 0.50, 1.00
Discontinuities: B - Bedding, J - Joint, S - Shear, F - Fault

RL	Depth (m)	Description of Strata	Water	Type	Core Rec. %	RQD %	Test Results & Comments
		SAND - Medium dense, brown, fine to medium grained sand, with some fine sized subangular / subrounded gravel and trace clay, humid		A			
	0.3	SILTY CLAY - Stiff, brown silty clay, M>Wp					
25							
		From 0.80m, light brown mottled light grey					
	1			S			pp = 110-150 1,2,2 N = 4
24							
	2.0	SILT - Very stiff to hard, light grey mottled orange silt, with some fine to medium grained sandy silt bands, M<Wp					
23		From 2.50m, some rock structure evident		S			pp >400 7,10,14 N = 24
	3						
22							
	3.85	CARBONACEOUS SILTY CLAY - Stiff to very stiff, dark brown to black carbonaceous silty clay (weathered coal)					
	4			S			pp = 200-300 4,5,6 N = 11
21							

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 5.70m

TYPE OF BORING: Solid flight augering (v-bit) to 5.76m, NMLC coring to 8.31m

WATER OBSERVATIONS: Free groundwater obscured due to drilling methods

REMARKS: 25% water loss from 5.76m, 8m north west of G block, 0.5m north west of concrete slab / driveway. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

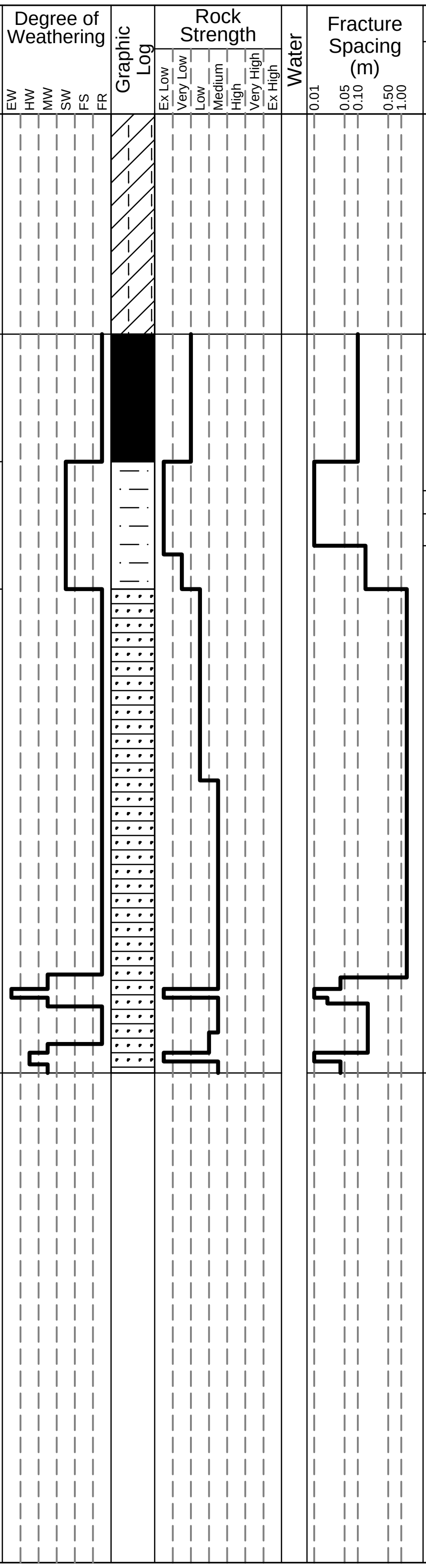
Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.5 AHD
EASTING: 377648
NORTHING: 6349532
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 81598.01
DATE: 29/9/2015
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
20		CARBONACEOUS SILTY CLAY - Stiff to very stiff, dark brown to black carbonaceous silty clay (weathered coal) *(continued)* From 5.50m, black coal		S			16,20/110mm,-refusal
	5.76 6	COAL - Very low to low strength, fresh, highly fractured, black coal	5.87m: J, 20°, Ir, Ro 5.97m: J, 10°, Ir, Ro	C	100	65	PL(A) = 0.03
19	6.2	SILTSTONE - Extremely low strength, slightly weathered, fractured, grey siltstone, with coal bands and lenses from 1mm to 25mm thick From 6.52m, very low strength	6.24m: BP, 2°, Pl, ti, coal 6.3m: BP, 10°, P, ti, coal 6.38m: BP, 5°, Pl, ti, coal 6.49m: P, 7°, Pl, SL, Cn				PL(A) = 0.01 PL(A) = 0.06 PL(D) = 0.04
18	6.64 7	LAMINITE - Low strength, fresh, unbroken, fine to medium grained sandstone interbedded, with bands of siltstone up to 25mm thick					PL(A) = 0.26 PL(D) = 0.31
		From 7.30m, medium strength		C	100	86	PL(A) = 0.36 PL(A) = 0.61
	8	From 7.97m, moderately weathered, with bands of extremely low strength, extremely weathered, siltstone	7.98m: P, 3°, Pl, Sl 8.07m: P, 3°, Pl, Sl, Cn 8.24m: P, 3°, Ir, Sm, Cn				PL(A) = 0.42 PL(A) = 0.32
17 16	8.31 9	Bore discontinued at 8.31m, limit of investigation					

RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** HQ to 5.70m

TYPE OF BORING: Solid flight augering (v-bit) to 5.76m, NMLC coring to 8.31m

WATER OBSERVATIONS: Free groundwater obscured due to drilling methods

REMARKS: 25% water loss from 5.76m, 8m north west of G block, 0.5m north west of concrete slab / driveway. Surface level interpolated from plan provided by client.

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

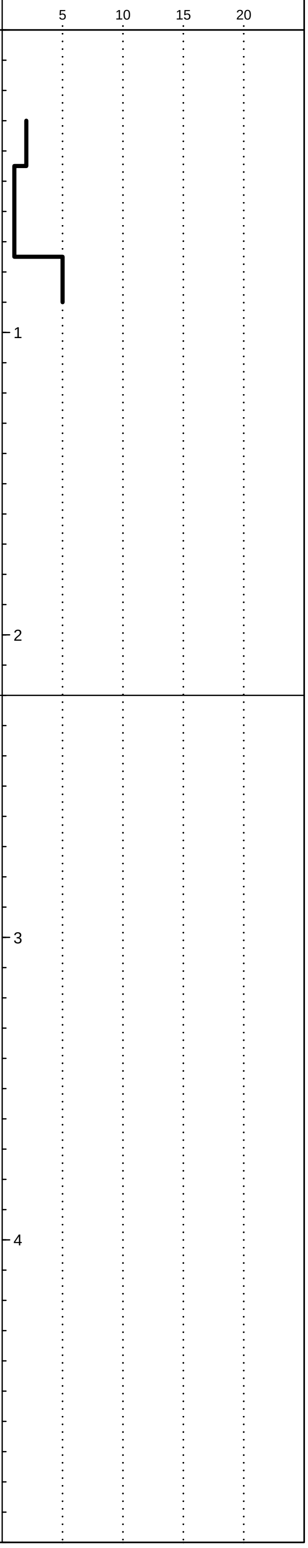
CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 28.6 AHD
EASTING: 377675
NORTHING: 6349559
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 81598.01
DATE: 29/9/2015
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Sampling & In Situ Testing Type	Depth	Sample	Results & Comments	Water
	0.03	WEARING SURFACE - Black spray seal, 25mm thick					
	0.3	FILLING - (Dense), brown, sandy gravel filling, comprising fine to medium grained sand and coarse sized subangular gravel (slag), humid	A	0.2			
28		FILLING - Generally comprising brown silty clay filling, with some bands of fine to medium grained sand, M>Wp					
	1		S	1.0 1.45		2,3,2 N = 5	
27	1.6	CLAYEY SILT / SILTY CLAY - Very stiff, grey mottled orange silty clay / clayey silt, M>Wp	U_{50}	1.7 1.91		pp = 300-350	
	1.9	SILTSTONE - Extremely low strength, extremely weathered, friable, grey and orange siltstone, soil like properties to 2.10m, v-bit refusal at 2.10m					
	2.2	Bore discontinued at 2.2m, refusal	A	2.2			
26							
	3						
25							
	4						
24							

Dynamic Penetrometer Test (blows per 150mm)



RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** Nil

TYPE OF BORING: Solid flight augering (v-bit) to 2.10m, solid flight auger (tc-bit) to 2.20m

WATER OBSERVATIONS: No free groundwater observed

REMARKS: 3.10m south - west and 2.20m north - west of retaining wall. Surface level interpolated from plan provided by client.

☐ 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

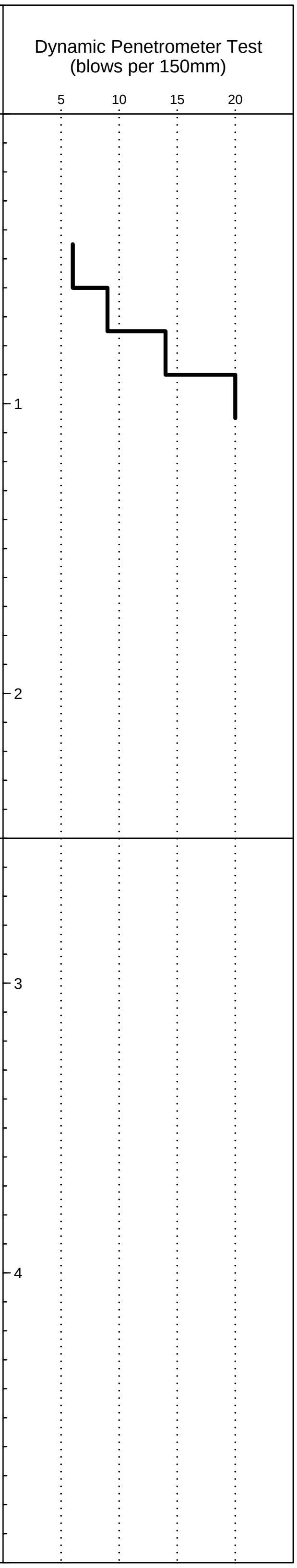
BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

SURFACE LEVEL: 25.8 AHD
EASTING: 377695
NORTHING: 6349509
DIP/AZIMUTH: 90°/--

BORE No: 106
PROJECT No: 81598.01
DATE: 30/9/2015
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water
0.03	WEARING SURFACE - Black spray seal, 25mm thick					
	FILLING - Brown, sandy gravel filling, comprising fine to medium grained sand and fine to medium sized subrounded gravel, moist	A	0.2			
0.4	SILTY CLAY - Very stiff - hard, light grey mottled brown silty clay, M>Wp	A	0.4		pp = 250-300	
0.85	CLAYEY SILT - Hard, light grey mottled orange clayey silt, M<Wp From 1.0m, some rock structure evident	S	1.0		pp >600 6,25/150mm,- refusal	
1.25	SILTSTONE - (Extremely low to very low strength), highly weathered, friable, light grey siltstone, with soil like properties v-bit refusal at 1.35m	A	1.3			
1.35	SILTSTONE - (Low strength), highly weathered, red siltstone From 1.50m, 20mm extremely low strength, extremely weathered (decreased resistance)	A A	1.4 1.55			
	From 2.10m, Low to medium strength, (slow progress)	A	2.1 2.4			
2.5	Bore discontinued at 2.5m, refusal	A	2.5			



RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** Nil

TYPE OF BORING: Solid flight augering (v-bit) to 1.35m, solid flight auger (tc-bit) to 2.50m

WATER OBSERVATIONS: No free groundwater observed

REMARKS: 2.35m south-east of edge of formation of carpark. Surface level interpolated from plan provided by client.

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

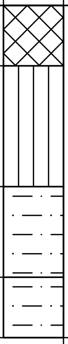
BOREHOLE LOG

CLIENT: NSW Public Works
PROJECT: Proposed Development
LOCATION: Hunter Sports High School, Gateshead

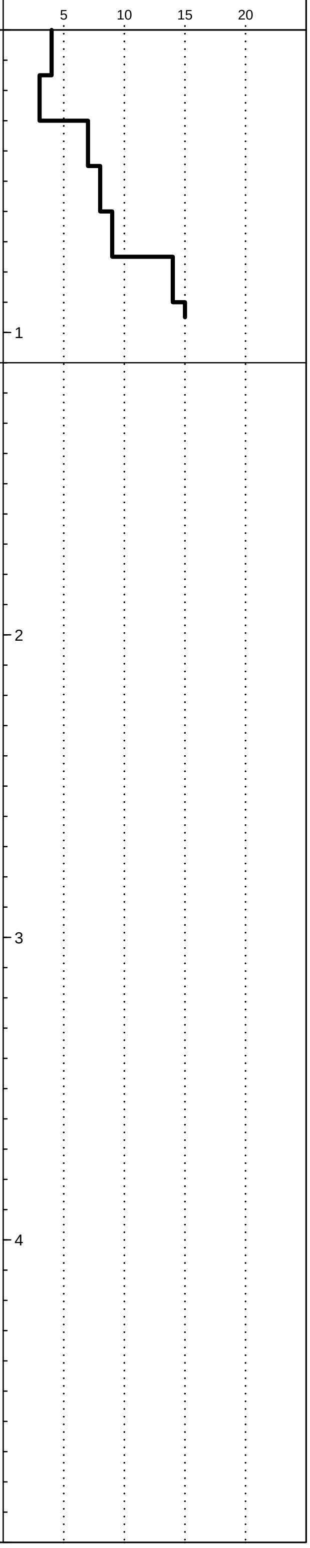
SURFACE LEVEL: 25.9 AHD
EASTING: 377668
NORTHING: 6349544
DIP/AZIMUTH: 90°/--

BORE No: 107
PROJECT No: 81598.01
DATE: 30/9/2015
SHEET 1 OF 1

Depth (m)	Description of Strata	Sampling & In Situ Testing Type	Depth	Sample	Results & Comments	Water
0.2	FILLING - Generally comprising dark brown, clayey sand filling, with some silt, siltstone and plastic (old pipe)	A	0.05			
0.6	SILT - Very stiff, light brown mottled grey silt, with some clay and trace fine grained sand, M ≪Wp	A	0.4			
0.9	SILTSTONE - Extremely low to very low strength, highly weathered, red siltstone From 0.60m to 0.70m, soil like properties, v-bit refusal at 0.70m	A	0.7			
1.1	SILTSTONE - (Low to medium strength), highly weathered, red and purple siltstone From 1.0m, (medium to high strength), very slow progress	A A	1.0 1.1			
	Bore discontinued at 1.1m, refusal					



Dynamic Penetrometer Test (blows per 150mm)



RIG: FG102 **DRILLER:** Fico **LOGGED:** Fulham **CASING:** Nil
TYPE OF BORING: Solid flight augering (v-bit) to 0.70m, solid flight auger (tc-bit) to 1.1m
WATER OBSERVATIONS: No free groundwater observed
REMARKS: 0.85m north west and 0.90m south west of existing concrete slab, 2.30m from retaining wall. Surface level interpolated from plan provided by client.

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

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

HUNTER SPORTS HIGH SCHOOL

BORE BH101 PROJECT 81598.01 2015

1.05 m to 7.10 m

Douglas Partners Geotechnics / Environment / Groundwater	**Core Photoplates**	PROJECT:	81598.01
	Proposed Development	PLATE No:	1
	Hunter Sports High School, Pacific Highway Gateshead	REV:	A
	CLIENT: NSW Public Works	DATE:	9-Oct-15

DOUGLAS PARTNERS PTY LTD

HUNTER SPORTS HIGH SCHOOL

BORE BH102 PROJECT 81598.01 2015

4.90 m to 6.20 m

Douglas Partners Geotechnics / Environment / Groundwater	**Core Photoplates**	PROJECT:	81598.01
	Proposed Development	PLATE No:	2
	Hunter Sports High School, Pacific Highway Gateshead	REV:	A
	CLIENT: NSW Public Works	DATE:	9-Oct-15

DOUGLAS PARTNERS PTY LTD

HUNTER SPORTS HIGH SCHOOL

BORE BH103 PROJECT 81598.01 2015

5.80 m to 10.0 m

DOUGLAS PARTNERS PTY LTD

HUNTER SPORTS HIGH SCHOOL

BORE BH103 PROJECT 81598.01 2015

10.0 m to 13.2 m

Douglas Partners
Geotechnics / Environment / Groundwater

Core Photoplates

Proposed Development

Hunter Sports High School,
Pacific Highway Gateshead

CLIENT: NSW Public Works

PROJECT:	81598.01
PLATE No:	3
REV:	A
DATE:	9-Oct-15

DOUGLAS PARTNERS PTY LTD

HUNTER SPORTS HIGH SCHOOL

BORE BH104 PROJECT 81598.01 2015

5.76 m to 8.30 m

Douglas Partners
Geotechnics / Environment / Groundwater

Core Photoplates

Proposed Development

**Hunter Sports High School,
Pacific Highway Gateshead**

CLIENT: NSW Public Works

PROJECT: 81598.01
PLATE No: 4
REV: A
DATE: 9-Oct-15

Douglas Partners
Geotechnics / Environment / Groundwater

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PO Box 324
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Fax (02) 4960 9601

Results of Dynamic Penetrometer Tests

Client NSW Public Works
Project Proposed Development
Location Hunter Sports High School, Gateshead

Project No. 81598.01
Date 1/10/2015
Page No. 1 of 1

Test Locations	101	102	103	104	105	106	107			
RL of Test (AHD)										
Depth (m)	**Penetration Resistance** Blows/150 mm									
0.00 – 0.15	-	-	3	6	-	-	4			
0.15 – 0.30	-	-	4	7	-	-	3			
0.30 – 0.45	-	5	4	4	-	-	7			
0.45 – 0.60	-	3	5	4	2	6	8			
0.60 – 0.75	6	6	6	6	1	9	9			
0.75 – 0.90	7	13	4	8	1	14	14			
0.90 – 1.05	9	20	7	11	5	20	15/150			
1.05 – 1.20	bouncing									
1.20 – 1.35										
1.35 – 1.50										
1.50 – 1.65										
1.65 – 1.80										
1.80 – 1.95										
1.95 – 2.10										
2.10 – 2.25										
2.25 – 2.40										
2.40 – 2.55										
2.55 – 2.70										
2.70 – 2.85										
2.85 – 3.00										
3.00 – 3.15										
3.15 – 3.30										
3.30 – 3.45										
3.45 – 3.60										

Test Method AS 1289.6.3.2, Cone Penetrometer ☑
AS 1289.6.3.3, Sand Penetrometer ☐

Tested By KMF
Checked By TAC

Remarks Ref = Refusal, 25/110 indicates 25 blows for 110 mm penetration

Appendix C

Laboratory Test Results

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
15 Callistemon Close
Warabrook NSW 2304
PO Box 324
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Phone (02) 4960 9600
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Result of Shrink-Swell Index Determination

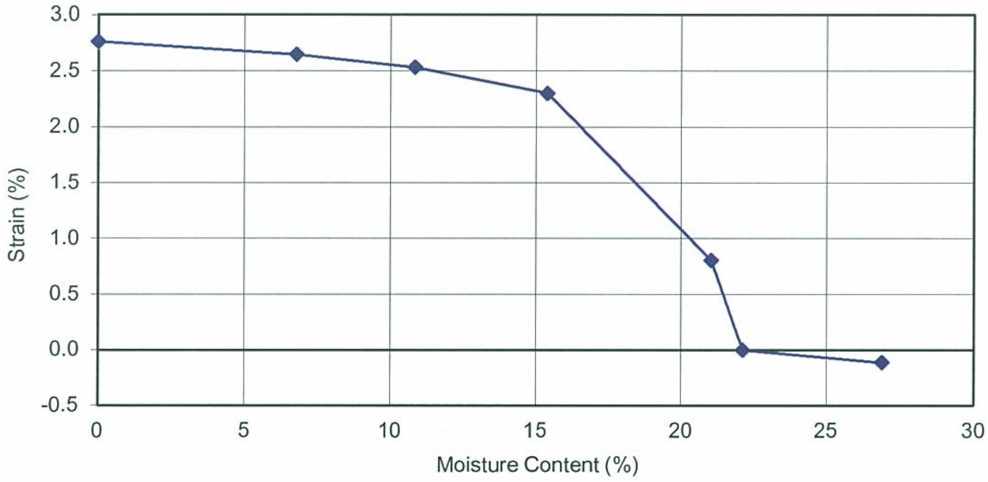
Client :	NSW Public Works	**Project No. :**	81598.01
		Report No. :	N15-193_1
Project :	Proposed Development	**Report Date :**	09.10.2015
		Date Sampled :	28-30.09.15
Location :	Gateshead	**Date of Test:**	06.10.2015
Test Location :	Bore 103		
Depth / Layer :	0.35 - 0.80m	**Page:**	1 of 1

CORE SHRINKAGE TEST

Shrinkage - air dried	2.6 %
Shrinkage - oven dried	2.8 %
Significant inert inclusions	0.0 %
Extent of cracking	MC
Extent of soil crumbling	0.0 %
Moisture content of core	22.1 %

SWELL TEST

Pocket penetrometer reading at initial moisture content	180 kPa
Pocket penetrometer reading at final moisture content	140 kPa
Initial Moisture Content	22.6 %
Final Moisture Content	26.9 %
Swell under 25kPa	0.1 %



SHRINK-SWELL INDEX Iss 1.6% per Δ pF

Description: Sandy CLAY - Brown/grey mottled orange

Test Method(s): AS 1289.7.1.1, AS 1289.2.1.1

Sampling Method(s): Sampled by Newcastle Engineering Department

Extent of Cracking:

- **UC** - Uncracked
- **SC** - Slightly cracked
- **MC** - Moderately cracked
- **HC** - Highly cracked
- **FR** - Fractured

Remarks:

Note that NATA accreditation does not cover the performance of pocket penetrometer readings

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory Number: 828

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

Tested: DM
Checked: DM

Dave Millard
Laboratory Manager

Douglas Partners
Geotechnics / Environment / Groundwater

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Result of Shrink-Swell Index Determination

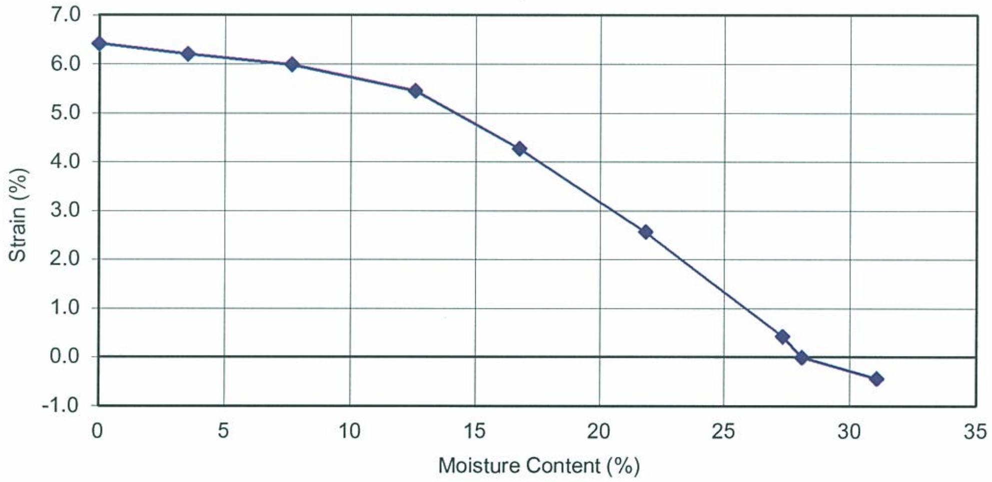
Client :	NSW Public Works	**Project No. :**	81598.01
		Report No. :	N15-193_2
Project :	Proposed Development	**Report Date :**	09.10.2015
		Date Sampled :	28-30.09.15
Location :	Gateshead	**Date of Test:**	06.10.2015
Test Location :	Bore 105		
Depth / Layer :	1.70 - 1.81m	**Page:**	1 of 1

CORE SHRINKAGE TEST

Shrinkage - air dried	6.2 %
Shrinkage - oven dried	6.4 %
Significant inert inclusions	0.0 %
Extent of cracking	SC
Extent of soil crumbling	0.0 %
Moisture content of core	28.1 %

SWELL TEST

Pocket penetrometer reading at initial moisture content	240 kPa
Pocket penetrometer reading at final moisture content	140 kPa
Initial Moisture Content	27.0 %
Final Moisture Content	31.1 %
Swell under 25kPa	0.4 %



SHRINK-SWELL INDEX Iss 3.7% per Δ pF

Description: Clayey SILT/Silty CLAY - Grey mottled orange

Test Method(s): AS 1289.7.1.1, AS 1289.2.1.1

Sampling Method(s): Sampled by Newcastle Engineering Department

Extent of Cracking:

- **UC** - Uncracked
- **SC** - Slightly cracked
- **MC** - Moderately cracked
- **HC** - Highly cracked
- **FR** - Fractured

Remarks:

Note that NATA accreditation does not cover
the performance of pocket penetrometer readings

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory Number: 828

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

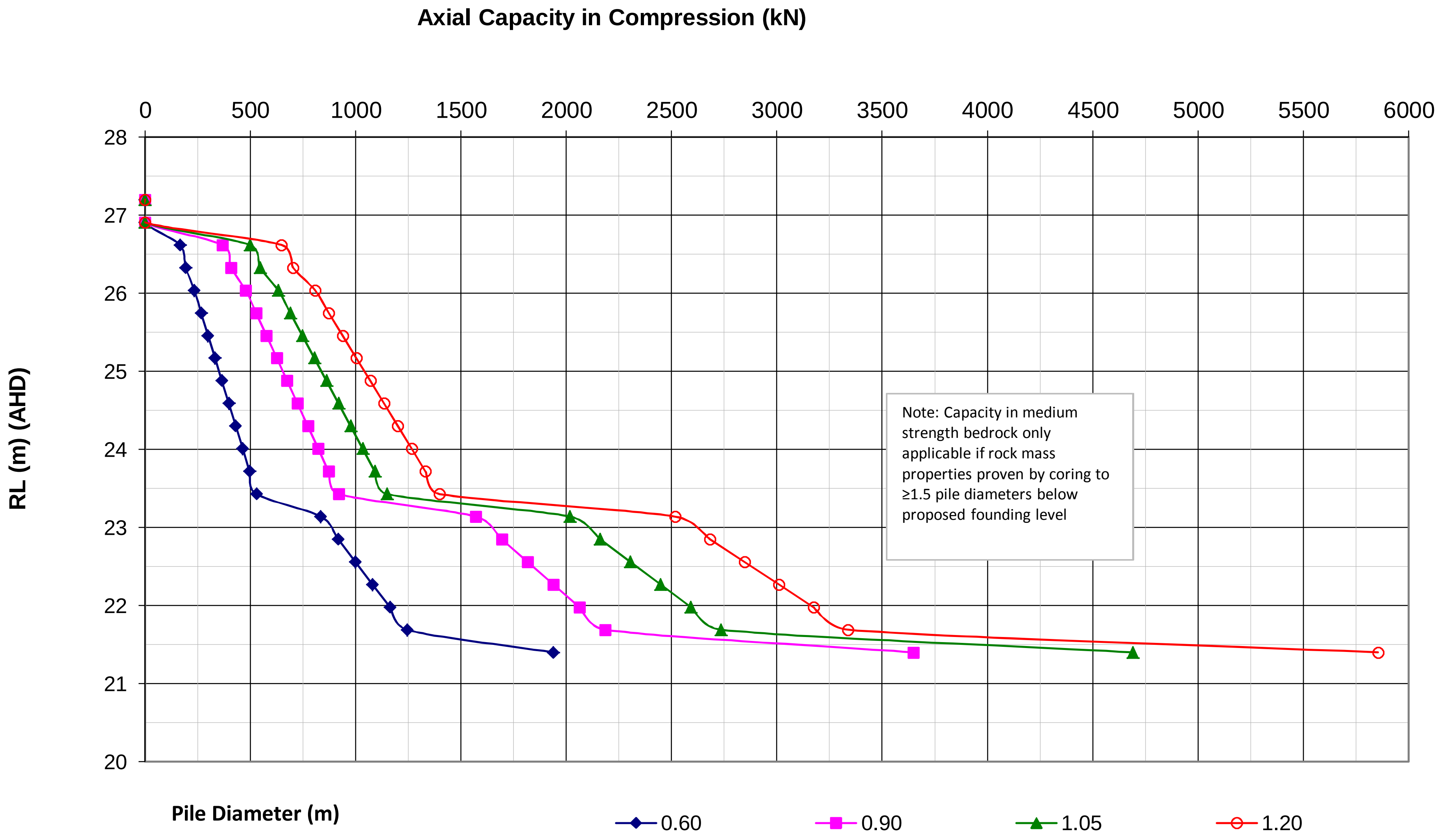
Tested: DM
Checked: DM

Dave Millard
Laboratory Manager

Appendix D

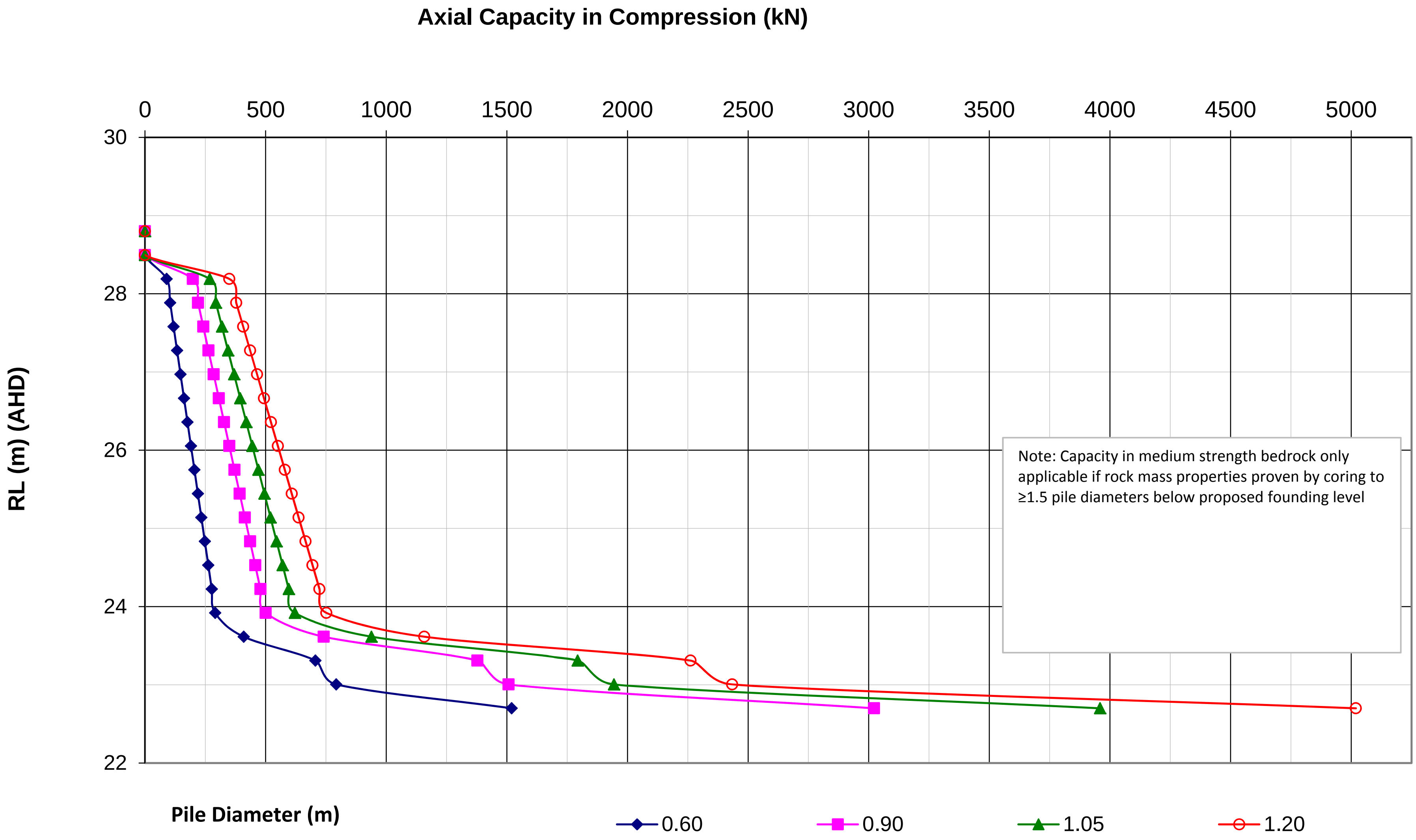
Pile Capacity Estimate Charts

Bore 101 - Allowable Axial Capacity of Bored / CFA Piles



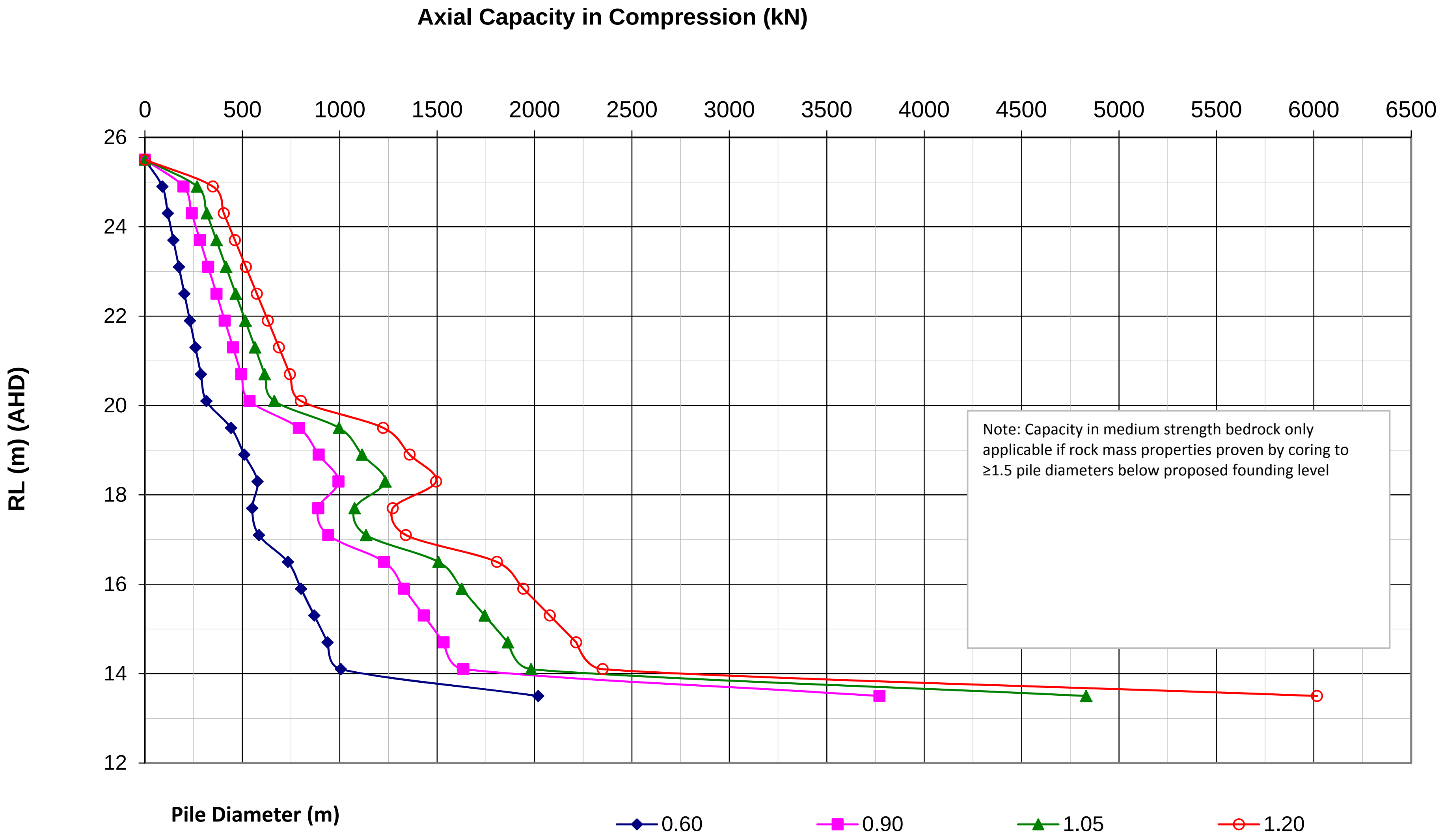
Note: Capacity in medium strength bedrock only applicable if rock mass properties proven by coring to ≥1.5 pile diameters below proposed founding level

Bore 102 - Allowable Axial Capacity of Bored / CFA Piles



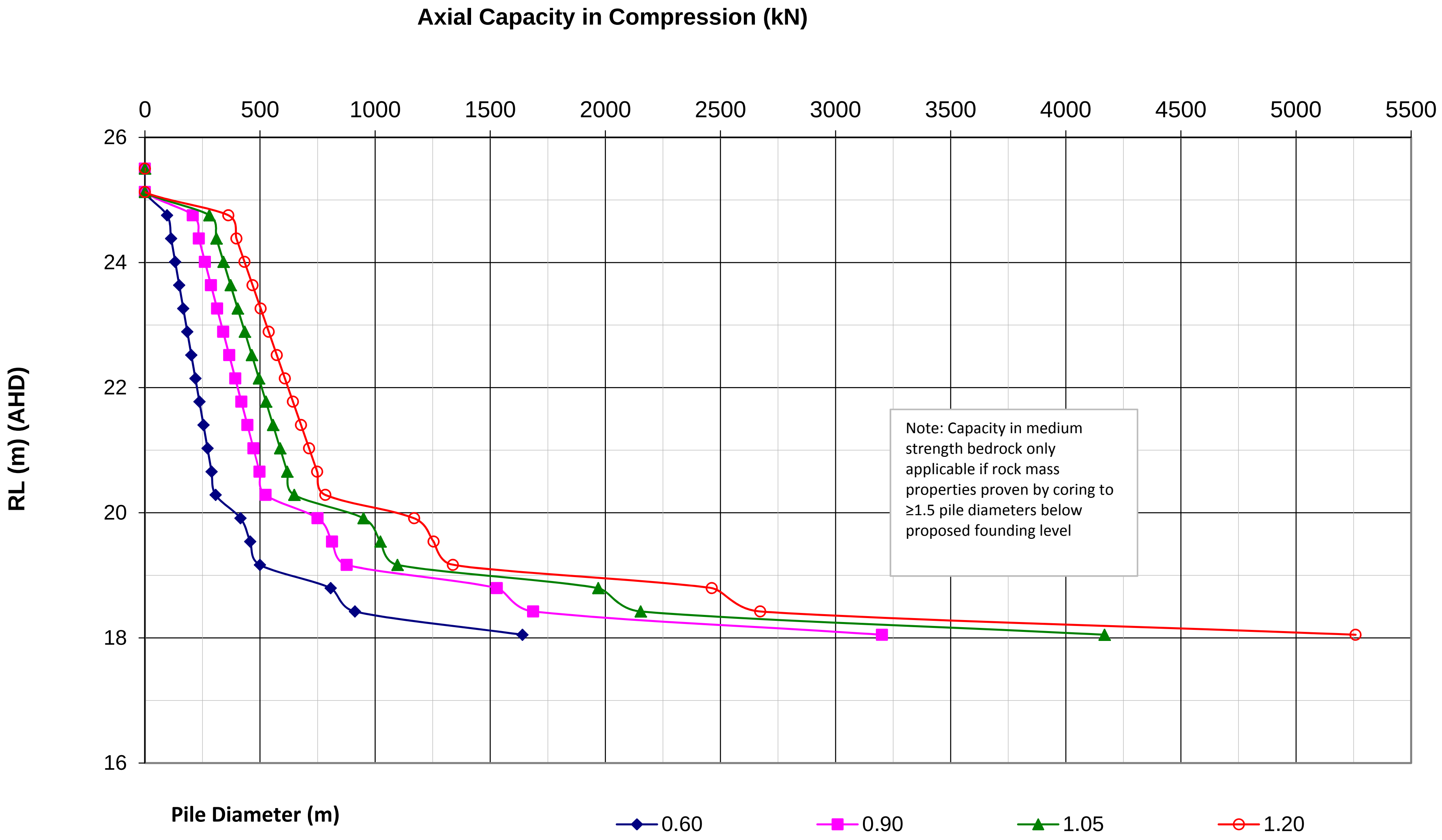
Note: Capacity in medium strength bedrock only applicable if rock mass properties proven by coring to ≥1.5 pile diameters below proposed founding level

Bore 103 - Allowable Axial Capacity of Bored / CFA Piles



Note: Capacity in medium strength bedrock only applicable if rock mass properties proven by coring to ≥1.5 pile diameters below proposed founding level

Bore 104 - Allowable Axial Capacity of Bored / CFA Piles

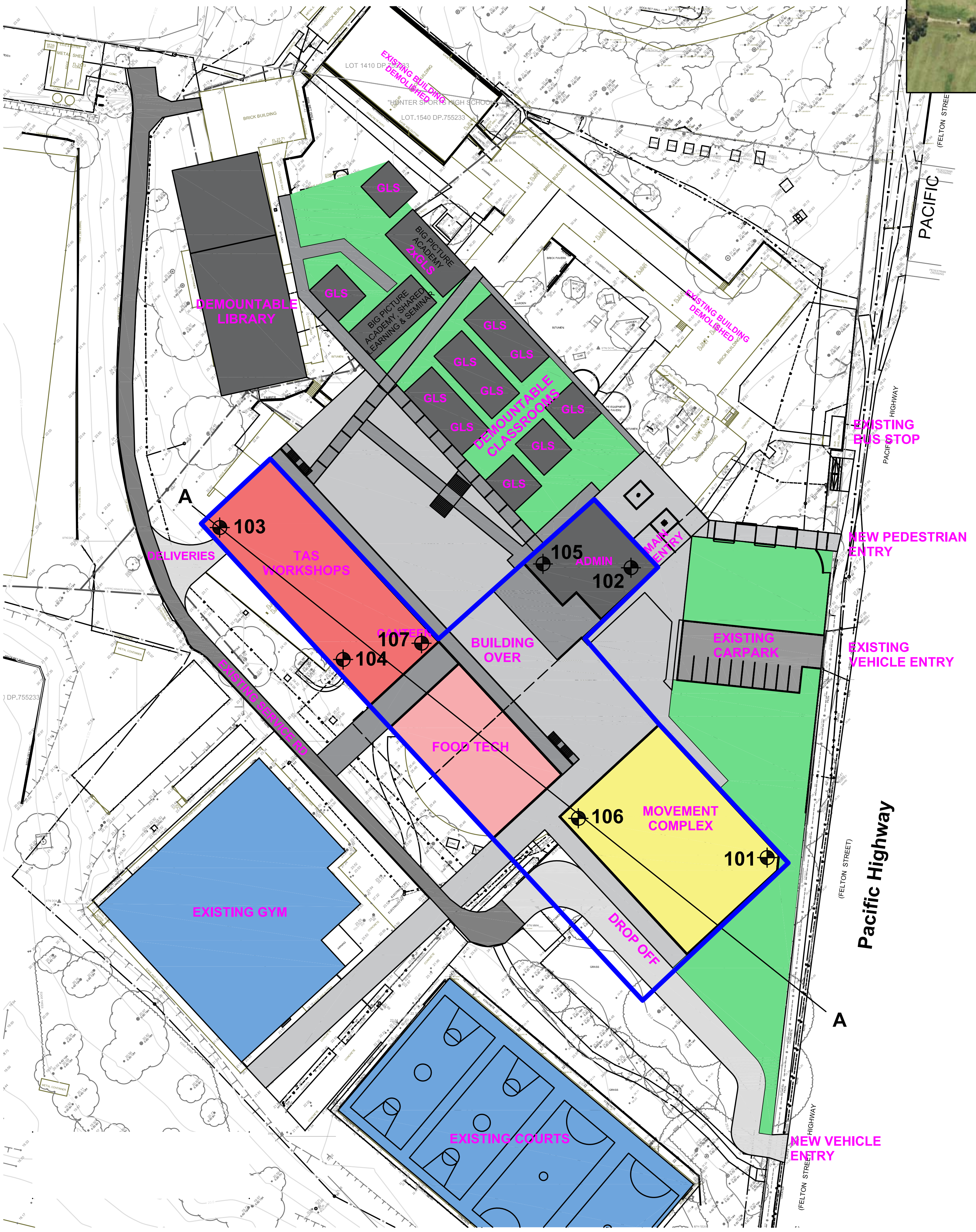


Note: Capacity in medium strength bedrock only applicable if rock mass properties proven by coring to ≥1.5 pile diameters below proposed founding level

Appendix E

Drawing 1 – Test Location Plan
Drawing 2 – Section A-A’

Locality Plan



NOTE: Base drawing from EJE Architecture, Project No 11029
SK A## RevA dated October 2015

0 10 20 30 40 60 80 100 150 200m

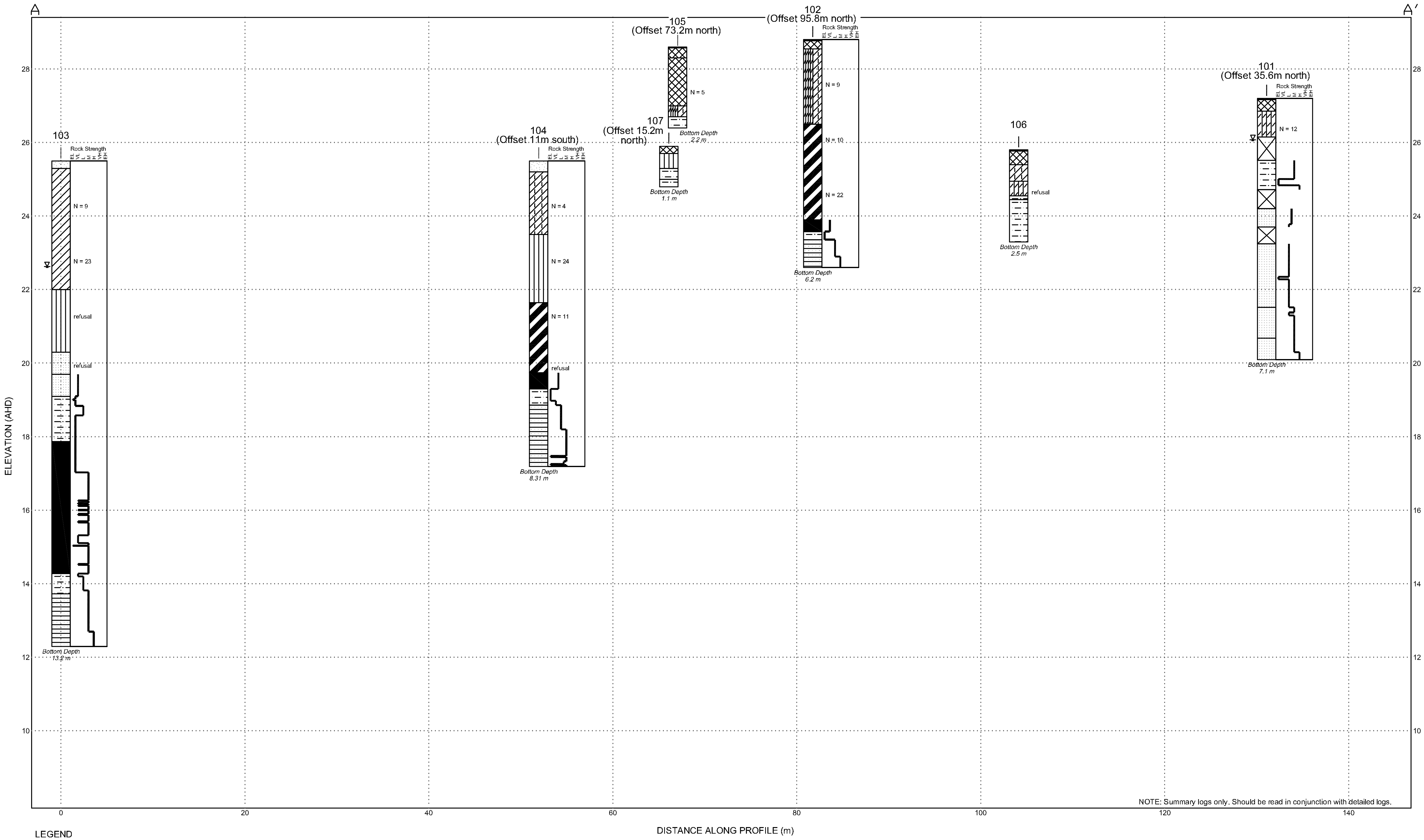
1:2000 @ A3

LEGEND

⊕ Approximate Borehole Location

▬ Approximate Site Area Subject to the Current Assessment

Douglas Partners Geotechnics / Environment / Groundwater	TITLE: **Test Location Plan** **Proposed Development** **Hunter Sports High School, Gateshead**		N	OFFICE: Newcastle DRAWN BY: PLH DATE: 09.10.2015
CLIENT: NSW Public Works	PROJECT No: 81598.01	DRAWING No: 1	REVISION: 0	SCALE: 1:2000@A3 Sheet



LEGEND

☒ Core Loss
■ Coal
░ Sandstone
▨ Carbonaceous Clay
■ Asphaltic Concrete
▩ Filling
▥ Silt
▨ Clay
▤ Laminite
▤ Siltstone
▥ Clayey Silt
░ Sand
▨ Silty Clay

TESTS / OTHER
N - Standard penetration test value
▽ - Water level

0 8
Horizontal Scale (metres)

Vertical Exaggeration = 4.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: NSW Public Works
OFFICE: Newcastle
DRAWN BY: PLH
SCALE: 1:400 (H) 1:100 (V) @ A3
DATE: 14.10.2015

TITLE: Section A-A
Proposed Development
Hunter Sports High School, Gateshead

PROJECT No: 81598.01
DRAWING No: 2
REVISION: 0