Geotechnical Investigation

235 Grose Vale Road, North Richmond, NSW

Prepared for: St John of God Health Care Inc C/- Johnstaff NSW Pty Ltd

EP1494.002 13 February 2020

APPROVED COMPANY ISO 9001 Quality Management Systems QMS Certification Services

APPROVED COMPANY AS/NZS 4801 OH&S Management Systems QMS Certification Services

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EP RISK

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St John of God Health Care Inc C/- Johnstaff NSW Pty Ltd
235 Grose Vale Road,
North Richmond, NSW 2754

13 February 2020

Our Ref: EP1494.002

LIMITATIONS

This Geotechnical Investigation was conducted on the behalf of St John of God Health Care Inc C/- Johnstaff NSW Pty Ltd for the purpose/s stated in **Section 1**.

EP Risk has prepared this document in good faith but is unable to provide certification outside of areas over which EP Risk had some control or were reasonably able to check. The report also relies upon information provided by third parties. EP Risk has undertaken all practical steps to confirm the reliability of the information provided by third parties and do not accept any liability for false or misleading information provided by these parties.

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QUALITY CONTROL

Version	Author	Date	Reviewer	Date	Quality Review	Date
V1	N. McGuire	11/02/2020	J Young	11/02/2020	J. Young	13/02/2020

DOCUMENT CONTROL

Version	Date	Reference	Submitted to
V1	13/02/2020	EP1494.002 v1	St John of God Health Care Inc C/- Johnstaff NSW Pty Ltd

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1 Introduction

EP Risk Management Pty Ltd ('EP Risk') was engaged by St John of God Health Care Inc ('SJGHC') to undertake a Geotechnical Investigation ('the Investigation') at the proposed 'St John of God Richmond Redevelopment' ('Proposed Development') at a property located at 235 Grose Vale Road, North Richmond, NSW ('the Site'). The Site is contained within Lot 11 in Deposited Plan ('DP') 1134453 as shown on **Figure 1**.

It is understood that the Proposed Development includes the upgrade and expansion St John of God Richmond Hospital comprising demolition of a portion of the existing facilities; upgrading of existing facilities to contemporary, best-practice standards; and construction of new multi- storey medical facilities including an increase in inpatient accommodation capacity from 88 to 112 beds.

For the purpose of the Investigation, concept plans of the proposed building locations were provided by SJGHC. Proposed development plans were provided to EP Risk and are attached as **Appendix A**.

It is understood that the geotechnical investigation is required to provide subsurface investigation to inform the structural design of the proposed Development and provide the relevant information required to undertake the structural design as requested in the brief from the Structural Engineer which included:

- Design of piled foundations, strip footings and pads, as appropriate;
- Design of high-level footings;
- Design of slabs on ground;
- Design of pavements and driveways;
- Design of retaining structures;
- Stability of batters and excavated faces, both temporary and permanent;
- Recommendations for shoring and underpinning of adjacent properties;
- Provision of requirements and methodology for site earthworks.
- Assessment of existing pavements.

2 Site Description

EP Risk undertook a site inspection on the 16th December 2019 comprising of a site walkover and visual assessment. The general site features and infrastructure observed during the inspection are presented in the Site photos attached in **Appendix B**. The general site features are discussed in more detail below.

The Site is currently operating as a mental health care hospital and the following site features were observed:

- A number of buildings across the Site utilised for various purposes including accommodation, treatment clinics, dining areas, cafes, places of worship, maintenance facilities, monasteries;
- A number of recreational facilities including tennis courts, a pool and a small golf course;
- A visitors carpark;
- The Battle of Richmond Hill Memorial Garden, a place of aboriginal cultural heritage significance located in the north east portion of the Site;
- An above ground diesel storage tank;
- Two liquified petroleum gas ('LPG') storage tanks;
- A large underground water storage tank used to store reclaimed water for irrigation adjacent to the Admin Building;

- Four large above ground fire hydrant water storage tanks adjacent to the visitors carpark;
- A large above ground reclaimed water storage tank adjacent to the maintenance shed.

Numerous mature native and ornamental trees were located at the Site, in particular in the eastern and southern portions of the Site.

Topographically the Site is situated on Richmond Hill which sits at an elevation of approximately 65 m to 70 m Australian Height Datum ('m AHD'). The access driveway into the Site sits on a south east ridgeline and the majority of the onsite buildings are situated on top of the Richmond Hill with moderate to steep slopes completely surrounding the Site. The topography of the surrounding area is hilly to undulating with moderate to steep slopes. The south east portion of the Site is bounded by very steep to extreme slopes (approximately 35 degrees to 55 degrees).

3 Investigation Methodology

The site investigation was undertaken on the 16th, 17th, 18th and 19th December 2019, comprising of the following:

- Drilling of five cored boreholes (BH1-BH5[1]) within the proposed NCCG building footprint with a truck mounted drill rig fitted with 125 mm solid flight augers and Tungsten-Carbide ('TC') bit attachment. Boreholes were drilled until TC bit refusal then advanced using NMLC HQ coring methods to a minimum depth of 5 m below TC bit refusal.
- Two hand auger boreholes (BH7 and BH8) in the proposed single-story health and wellness centre to confirm founding conditions.
- Five test bores (BH9-BH13) in the proposed road locations to confirm the design CBR and pavement design considerations.
- Standard penetration tests (SPT) were undertaken at the surface and at 1.5 m intervals in soils to assess soil strength parameters.
- Boreholes were backfilled with excavated spoil on completion.
- Sampling of the subsurface profile encountered, both soil and rock
- Core photography

Field investigation including logging of subsurface profiles and collection of samples was carried out by a geotechnical engineer from EP Risk. Boreholes were marked out and the co-ordinates including elevation were recorded by taking hand measurements from benchmark elevations and positions marked out by a registered surveyor. The locations of the boreholes are displayed on **Figure 1**. Subsurface conditions are summarised in **Section 5.2** below and detailed in the engineering logs attached in **Appendix C**, together with explanatory notes.

Sketch cross sections of the site and building locations showing interpreted subsurface profiles are presented in **Figure 2** and **Figure 3**.

[1] BH6 was abandoned at refusal on a concrete slab at approximately 0.3 m BGL. It is believed the concrete slab was a former footpath prior to the construction of the paved road.

4 Investigation Findings

4.1 Published Data

Based on the information contained in the NSW Department of Industry, Resources and Energy 1:100,000 Penrith Geological Map, the Site is predominately underlain by Middle Triassic aged Ashfield Shale comprising dark grey to black claystone-siltstone and fine sandstone-siltstone laminate. The north west portion of the Site is underlain by Bringelly Shale and Minchinbury Sandstone. There are no geological structures or faults located on the site or within the 1 km buffer around the Site.

Based on the NSW Mine Subsidence District Maps, the site is not located within any proclaimed mine subsidence district.

4.2 Subsurface Conditions

The subsurface conditions encountered in the borehole locations comprised:

- FILL: Sandy Silty GRAVEL/Sandy Gravelly SILT – pavement material, dry, medium to coarse sub-angular gravel.
- RESIDUAL: Sandy CLAY/Sandy CLAY with Gravel stiff to very stiff, medium to high plasticity, dry.
- Extremely Weathered (XW) SHALE estimated very low strength, close to medium spaced defects.
- SHALE: estimated low to medium strength, distinctly weathered (DW), wide spaced defects.

Groundwater was encountered within the fractured shale layer at between Reduced Level (R.L) 55 m AHD and 57 m AHD across the Site. It should be noted that groundwater levels are likely to fluctuate with site and climatic conditions.

The subsurface profiles encountered in the boreholes are detailed in the engineering logs attached in **Appendix C** together with Explanatory Notes. In addition to the engineering logs, bore hole locations are depicted in **Figure 1** attached.

A summary of the subsurface profile encountered in the boreholes is presented in **Table 1** based on geotechnical units and the depth the units were encountered below existing ground level. Comments are provided in relation to material properties, expected performance and the soil moisture status.

Table 1 – Geotechnical Units

Unit	Material	Description / Depth Encountered
1A	ASPHALT	Asphalt 30-50 mm seal encountered in BH1-BH5.
1B	TOPSOIL	Sandy SILT – Encountered in the northern portion of the Site from the surface to approximately 0.2 m BGL.
2A	FILL	Sandy Silty GRAVEL/Sandy Gravelly SILT – pavement material, dry, medium to coarse sub-angular gravel. Encountered in the southern portion of the Site from the surface to approximately 0.5 m BGL.
2B	Residual - Sandy Silty CLAY	Stiff to very stiff, medium to high plasticity, dry. Encountered in the northern portion of the Site from 0.2 m BGL to >3 m BGL.
3A	XW SHALE	Extremely weathered, estimated very low strength. Encountered from 0.3-05 m BGL to 0.5-0.7 m BGL.
3B	DW SHALE – very low to medium strength.	Distinctly weathered, estimated very low to medium strength, laminated bedding at 0 to 10 degrees. Encountered from 0.5-0.7 m BGL to 6 m BGL.
3C	SHALE – medium to high strength	Distinctly weathered, estimated medium to high strength, laminated bedding at 0 to 10 degrees. Encountered from >6 m BGL.

4.3 Groundwater Levels

A summary of the groundwater elevation levels is presented in **Table 2**.

Table 2 – Encountered Groundwater Levels

Bore ID	Depth to Groundwater from Top of Casing (m)	Elevation of Top of Casing (m AHD)	Groundwater Elevation (m AHD)
BH1 (MW01)	7.18	63.5	56.32
BH3 (MW02)	6.51	63.16	56.65
BH5 (MW03)	7.23	62.9	55.67

Groundwater was generally encountered at between R.L 55-57m AHD across the Site and the general groundwater flow direction is considered to be east to north east.

5 Laboratory Results

The laboratory certificates of analysis are presented as **Appendix E** and a summary of the results has been provided below.

5.1 Particle Size Distribution Results

Particle Size Distribution Tests (PSDs) were undertaken at three boreholes to aid in soil classification. The results of the testing are summarised in **Table 3**.

Table 3 – Particle Size Distribution Results

Bore ID	Depth (m)	% passing 2.36 mm sieve	% passing 75 µm sieve	Sample Description
BH5	0.0-0.3	55	15	Sandy Silty GRAVEL (GW)
BH10	1.0-1.5	78	58	Sandy CLAY with gravel, medium plasticity (CI)
BH12	2.5-3.0	99	73	Sandy Silty CLAY, high plasticity (CL)

5.2 Uniaxial Compressive Strength Test Results

One Uniaxial Compressive Strength Test per borehole was undertaken to aid in determining the end bearing capacity for foundations. The results of the testing are summarised in **Table 4**.

Table 4 – Uniaxial Compressive Strength Test Results

Bore ID	Depth (m)	Sample Description	Uniaxial Compressive Strength qu (MPa)
BH1	6.5-6.6	SHALE	1.88 (Very Low Strength)
BH2	4.4-4.5	SHALE	5.48 (Low Strength)
BH3	6.2-6.4	SHALE	22.8 (High Strength)
BH4	5.9-6.0	SHALE	18.3 (Medium Strength)
	8.0-8.2	SHALE	9.14 (Medium Strength)
BH5	5.9-6.0	SHALE	1.28 (Very Low Strength)

The uniaxial compressive strength results indicate a high variability in rock strength however medium or higher strength rock was encountered in BH3 and BH4 at a depth greater than 6 m.

5.3 Point Load Index Test Results

Point Load Index Tests (PLIs) were undertaken on each borehole to assist estimating rock strength of the recovered core. The results of the testing are summarised in **Table 5**.

Table 5 – Point Load Index Test Results

Bore ID	Depth (m)	Sample Description	Is_{50} (MPa) and Strength Category
BH1	2.0	SHALE	1.3 (High)
	3.0	SHALE	0.6 (Medium)
	4.0	SHALE	3.4 (Very High)
	5.0	SHALE	0.43 (Medium)
	6.0	SHALE	0.12 (Low)
	7.0	SHALE	0.084 (Very Low)
BH2	1.0	SHALE	0.29 (Low)
	2.0	SHALE	0.47 (Medium)
	3.0	SHALE	0.21 (Low)
	4.0	SHALE	0.42 (Medium)
	5.0	SHALE	0.21 (Low)
	6.0	SHALE	0.33 (Medium)
	7.0	SHALE	N/A
BH3	1.0	SHALE	0.034 (Very Low)
	2.0	SHALE	0.044 (Very Low)
	3.0	SHALE	0.12 (Low)
	4.0	SHALE	N/A
	5.0	SHALE	0.45 (Medium)
	6.0	SHALE	0.79 (Medium)
	7.0	SHALE	0.55 (Medium)
BH4	2.0	SHALE	0.12 (Low)
	3.0	SHALE	0.81 (Medium)
	4.0	SHALE	0.34 (Medium)
	5.0	SHALE	1.2 (High)
	6.0	SHALE	2.7 (High)
	7.0	SHALE	1.2 (High)
	8.0	SHALE	1.9 (High)
BH5	1.0	SHALE	4.9 (Very High)
	2.0	SHALE	2.3 (High)
	3.0	SHALE	N/A
	4.0	SHALE	N/A
	5.0	SHALE	0.97 (Medium)
	6.0	SHALE	0.55 (Medium)
	7.0	SHALE	1.6 (High)

5.1 California Bearing Ratio Results

Two California Bearing Ratio (CBR) tests were undertaken at BH9 and BH12 to confirm the design CBR for the proposed pavements. Results of the testing are summarised in **Table 6**.

Table 6 – California Bearing Ratio Test Results

Test Pit	Depth (m)	Sample Description	W[2] (%)	SOMC[3] (%)	SMDD[4] (%)	Swell (%)	CBR (%)
BH9	0.5-1.0	XW SHALE	6.2	11.4	1.96	0.0	15
BH12	1.0-1.5	Sandy CLAY, medium plasticity (CI)	16.0	21.8	1.64	0.0	8

The results of the laboratory testing indicate the subgrade within the proposed road has a moisture content dry of SOMC at the time of fieldwork and the CBR ranged from 8-15% based on the expected subgrades. The CBR where remoulded to approximately 100% standard relative density at approximately standard optimum moisture content and soaked for 4 days prior to testing. DCP correlations indicated an in-situ CBR of between 7->10 %. It should be noted that DCP correlations are moisture sensitive and the investigation was undertaken during a relatively dry period, however in-situ results correlate well with soaked CBRs

5.2 Soil Aggressivity Results

The results of the chemical testing on four samples of the clay soil obtained from the boreholes are presented in **Table 7** below.

Table 7 – Soil Aggressivity Results

Bore ID	Depth	Sample Description	pH	EC (µs/cm)	Sulfate as SO_4 (mg/kg)	Chloride (mg/kg)	Exposure Classification
BH1	1.0	XW SHALE	6.6	44	50	36	Non-aggressive
BH3	0.5	FILL	5.9	48	36	12	Non-aggressive
BH5	0.4	FILL	7.3	89	110	110	Non-aggressive
BH12	2.0	Sandy Silty CLAY	5.4	64	<30	340	Mild

BH1, BH3 and BH5 are in locations of the proposed concrete piles and therefore based on these results it is considered that the 'mild' exposure classification for concrete piles (as recommended Table 6.4.2(C) in AS2159-2009) is applicable for this site.

The results of the chemical testing undertaken imply that a classification of 'Non-aggressive' conditions applies for steel piles in soil.

[2] Field Moisture Content.
[3] Standard Optimum Moisture Content.
[4] Standard Maximum Dry Density.

5.1 Groundwater Corrosion Results

The results of the chemical testing on two samples of the groundwater obtained are presented in Table 8 below.

Table 8 – Groundwater Corrosion Results

Bore ID	pH	Sulphite as SO_3 (mg/L)	Sulphate as SO_4 (mg/L)	Chloride (mg/L)	Exposure Classification
BH3 (MW02)	7.4	<2.5	790	480	Non-aggressive
BH5 (MW03)	6.9	<2.5	79	750	Non-aggressive

BH1, BH3 and BH5 are in locations of the proposed concrete piles and therefore based on these results it is considered that the 'non aggressive' exposure classification for concrete piles (as recommended Table 6.4.2(C) in AS2159-2009) is applicable for this site.

The results of the chemical testing undertaken imply that a classification of 'Non-aggressive' conditions applies for steel piles in soil.

6 Foundation Conditions and Recommendations

6.1 Proposed Development

It is understood that the proposed development includes the construction of multiple two and three-storey buildings in the south east portion of the Site.

Concept plans of the proposed development were provided for the purposes of undertaking the Investigation and are attached as **Appendix A**. The plans show a design floor level of 62.5 m AHD for the proposed buildings. Based on this, site regrade is expected to be approximately 0.0 m and 2.0 m below existing ground level with the majority of cut required for the proposed pavilion 1 and 2 portions on the western side of the proposed building.

6.2 Foundation Conditions and Parameters

Given the proposed development and the likely relatively high loads associated with the proposed multistorey buildings, the most appropriate footing system will likely comprise piled footings founded in rock below the shallow Units 1a fill and 1bresidual soil profile.

All structural elements should be founded on similar materials to reduce the potential for differential settlements and subsequent damage to the structures.

Discussion of the options and design parameters for the relevant materials is provided in the following sections.

6.3 Site Classification

AS2870-2011, '*Residential Slabs and Footings*', sets out criteria for the classification of a site and the design and construction of a footing system for a single dwelling house, townhouse or a similar structure. The standard can also be used for other forms of construction, including some light industrial, commercial and institutional buildings if they are similar in size, loading and performance expectation to a typical domestic structure.

The potential site classifications are preliminary in nature and will require confirmation following site re-grading once final site levels and natural/fill soil profiles are known.

Due to the presence of uncontrolled fill to depths of greater than 0.4 m the site, disturbance caused by demolition of existing structures and presence of trees, in its current condition the site is classified as **Class 'P'** in accordance with AS2870-2011. Either high level or deep (piled) footings can be adopted., **Class M**, Moderately Reactive would be appropriate for founding of high-level footings where appropriate in residual clays.

6.4 Geotechnical Design Parameters

Based on the subsurface investigation results, general material design parameters are presented in **Table 9** for the geotechnical units outlined in **Table 1** and should be used as guidance for the design. The detailed design of foundations should consider the structural loads against serviceability and ultimate limit state criteria. It is recommended that analysis be undertaken during structural design to enable suitable foundation selection and to determine expected settlements.

Consideration should be given to proximity of existing foundations below neighbouring structures during design and construction. Dilapidation surveys are recommended where piling would impact neighbouring structures through vibration transfer or piling excavations.

Table 9 – Geotechnical Material Design Parameters

Geotechnical Unit	2A	2B	3A	3B	3C
Material Type	Controlled FILL – Sandy Silty CLAY	RESIDUAL – Sandy Silty CLAY	XW SHALE – Very Low Strength	DW SHALE – Very Low Strength	SW SHALE – Medium to High strength
Bulk Density (kN/m^3)	19	19	21	22	22
Angle of friction (Phi')	25	25	30	35	35
Cohesion (kPa)	5	5	100	300	400
Undrained shear strength Su (kPa)	50	<4m depth 50 >4m depth 75	-	-	-
Effective Elastic Modulus E' (MPa)	20	<4m depth 20 >4m depth 30	100	200	500
Poisson's Ratio (v)	0.3	0.3	0.3	0.3	0.25

6.5 Shallow Footings

High level footings may comprise raft, pad and/or strip footings supporting line loads or column loads. High level footings can be designed for allowable bearing pressures of 150 kPa where founded in residual stiff Clay soils (Unit 2B) with allowance for side adhesion of 20 kPa.

The assessment of serviceability beneath shallow footings founded as described above may be undertaken assuming a Young's modulus of the founding material of E = 20 MPa for the Unit 2B Clay soils to 4 m depth.

Shallow footings where required founded within Controlled Fill or residual soils of at least very stiff strength can be designed on the basis of a maximum allowable base bearing pressure of 150kPa.

Footings should not be founded in topsoil or uncontrolled fill that has not been placed in accordance with Level 1 inspection and testing requirements as defined by AS3798-2007.

If further site regrading works are undertaken at the site, reclassification may be required once final cut and fill depths and fill material types are known.

It is typically recommended that all footings for an individual structure be founded on similar materials. All footings, edge beams and internal beams should be founded outside or below the zones of influence resulting from existing or future service trenches, retaining walls or other subsurface structures.

6.6 Pile Foundations

6.6.1 General

As discussed above, piles are likely to be the most appropriate footing system given the proposed development. Key criteria in the selection and design of pile type are:

- Shallow soil profile approximately 0.5 m depth in the proposed building area.
- Relatively deep groundwater level in in fractured shale at approximately 6.5 to 7.2 m depth.
- Highly fractured shale rock strata that exhibits variable rock weathering, strength and fracture spacing.

Pile options include bored reinforced concrete piles, steel screw piles or driven displacement piles founded in weathered shale.

Bored piers founded within the rock strata are a suitable option. Based on the shallow soil profile overlying rock, it is expected that bored piers will temporarily stand unsupported and that caisson support will not be required, however contractors should make their own assessment of this. Groundwater inflows into pier holes may be encountered during drilling and should be considered in the construction sequence if bored reinforced concrete piles are proposed and will be dependent on weather conditions prior and during construction and depth of foundations.

Driven displacement piles such as treated hardwood, steel or concrete piles driven to refusal in the rock strata are a potential foundation option. However, due to the close proximity of nearby structures and sensitive receptors, ground vibration from driven piles is likely to be a constraint.

6.6.2 Geotechnical Reduction Factor

The design should include assessment of both strength and serviceability limit states.

With reference to AS2159-2009, Piling - Design & Installation [2], and considering the existing geotechnical information along with common design and construction practices, a basic geotechnical reduction factor (φ_{gb}) of **0.45** should be applied to the ultimate values.

The geotechnical reduction factor is based on low redundancy system, assumes no pile testing during construction. Further reduction can be undertaken by imposing a pile testing program and re-assessing the geotechnical reduction factor.

6.6.3 Bored Piers

Bored piers founded in Unit 2, 3a, 3b or 4 rock may be proportioned, based on the parameters shown in **Table 10**.

Table 10 – Geotechnical Material Design Parameters – Bored Piers

Geotechnical Unit	2B	3A	3B	3C
Material Type	RESIDUAL – Sandy Silty CLAY	XW SHALE – Very Low Strength	DW SHALE – Low to medium strength	SW SHALE – medium to high strength
Ultimate End Bearing Capacity (kPa)	2-4m 450 >4m 675	1000	3500	5,000
Ultimate shaft Adhesion (kPa) - Downthrust	2-4m 25 >4m 50	150	300	450
Ultimate shaft Adhesion (kPa) - Uplift	2-4m 15 >4m 30	75	150	200

Notes to table:
Effects of buoyancy to be considered with unit weight
Shaft adhesion to be ignored for top 1.5 pile diameters
Ultimate end bearing capacity based on settlement of <1% of minimum footing width
Shaft adhesion values based on a clean and roughened socket

The above parameters assume that the pile base is cleaned of debris and compressible materials. For the design of piles with shaft adhesion in weathered rock, the walls of the holes should be roughened and free of clay smear prior to placing of reinforcement and concrete.

6.6.4 Driven Piles

Timber Piles

Suitably treated hardwood piles would be expected to meet practical refusal after minor penetration of the extremely weathered very low strength shale rock (Unit 2).

Timber pile penetration of the rock surface is expected to be minimal and uplift capacity of the shallow overlying Unit 1A or 1B is expected to be negligible (<2 m soil profile).

The down thrust load capacity of timber piles driven to practical refusal with appropriately sized equipment will approach the structural capacity of the pile and timber pile capacity can be checked during construction on the basis of a suitable dynamic pile formulae.

Load capacities for treated hardwood piles conforming to F27 strength grade can be obtained from suppliers. As a guideline maximum safe load capacity for pure axial loading for F27 treated hardwood timber piles is likely to range from 715kN for a 210mm toe diameter pile to 1460KN for a 300mm toe diameter pile.

An alternative to timber piles is driven steel or concrete piles.

Steel Piles/Concrete Piles

Steel and concrete pile capacity can be significantly greater than that available from timber piles. The down thrust load capacity of a steel pile section driven to practical refusal with appropriately sized equipment will approach the structural capacity of the pile.

Driven pile capacity should be checked during construction on the basis of a suitable dynamic pile formulae.

It is expected that driven steel H section piles will achieve rock embedment in Unit 2 very low strength extremely weathered shale. It is noted that whilst steel H piles have high driveability, they are prone to deflection if obstructions or inclined rock surfaces are encountered.

Load capacities for steel H piles can be obtained from suppliers. As a guideline high yield stress H steel piles are expected to have maximum safe load capacity for pure axial loading in the order of 1500 kN for 250 mm x 250 mm sections and 3000 kN for 300 mm x 300 mm sections.

7 Excavation and Retaining Structures

7.1 General

The plans show a final design floor level of 62.5 m AHD and as such excavation for the buildings are likely to be in the order of 0.5 m to 2 m below existing ground level.

Given the close proximity of onsite buildings to be retained, retention may be required and where practical temporary batter slopes should be adopted prior to wall construction. Where temporary batter slopes are adopted for excavations, permanent support can be provided on completion of excavation by engineered retaining walls constructed at the toe of the batter and subsequently backfilled.

7.2 Excavation Conditions and Batter Slopes

Excavations to the depths noted above will encounter Unit 1A fill soils generally comprising coarse grained road pavement material. These materials can be readily excavated using conventional earth-moving equipment, such as hydraulic excavators, dozers or similar.

Temporary batter slopes up to 2 m depth through the Unit 1B Clay soils may be cut at no steeper than 1H:1V. Temporary slopes should be protected by diverting surface water flows and by covering with impermeable plastic sheeting or similar during rainfall.

Permanent batter slopes in cut and fill to be constructed at no steeper than 2H:1V and protected by rapidly establishing vegetation cover. Suitable drainage measures should be provided for all batter slopes. As a minimum, surface drains should be installed at the top of the slope to divert water away from the face.

7.3 Groundwater and Dewatering

Groundwater at the time of investigation was encountered at between 55 and 57 m AHD in the proposed buildings. The proposed final design floor levels for is 62.5 m AHD and therefore groundwater is unlikely to be encountered with the exception of any proposed pile foundations exceeding a depth of 6 m below the existing ground level.

It is noted that groundwater conditions may change depending on rainfall conditions prior to and during construction.

7.4 Retaining Wall Design Parameters

Garden landscaping walls in excess of 1 m in height and all structural retaining walls should be designed by an engineer in accordance with AS 4778 – Earth retaining structures. Design of retaining walls should:

- Consider surcharge loading from slopes and structures above the wall;
- Take into account loading from any proposed compaction of fill behind the wall;
- Provide adequate surface and subsurface drainage behind all retaining walls, including a free draining granular backfill to prevent the build-up of hydrostatic pressures behind the wall;
- Utilise materials that are not susceptible to deterioration; and
- Ensure walls are founded in materials appropriate for the loading conditions.

Footings for proposed retaining walls should be founded below any topsoil and uncontrolled fill within stiff or better clay or weathered rock.
Suitable retaining walls for the site are likely to comprise gravity or cantilever walls in areas where temporary battered excavations are undertaken. Based on the limited excavation depth and the ground conditions, the use of a continuous pile wall construction is unlikely.

Recommended retaining wall design parameters are provided below in **Table 11** for the existing site soils. Design parameters for imported fill and drainage materials behind the wall to be assessed as part of the retaining wall design. Design must include assessment of the global stability of the wall.

Table 11 – Retaining Wall Design Parameters

Parameter	Controlled FILL/ Residual Sandy CLAY	Extremely Weathered Shale
Effective Friction Angle ϕ'°	25	30
Effective Cohesion c' kPa	5	5
Level Active earth pressure coefficient[(1)] K_a	0.3	0.35
Level Passive earth pressure coefficient[(1)] K_p	2.46	3
At Rest earth pressure coefficient K_o	0.58	0.5
Bulk unit weight kN/m^3	19	21
Undrained Cohesion kPa	50	100
Allowable Bearing Capacity (kPa)	100	200

Notes to Table

(1) Based on level ground surface adjacent footings values shall be adjusted accordingly to account for actual site slopes above and below retaining walls/footings.

Retaining walls may be designed on the basis of a triangular lateral earth pressure distribution in the Unit 1 materials using the following characteristic earth pressure coefficients and subsoil parameters:

- For cantilever wall or gravity walls where movement is of little concern, active earth pressure coefficient can be used assuming a horizontal backfill surface;
- If the top of the cantilever retaining wall is to be restrained, such as by ground floor slabs of the permanent structure, or if the walls are retaining areas which are sensitive to movement, an 'at rest' earth pressure coefficient (can be used;
- For lateral restraint the retaining walls must be embedded below the bulk excavation level and a passive earth pressure coefficient (kp) used. This value assumes that excavations are not carried out within the zone of influence of the wall toe and there are no soft zones evident in the floor of the excavation. This would need to be checked during construction;
- Any surcharge affecting the walls (eg: existing building footings, traffic loading, adjacent retaining walls and their backfill, etc.) should be allowed in the design. If inclined backfill surfaces are proposed, then the lateral earth pressure coefficient would have to be appropriately increased or the inclined surface treated as a surcharge.

7.5 Retaining Wall Drainage

The wall backfill should comprise free draining, granular material. Drainage behind the wall should comprise a geocomposite drain or geotextile wrapped gravel drain at the back of the wall that drains to a geotextile wrapped subsoil drain along the wall toe. The toe drain should discharge to the site stormwater system to provide long term drainage behind excavation walls. Flushing points should be incorporated into the design of the perimeter drain to allow periodic maintenance.

7.6 Temporary Working Platforms

Temporary work platforms constructed for piling rigs and cranes to be constructed based on specific assessment and analysis of the proposed loading conditions. General preparation will require stripping of any topsoil to expose a stiff clay or suitable fill subgrade which can be proportioned for an allowable bearing capacity of 100 kPa. The material composition, compaction and thickness of the working platform will need to be designed accordingly to limit applied stress at subgrade level to 100 kPa or less.

8 Pavement Design and Construction Considerations

8.1 Design Traffic

Design traffic loadings and pavement thickness design calculation has been undertaken by EP Risk in accordance with Council Engineering Requirements for Development [6] for all roads in proposed development. Based on a design CBR of 6% for sandy clay subgrade.

The design traffic data for construction has been determined on the basis of a local road in the absence of specific traffic data and the following assumptions in **Table 12.**

Table 12 – Recommended Road Type and Design ESA's

Road Type	Design ESA's
Commercial / Light Industrial (dead end)	$2x\ 10^6$

Where traffic data varies from the above assumptions a review of pavement design may be required particularly considering connectivity with adjacent developments.

8.1.1 Design Parameters

Pavement thickness design has been performed in accordance with Austroads AGPT02-17 Guide to Pavement Technology, Part 2: Pavement Structural Design [5] based on the following parameters.

- Design subgrade CBR of 6% for sandy clay.

The design subgrade has been determined in accordance with Section 5 of Austroads 2017 [5]on the basis of both laboratory and field-testing results and considered the surcharge applied by pavement layers.

8.1.2 Option 1 – Flexible Unbound Pavement (Clay Subgrade)

The option of pavement reconstruction utilising flexible unbound pavement materials is detailed in **Table 13.**

Table 13 – Recommended Flexible Pavement Compositions (Clay Subgrade)

Road Type	Local Access
Wearing Course (mm)	50 AC14*
Basecourse (mm)	150
Subbase (mm)	180
Select (mm)	-
Total Thickness (mm)	380
Subgrade CBR%	min 6%
Allowable DESA	2×10^6

Notes:
*AC 14 or AC10 with 7mm primer seal placed under all asphaltic concrete wearing surfaces.

A minimum of fourteen days duration shall apply following application of the primer seal prior to placement of the AC wearing course. That period may be extended or shortened subject to approval by Council.

8.2 Subgrade Preparations

Where construction of a new pavement is proposed, subgrade preparation should be in general accordance with the following procedures.

- Excavation to design subgrade level, removal of any uncontrolled fill with ripping to 300-350mm below design subgrade level and recompact to a minimum 100% of SMDD. Moisture contents should be within 60 to 90% of SOMC.
- Static proof-rolling of the exposed subgrade using a heavy (minimum 10 tonne) roller under the direction of an experienced geotechnical consultant
- Loose or yielding areas should be excavated and replaced with compacted select fill or suitable subgrade replacement comprising of material of similar consistency to the subgrade.
- Where filling or subgrade replacement is required, the materials employed should be free of organics or other deleterious material. The material should also have a maximum particle size of 100 mm or one third of the layer thickness, with a soaked CBR > 5% or depending on the pavement option adopted.
- Where a select layer is to be utilised in construction of the pavement. The material shall be well graded granular material with minimum 4 day soaked CBR of 15% and PI ≤15%. The select layer should be compacted to a minimum 100% of SMDD. Moisture contents should be within 60 to 90% of SOMC.

Following satisfactory preparation of the subgrade, the pavement should be placed in accordance with the requirements of the appropriate section of this report and council construction guidelines [6] depending on the subgrade type.

8.3 Materials

8.3.1 Specifications and Compaction Requirements

Pavement materials and compaction requirements for new pavement construction should conform to Council requirements and the following requirements.

Table 14 – New unbound pavement construction: Material specification and compaction requirements

Pavement Course	Material Specification	Compaction Requirements
Base Course DGB20 (Class 2) [6] & NGB20	Material complying with Council Specifications [7] with Category C CBR > 80%, with PI ≤ 6%	Min 98% Modified (AS 1289 5.2.1)
Subbase Subbase quality crushed rock	Material complying with Council Specifications [7] with CBR >30% with PI ≥2≤ 10%	Min 95% Modified (AS 1289 5.2.1
Select Granular material	Well graded granular material with CBR min 15% and PI ≤15%	Min 100% Standard (AS 1289 5.1.1)
Subgrade or replacement	Minimum CBR 6%	Min 100% Standard (AS 1289 5.1.1)

All granular pavement material quality for the subdivision roads should be in general accordance with RMS QA Specification 3051 for Traffic Category C equivalent to Section 3 of Council's Engineering Construction Guideline May 2018 [6].

Minimum testing on all potential imported pavement materials should be in accordance with RMS 3051 Ed 7. Pre-treatment of material prior to testing would be advisable for materials subject to breakdown.

8.3.2 Wearing Course

Wearing courses should be in accordance with Council's Engineering Construction Guidelines [6] for asphaltic concrete roadways with reference to RMS QA Specifications R106 for Sprayed Bituminous Surfacing for primer seal and RMS QA Specifications R116 for Dense Graded Asphalt.

The design and construction of wearing courses should be in in consultation with the preferred supplier taking into account traffic volume and type. All pavement surfaces should be primer sealed prior to the application of the asphaltic concrete (AC) wearing course. A minimum delay of 14 days is required after the primer seal before placement of the AC wearing course

8.3.3 Drainage

The moisture regime associated with a pavement has a major influence on the performance considering the stiffness/strength of the pavement materials is dependent on the moisture content of the material used. Accordingly, to protect the pavement materials from wetting up and softening, particular care would be required to provide a waterproof seal for the pavement materials, together with adequate surface and sub-surface drainage of the pavement and adjacent areas.

It is recommended that subsoil drainage be installed at subgrade level preferably along both sides of the road alignments, but at a minimum along the high side of any road and adjoining garden beds. The subgrade should be constructed with sufficient cross fall (in general 3%) to assist in reducing retention time for moisture entering the pavement. The subsoil drains should be placed under or at the back or kerb and the shoulder sealed with a low permeability material to prevent moisture ingress into the pavement. Sealing of shoulder / verges with low permeability material where kerb and gutter is not employed is recommended to reduce potential for moisture ingress into the pavement.

The selection, construction and maintenance of appropriate drainage mechanisms would be required for adequate performance. The selection of appropriate construction materials that are relatively insensitive to moisture change is also essential in area subject to periodic inundation, even if for a relatively short period of time.

8.4 General Construction Considerations

8.4.1 Pavement Interface and Tie-in

Where new pavement construction abuts an existing pavement care should be exercised to either create a clean vertical construction joint or bench into the base course layer for a minimum of 0.5 m for the entire pavement width. Longitudinal construction joints should be located outside of wheel paths. Pavement thickness designs for higher category roads should be continued into the adjoining road past the turning point and then tapered to avoid abrupt changes in subgrade stiffness.

Adequate compaction of the subgrade and pavements in this area is essential to maximise performance of the pavement. It is noted that where pavements of variable composition or thickness are abutted, the potential for localised failure is generally greater. Consideration should be given to sealing any cracks that may develop between existing and new pavements. The use of a strain alleviating membranes at the interface may also be appropriate. It is recommended to install intra-pavement drainage at subgrade level at interfaces of variable existing and new pavements.

8.4.2 Inspections

The subgrade will require inspection by an experienced geotechnical consultant after boxing out or filling to design subgrade level. The purpose of inspection is to confirm design parameters, assess the suitability of the subgrade to support the pavement, and delineate areas which may require subgrade replacement or remedial treatment prior to construction. DCP testing is recommended following stripping or preliminary boxing to confirm subgrade strength and design parameters.

All works and materials used in construction should be constructed in accordance with Councils Engineering Design and Construction [6] Guideline or as specified in this report. Where discrepancies may occur, clarification should be sought from Council.

9 Conclusions

EP Risk was engaged by SJGHC to undertake a Geotechnical Investigation for a proposed Richmond Hospital Upgrade located at 235 Grose Vale Road, North Richmond, NSW.

It is understood that the Proposed Development includes the upgrade and expansion St John of God Richmond Hospital comprising demolition of a portion of the existing facilities; upgrading of existing facilities to contemporary, best-practice standards; and construction of new facilities including an increase in capacity from 88 to 112 beds.

Topographically the Site is situated on Richmond Hill which sits approximately at an R.L of 65 m to 70 m Australian Height Datum ('m AHD'). The access driveway into the Site is situated on a south east ridgeline and the majority of the onsite buildings are situated on top of the Richmond Hill with moderate to steep slopes completely surrounding the Site. The topography of the surrounding area is hilly to undulating with moderate to steep slopes. The south east portion of the Site is bounded by very steep to extreme slopes (approximately 35 degrees to 55 degrees).

The subsurface conditions encountered in the borehole locations comprised:

- FILL: Sandy Silty GRAVEL/Sandy Gravelly SILT – pavement material, dry, medium to coarse sub-angular gravel.
- RESIDUAL: Sandy CLAY/Sandy CLAY with Gravel stiff to very stiff, medium to high plasticity, dry.
- Extremely Weathered SHALE estimated very low strength, close to medium spaced defects.
- SHALE: estimated low to medium strength, distinctly weathered, wide spaced defects.

Groundwater was encountered within the fractured shale layer at between 55 m AHD and 57 m AHD across the Site.

The SPT testing indicated that surface soils are typically firm to stiff with strength increasing with depth and very stiff to hard clays encountered above the weathered rock profile. Shallow foundations in stiff sandy clay could be proportioned for an allowable bearing capacity of 150 kPa.

Rock strength testing indicated variable strength with the test bores, but indicated an end bearing capacity of 1,500 kPa for extremely weathered shale and 3,500 kPa for low strength shale could be adopted.

Laboratory four day soaked CBR testing indicated CBR values of 8% for sandy clay subgrade and 15% for extremely weathered shale, a design CBR of 6% is considered appropriate for pavement design.. Based on test pitting the subgrade conditions along the proposed road alignments will comprise of weathered shale subgrade at the top of Richmond Hill transitioning to a sandy clay subgrade further down the hill towards the north. In the absence of specific traffic data, the pavements have been designed for 2 x 10^6 ESA's

Laboratory results indicated the residual and fill soils present mildly aggressive conditions to buried concrete and non-aggressive conditions for steel structures. Groundwater was non-aggressive to steel and concrete structures.

10 References

[1] Penrith 1:100 000 Geological Sheet 9030, Geological Survey of New South Wales, Sydney.

[2] Australian Standard AS2159-2009, “Piling - Design & Installation,” Standards Australia, 2009

[3] J.E. Bowles, 1996, “Foundation Analysis and Design” (fifth ed.), McGraw-Hill Inc., New York, 1996.

[4] Road and Maritime Service QA Specification B30 Excavation and Backfill for Bridges Edition 4 / Revision 1 March 2018.

[5] Austroads AGPT05-11, “Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design,” Austroads Ltd, October 2011.

[6] Hawkesbury City Council – Development Control Plan 2012

[7] RMS QA Specification 3051 (Ed 7 Rev 0), “Granular Base and Subbase Materials for Surfaced Road Pavements,” Roads and Maritime Services, August 2018.

Figures

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Geotechnical Investigation
235 Grose Vale Road, North Richmond, NSW

Figure 1 - Soil Sampling Locations

Job No: EP1494.001
Date: 30/01/2020
Version No: v1

N

0 20 40 60 m

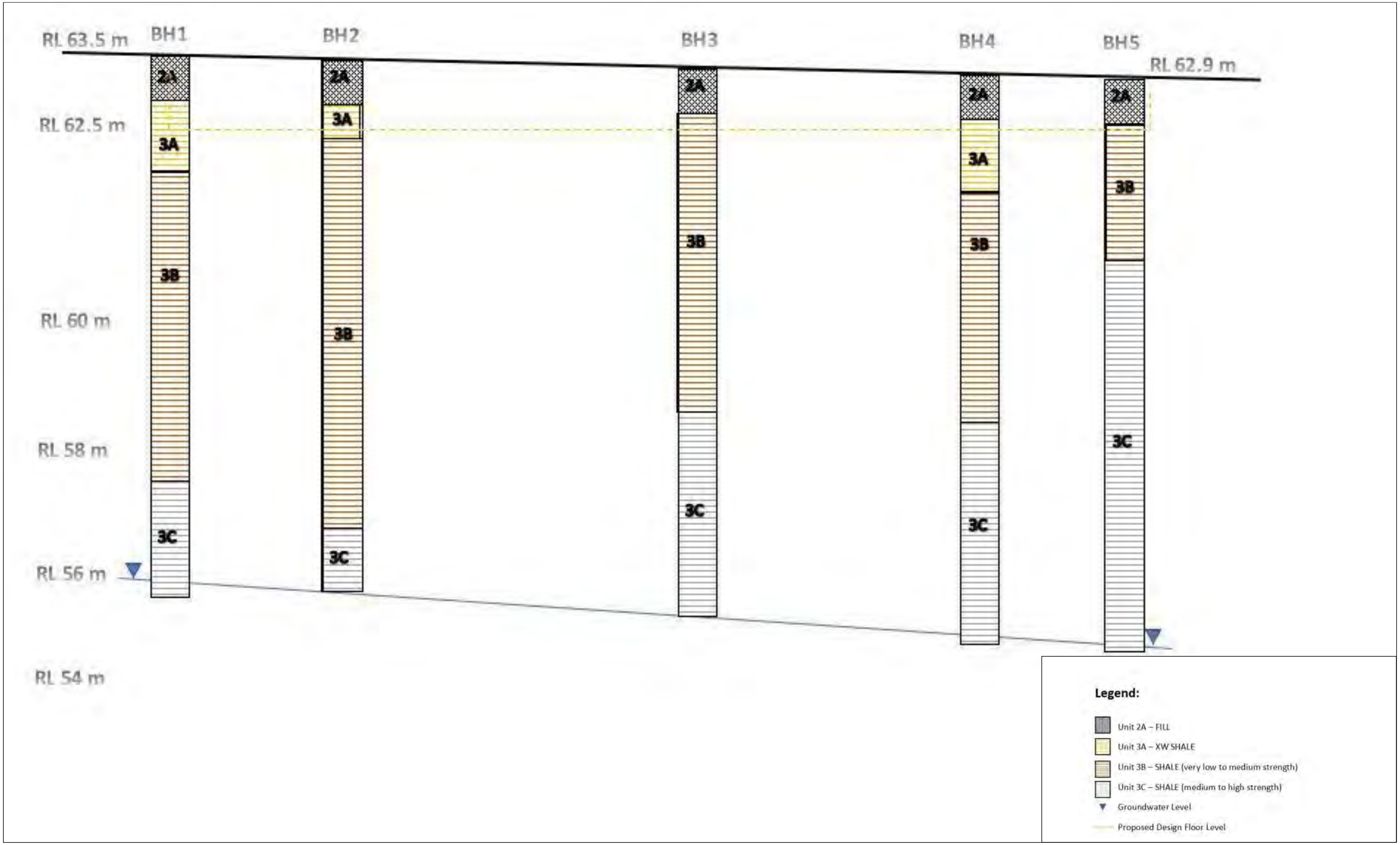
Approximate Scale Only

Coordinate System: MGA 56
Drawn by: NM Checked by: PS
Scale of regional map not shown
Source: Nearmaps

APPROVED COMPANY ISO 9001 Quality Management Systems

APPROVED COMPANY AS/NZS 4801 OH&S Management Systems

APPROVED COMPANY ISO 14001 Environmental Management Systems



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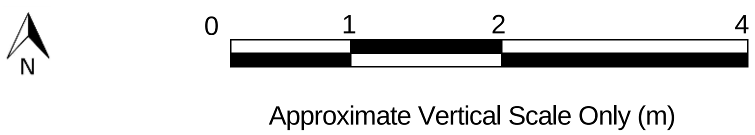
Geotechnical Investigation
235 Grose Vale Road, North Richmond, NSW

Figure 2 – Cross Section

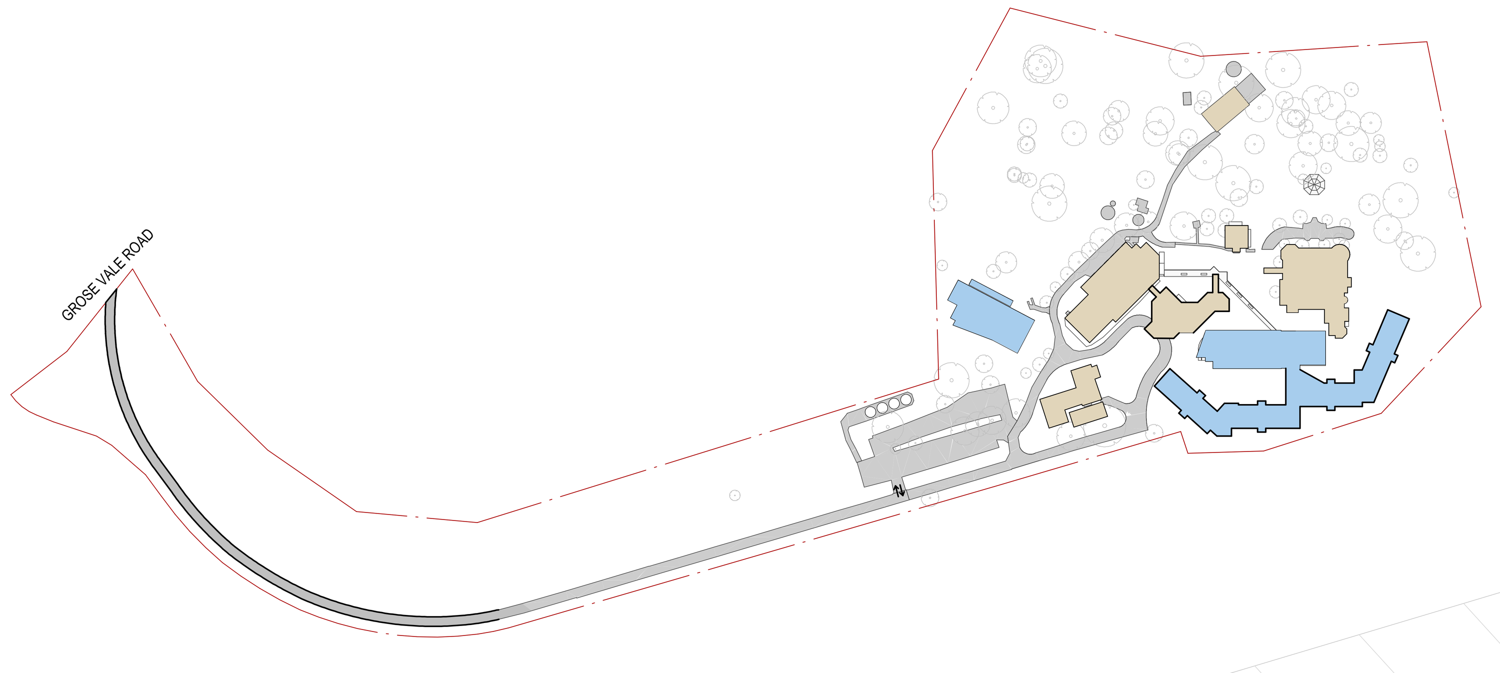
Job No: EP1494.002
Date: 13/02/2020
Drawing Ref: EP1494.002 Fig2_Cross Section
Version No: v1



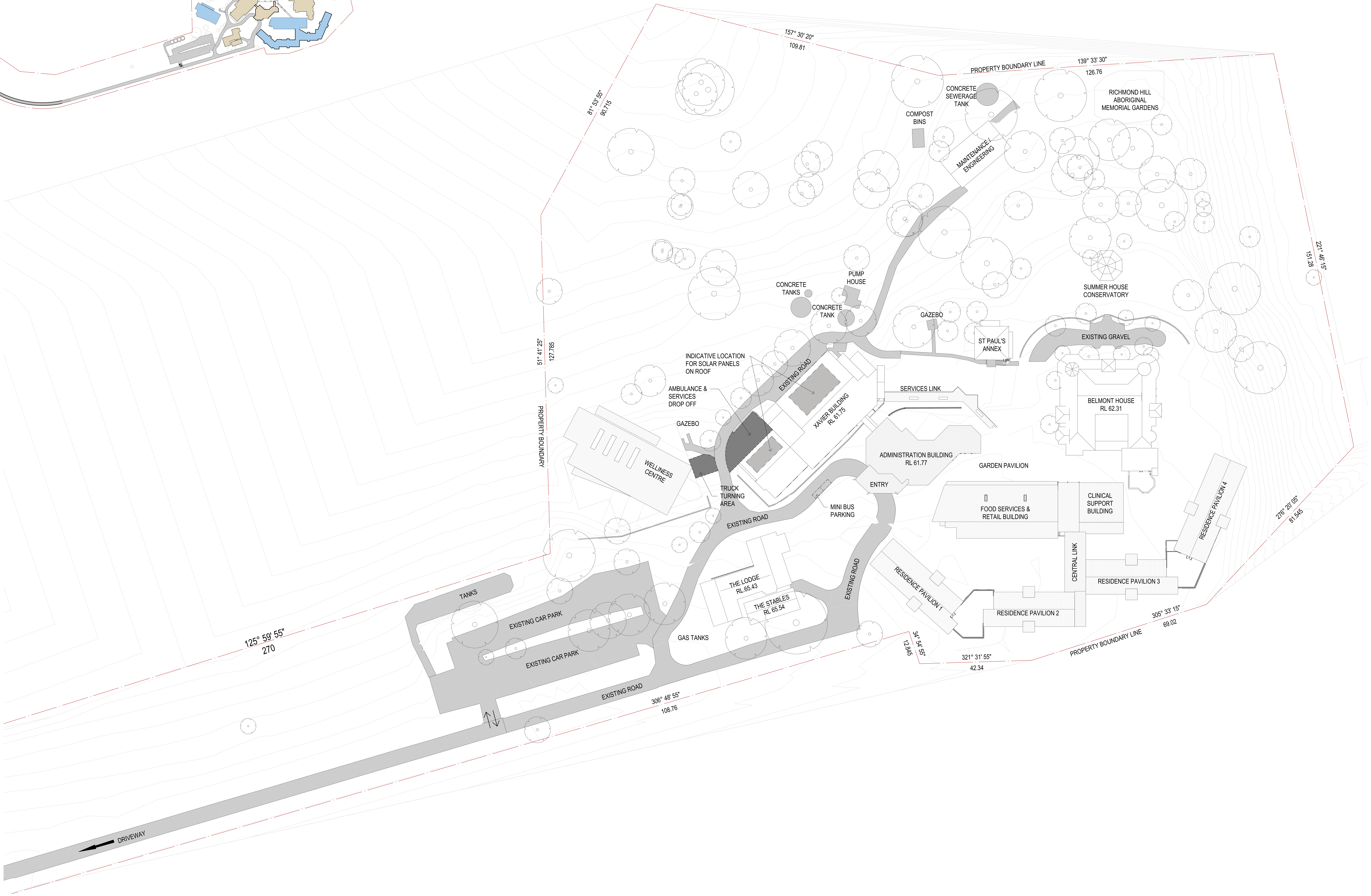
Co-ordinate system: MGA 56
Drawn by: NM Checked by: JY
Scale of regional map not shown

Appendix A

DEVELOPMENT PLANS



01 SITE CONTEXT
SCALE 1 : 2500



STH
SILVER THOMAS HANLEY

JOHNSTAFF

ST JOHN OF GOD Health Care

PROPOSED SITE PLAN

ST JOHN OF GOD RICHMOND HOSPITAL

Project Number **10444**

Scale @ A0 Sheet Size **As indicated**

Date **28.09.20**

Revision:

Drawing Number: **SSK03**

DATE: 28/09/2020 5:19:36 PM

Appendix B
SITE PHOTOGRAPHS

	**Plate 1** **Description:** Steep to extreme slopes and retaining wall on the eastern boundary of the Site. **Date:** 16/12/19
	**Plate 2** **Description:** The Chapel located in the central eastern portion of the Site. **Date:** 16/12/19

	**Plate 3** **Description:** Large underground water storage tank and pump house located in the central eastern portion of the Site. **Date:** 16/12/19
	**Plate 4** **Description:** Battle of Richmond Hill Memorial located in the northern portion of the Site. **Date:** 16/12/19

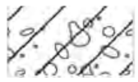
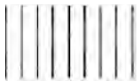
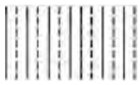
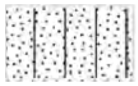
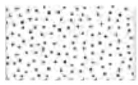
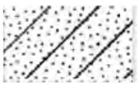
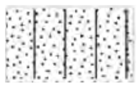
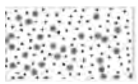
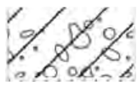
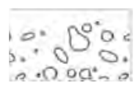
	**Plate 5** **Description:** Above ground diesel tank and chemical storage area located adjacent to the maintenance shed. **Date:** 16/12/19
	**Plate 6** **Description:** Open space area in the northern portion of the Site. **Date:** 16/12/19

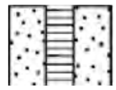
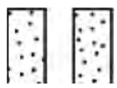
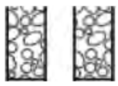
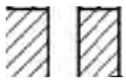
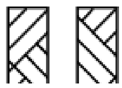
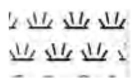
	**Plate 7** **Description:** Tennis courts located in the north west portion of the Site. **Date:** 16/12/19
	**Plate 8** **Description:** Xaviers building (left) and reception/admin building (right) in the central potion of the Site. **Date:** 16/12/19

	**Plate 9** **Description:** St Augustines (right) and CTC building (left) in the eastern portion of the Site. **Date:** 16/12/19
	**Plate 10** **Description:** Access road in the eastern portion of the Site. **Date:** 16/12/19

Appendix C
ENGINEERING LOGS

Soil Logging Symbols

CLAYS		Definition
	CLAY	USCS – CH
	silty CLAY	USCS – OH
	sandy CLAY	USCS – CL
	gravelly CLAY	USCS – GC
SILTS		
	SILT	USCS – ML
	clayey SILT	USCS – OL
	sandy SILT	USCS – SM
	gravelly SILT	USCS – GM
SANDS		
	SAND	USCS – SW
	clayey SAND	USCS – SC
	silty SAND	USCS – SM
	gravelly SAND	USCS – SP
GRAVELS		
	GRAVEL	USCS – GW
	clayey GRAVEL	USCS – GC
	silty GRAVEL	USCS – GM
	sandy GRAVEL	USCS – GP

SEDIMENTARY ROCK		Definition
	SANDSTONE	BGS - SNDST
	SILTSTONE	BGS - SLTST
	SHALE	BGS - SHALE
	CONGLOMERATE	BGS - CONG
FILL		
	FILL	OTHER – 01
	CONCRETE	BKFL-41
	ASPHALT	OTHER – 04
GROUNDWATER WELL SYMBOLS		
	WELL SCREEN	BKFL-31
	CASING – filter pack	BKFL-31
	CASING – backfill	BKFL-10
	CASING – bentonite seal	BKFL-22
	CASING – grout seal	BKFL-42
	BACKFILL	BKFL-10
OTHER		
	TOPSOIL – sandy SILT	OTHER – 05
	TOPSOIL – highly organic	USCS – PT

Rock Description Explanation Sheet (1 of 2)

Weathering Condition (Degree of Weathering):

The degree of weathering is a continuum from fresh rock to soil. Boundaries between weathering grades may be abrupt or gradational.

Rock Material Weathering Classification		
Weathering Grade	**Symbol**	**Definition**
Residual Soil	RS	Soil-like material developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume, but the material has not been significantly transported.
Extremely Weathered Rock	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded in water, but substance fabric and rock structure still recognisable.
Highly Weathered Rock	HW	Strong discolouration is evident throughout the rock mass, often with significant change in the constituent minerals. The intact rock strength is generally much weaker than that of the fresh rock.
Moderately Weathered Rock	MW	Modest discolouration is evident throughout the rock fabric, often with some change in the constituent minerals. The intact rock strength is usually noticeably weaker than that of the fresh rock.
Slightly Weathered Rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh Rock	FR	Rock shows no sign of decomposition or staining.

Notes:
1. Minor variations within broader weathering grade zones will be noted on the engineering borehole logs.
2. Extremely weathered rock is described in terms of soil engineering properties.
3. Weathering may be pervasive throughout the rock mass or may penetrate inwards from discontinuities to some extent.
4. The 'Distinctly Weathered (DW)' class as defined in AS1726-2017 is divided to incorporate HW and MW in the above table. The symbol DW should not be used.

Strength Condition (Intact Rock Strength):

Strength of Rock Material (Based on Point Load Strength Index, corrected to 50mm diameter – $I_{s(50)}$. Field guide used if no tests available. Refer to AS 4133.4.1-2007.				
Term	**Symbol**	**Point Load Index (MPa) $I_{s(50)}$**		**Field Guide to Strength**
Very Low	VL	>0.03	≤0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 3cm thick can be broken by finger pressure.
Low	L	>0.1	≤0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	M	>0.3	≤1.0	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	H	>1	≤3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	VH	>3	≤10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High	EH	>10		Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Notes:
1. These terms refer to the strength of the rock material and not to the strength of the rock mass which may be considerably weaker due to the effect of rock defects.
2. Anisotropy of rock material samples may affect the field assessment of strength.
3. Extremely Low Strength ('EL') is now not considered a description of rock strength in line with the updated AS1726-2017 as by definition EL rock should be described in terms of soil properties.

Rock Description Explanation Sheet (2 of 2)

Discontinuity Description: Refer to AS1726-2017, Table A10.

Anisotropic Fabric	
BED	Bedding
FOL	Foliation
LIN	Mineral lineation
Defect Type	
LP	Lamination Parting
Pt	Bedding Parting
FP	Cleavage / Foliation Parting
Jt	Joint
SZ	Sheared Zone
CZ	Crushed Zone
BZ	Broken Zone
HFZ	Highly Fractured Zone
AZ	Alteration Zone
VN	Vein

Roughness (e.g. Planar, Smooth is abbreviated Pln / Sm)		**Class**
Stepped (Stp)	Rough or irregular (R or Irr)	I
	Smooth (Sm)	II
	Slickensided (Sl)	III
Undulating (Un)	Rough (R)	IV
	Smooth (Sm)	V
	Slickensided (Sl)	VI
Planar (Pln)	Rough (R)	VII
	Smooth (Sm)	VIII
	Slickensided (Sl)	IX

Aperture		**Infilling**		
Closed	CD	No visible coating or infill	Clean	Cn
Open	OP	Surfaces discoloured by mineral/s	Stain	St
Filled	FL	Visible mineral or soil infill <1mm	Veneer	Vr
Tight	TI	Visible mineral or soil infill >1mm	Coating	Ct

Other	
Clay	Clay
Fe	Iron
Co	Coal
Carb	Carbonaceous
Sinf	Soil Infill Zone
Qz	Quartz
Ca	Calcite
Chl	Chlorite
Py	Pyrite
Int	Intersecting
Inc	Incipient
DI	Drilling Induced
H	Horizontal
V	Vertical

Note: Describe ‘Zones’ and ‘Coatings’ in terms of composition and thickness (mm).

Discontinuity Spacing: On the geotechnical borehole log, a graphical representation of defect spacing vs depth is shown. This representation takes into account all the natural rock defects occurring within a given depth interval, excluding breaks induced by the drilling / handling of core. Refer to AS1726-2017, BS5930-1999.

Defect Spacing			**Bedding Thickness (Sedimentary Rock Stratification)**	
Spacing/Width (mm)	Descriptor	Symbol	Descriptor	Spacing/Width (mm)
			Thinly Laminated	< 6
<20	Extremely Close	EC	Thickly Laminated	6 – 20
20 – 60	Very Close	VC	Very Thinly Bedded	20 – 60
60 – 200	Close	C	Thinly Bedded	60 – 200
200 – 600	Medium	M	Medium Bedded	200 – 600
600 – 2000	Wide	W	Thickly Bedded	600 – 2000
2000 – 6000	Very Wide	VW	Very Thickly Bedded	> 2000
>6000	Extremely Wide	EW		

Defect Spacing in 3D	
Term	**Description**
Blocky	Equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section

Defect Persistence (areal extent)
Trace length of defect given in metres

Symbols:

The list below provides an explanation of terms and symbols used on the geotechnical borehole, test pit and penetrometer logs.

Test Results			
PI	Plasticity Index	c'	Effective Cohesion
LL	Liquid Limit	c_u	Undrained Cohesion
LI	Liquidity Index	c'_R	Residual Cohesion
DD	Dry Density	ϕ'	Effective Angle of Internal Friction
WD	Wet Density	ϕ_u	Undrained Angle of Internal Friction
LS	Linear Shrinkage	ϕ'_R	Residual Angle of Internal Friction
MC	Moisture Content	c_v	Coefficient of Consolidation
OC	Organic Content	m_v	Coefficient of Volume Compressibility
WPI	Weighted Plasticity Index	$c_{\alpha\varepsilon}$	Coefficient of Secondary Compression
WLS	Weighted Linear Shrinkage	e	Voids Ratio
DoS	Degree of Saturation	ϕ'_{cv}	Constant Volume Friction Angle
APD	Apparent Particle Density	q_t / q_c	Piezocone Tip Resistance (corrected / uncorrected)
s_u	Undrained Shear Strength	q_d	PANDA Cone Resistance
q_u	Unconfined Compressive Strength	$I_{s(50)}$	Point Load Strength Index
TCR	Total Core Recovery	*RQD*	Rock Quality Designation

Test Symbols	
DCP	Dynamic Cone Penetrometer
SPT	Standard Penetration Test
CPTu	Cone Penetrometer (Piezocone) Test
PANDA	Variable Energy DCP
PP	Pocket Penetrometer Test
U50	Undisturbed Sample 50 mm (nominal diameter)
U100	Undisturbed Sample 100mm (nominal diameter)
UCS	Uniaxial Compressive Strength
Pm	Pressuremeter
FSV	Field Shear Vane
DST	Direct Shear Test
PR	Penetration Rate
PLI	Point Load Index Test (axial)
D	Point Load Test (diametral)
L	Point Load Test (irregular lump)

Groundwater level	Water Inflow	Water Outflow

EP Risk Management
3/19 Bolton Street
Newcastle NSW 2300
Telephone: 02 4048 2845

BOREHOLE NUMBER BH1

CLIENT St John of God Health Care
PROJECT NAME PSI and Geotechnical Investigation
PROJECT NUMBER EP1494
PROJECT LOCATION 235 Grose Vale Road, North Richmond, NSW

DATE STARTED 19/12/19 **COMPLETED** 19/12/19 **R.L. SURFACE** 63.5 **DATUM** m AHD
DRILLING CONTRACTOR Terratest **SLOPE** 90° **BEARING** -
EQUIPMENT Geo Probe **HOLE LOCATION**
HOLE SIZE **LOGGED BY** NM **CHECKED BY** JY
NOTES

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
							ASPHALT: 30 mm.		
						GW	Sandy GRAVEL: Brown, dry, medium to coarse sub angular gravel, basecoarse.	Environmental Sample	
	-		63.0	0.5			XW SHALE: Brown, dry, laminated, recovered as gravelly silt.	Environmental Sample	
			62.5	1.0				Environmental Sample	
			62.0	1.5				Environmental Sample	
							Borehole BH1 continued as cored hole		
			61.5	2.0					