Environmental - Remediation - Engineering - Laboratories - Drilling

GEOTECHNICAL INVESTIGATION REPORT

Nos. 160-178 Stoney Creek Road Beverly Hills NSW 2209

Prepared for

Cuzeno Pty Ltd

Report No. GS6759-1A

27th January 2017

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REFERENCES

1. Australian Standard – AS 1726-1993 Geotechnical Site Investigation.
2. Australian Standard – AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake actions in Australia.
3. Australian Standard – AS3798-2007 Guidelines on Earthworks for Commercial and Residential Developments.
4. Australian Standard – AS 2870-2011 Residential slabs and footings.
5. Australian Standard – AS 2159-2009 Piling - Design and installation.
6. Pells P.J.N, Mostyn, G. & Walker B.F., “Foundations on Sandstone and Shale in the Sydney Region”, Australian Geomechanics Journal, 1998.

1. INTRODUCTION

Aargus Pty Ltd (Aargus) has been commissioned by Cuzeno Pty Ltd to carry out a geotechnical site investigation at nos. 160-178 Stoney Creek Road, Beverly Hills, NSW 2209. The site investigation was carried out on the 16th and 17th January 2017 and was followed by geotechnical interpretation, assessment and preparation of a geotechnical report.

The purpose of the investigation was to assess the ground conditions and feasibility, from a geotechnical perspective, of the site for a proposed residential development.

This report presents results of the geotechnical site investigation, laboratory testing, interpretation, and assessment of the site existing geotechnical conditions, as a basis to provide recommendations for design and construction of ground structures for the proposed development.

To assist in reading the report, reference should be made to the “Important Information About Your Geotechnical Report” attached as Appendix A.

2. AVAILABLE INFORMATION

Prior to preparation of this report, the following information was made available to Aargus:

- Preliminary Architectural drawings titled “160-178 Stoney Creek Road, Beverly Hills” prepared by Candalepas Associates, referenced Job No. 5728 and included drawing nos:
    - DA-1000;
    - DA-1101 to DA-1108 inclusive;
    - DA-1201 to DA-1203 inclusive;
    - DA-1301 and DA-1302; and
    - DA-1850 and DA-1851.
- Site Survey Plan for “160-178 Stoney Creek Road Beverly Hills” prepared by Stuart De Nett Land Surveyors, and referenced No. 11025C and No. 11025D.

3. SCOPE OF WORK

In accordance with the brief, fieldwork for the geotechnical site investigation was carried out by an experienced Geotechnical Engineer from Aargus; following in general the guidelines provided in Australian Standard AS 1726-1993 (Reference 1) and comprised the following:

- Collection and review of Dial-Before-You-Dig (DBYD) plans;
- A site walkover inspection in order to determine the overall surface conditions and to identify any relevant site features;
- Service locating using electromagnetic detection equipment to ensure that the investigation area is free from underground services;
- Machine drilling of four (4) borehole identified as BH1 to BH4 inclusive, using a Truck Mounted Drilling Rig owned and operated by a subcontractor;

- Standard Penetration Tests (SPT) was conducted within the machine drilled boreholes to assess the in-situ strength of subsurface soil layers;
- Collection of soil samples during drilling;
- Installation of one (1) standpipe piezometer in borehole BH2 and identified as GW1, to assess the groundwater conditions; and
- Reinstatement of the borehole with soil cuttings generated from the auger drilling and excavation process.

The approximate location of the boreholes completed during the geotechnical site investigation are shown on "Figure 1 - Site Plan" attached in Appendix B.

Boreholes BH1 to BH4 inclusive were augered to Tungsten Carbide (TC) refusal and terminated depths ranging from approximately 4.5m to 8.6m below ground level (bgl). Following refusal and terminated depths, boreholes BH2 and BH4 were continued using NMLC to terminated depths of approximately 9.40m and 14.12m bgl, respectively.

Following completion of the site investigation, laboratory testing was carried out on selected rock core samples recovered from the borehole, and consisted of:

- Point Load Index testing on five (5) selected rock cores.

Based on the results of the site investigation and laboratory testing, Aargus carried out geotechnical interpretation and assessment of the main potential geotechnical issues that may be associated with the proposed development. A geotechnical report (this report) was prepared to summarise the results of the geotechnical site investigation and to provide comments and recommendations relating to:

- Excavation conditions;
- Stability of basement excavation;
- Suitable foundations;
- Allowable bearing pressure (and shaft adhesion for piles);
- Lateral pressure for design of retaining walls;
- Groundwater; and
- Site earthquake classification.

4. SITE DESCRIPTION

The site is an irregular rectangular shaped land with an approximate area of 3,780m^2, and consists of an amalgamation of properties, being nos. 160-178 Stoney Creek Road.

At the time of the investigation, a two storey brick commercial building was present within the property nos. 160-166, and was accompanied by associated concrete slabs. The remaining site area, being the property No. 178, was a vacant land with a number of mature trees and vegetation scattered throughout.

The site is located within the Hurstville City Council area, at the intersection of King Georges Road and Stoney Creek Road, which are major road reserves within the local area. The site is also located approximately 200m to the west of a stormwater drain. The site is bounded by the following properties, public roads and infrastructure:

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- Residential dwellings and a Lane Way for car access to the north of the site;
- Lee Avenue carriageway and road reserve to the east of the site;
- Stoney Creek Road carriageway and road reserve to the south of the site; and
- Commercial buildings and King Georges Road carriageway and road reserve to the west.

The site topography during the investigation was generally level with a gently slope towards the north to north-east. The local topography was also generally level with a gentle sloping towards the north to north-east.

5. PROPOSED DEVELOPMENT

The architectural drawings (referenced in Section 2) indicate the proposed development consists of the demolition of the existing buildings, and the construction of a three to five storey building, overlying three underground basement levels. Vehicular access to the basements will be via a ramp from Lee Avenue along the site eastern boundary.

The elevation of the proposed lower basement level is 22.06m Australian Height Datum (AHD). Maximum excavation depths of approximately 12.0m to 14.0m will be required for the proposed three basement levels (varying within the site).

The proposed lift shafts within the building are expected to require a further 1.5m of excavation below the basement Finished Floor Level (FFL).

6. SUBSURFACE CONDITIONS

6.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 Edition 1, dated 1983, by the Geological Survey of New South Wales, Department of Mineral Resources, indicates the site is located at a geological boundary underlain by Triassic Age Ashfield Shale (Rwa). The Ashfield Shale is described as “black to dark-grey shale and laminate”.

Assessment of the subsurface materials, discussed in Section 6.2, confirms the published geology.

It should be noted this geological profile does not take into account the residual soils derived from in-situ weathering of the bedrock, or the presence of fill that may have been generated from previous earthworks.

6.2 Ground Profile

The subsoil conditions encountered within the boreholes are summarised in Table 1 and detailed on the attached Engineering Borehole Logs presented in Appendix C with Core Photographs and Results of Point Load Index Test in Appendix D and Appendix E, respectively. It should be noted that reference should be made to the logs and/or specific test results for design purposes.

Table 1: Summary of Subsurface Conditions

Unit	Description	BH1 (m)	BH2 (m)	BH3 (m)	BH4 (m)
	Ground Surface Level (m AHD)	***RL35.4***	***RL36.5***	***RL34.8***	***RL33.4***
Fill	Sandy Gravelly CLAY, low to medium plasticity, brown, fine to coarse grained gravel, fine grained sand.	0.0 – 1.5	0.0 – 1.4	0.0 – 1.6	0.1 – 0.4
	Sandy CLAY, medium plasticity, brown, some fine grained rounded gravel, fine grained sand, firm to stiff.	1.5 – 3.2	–	1.6 – 2.2	–
Residual Soils	Silty CLAY, medium to high plasticity, brown to grey, mottled red, some fine grained ironstone gravel, firm to stiff,	–	1.4 – 2.4	–	0.4 – 1.3
	Silty CLAY, medium to high plasticity, pale grey, some fine to coarse grained ironstone gravel, interbedded with shale, firm to stiff.	–	2.4 – 3.0	2.2 – 2.6	1.3 – 1.8
Bedrock[1]	SHALE, pale grey to brown, extremely weathered, extremely low strength. Class V Shale.	3.2 – 8.6+	3.0 – 4.5	2.6 – 6.5+	1.8 – 8.0
	SHALE, grey to brown, occasional pale grey laminations, extremely to highly weathered, low strength. Class V Shale.	–	4.5 – 6.6	–	10.2 – 11.8[2]
	SHALE, dark grey, pale grey laminations, highly to moderately weathered, low to medium strength. Class IV Shale.	–	6.6 – 9.4+	–	8.0 – 10.2
	SHALE, dark grey, pale grey laminations, highly to moderately weathered, low to medium strength. Class III Shale.	–	–	–	11.8 – 14.12+

[1]Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998 (Reference 6).
[2]Coreloss encountered during augering in borehole BH4 at depths of approximately 10.2m to 11.8m may be associated with a layer of extremely weathered, extremely low strength, Class V Shale. This layer may be formed due to poorly consolidated Shale bedrock, and has been washed away during coring within the borehole.

It is understood that the deep fill encountered within the site, (predominately boreholes BH1 and BH3), may be associated with the decommissioning of a previously existing petrol station located within the site boundaries. The fill material may have been placed after the removal of underground petrol tanks, and could therefore be associated with the strong fuel odour emanating from the fill (BH1 and BH3).

6.3 Groundwater

Groundwater was not encountered during augering in boreholes BH1 to BH4 inclusive. Measurement of water levels during core drilling in boreholes BH2 and BH4, below the depths achieved by augering within the boreholes was not possible due to the introduction of water required for coring.

Water levels measured on the 23rd January 2016 in piezometer GW1 installed in borehole BH2 indicate that groundwater was present at a depth of approximately 8.4m (RL28.1m AHD).

It is inferred that natural groundwater levels may be in the form of seepage through fissures and natural defects in the underlying weathered bedrock. Further, it should be noted that groundwater levels may be subject to seasonal and daily fluctuations influenced by factors such as rainfall and future development of the surrounding lands. Soil moisture within the site may be influenced by events within the property and the adjoining road and properties such as breakage of water mains, or stormwater pipes.

7. GEOTECHNICAL ASSESSMENT

7.1 General

Piezometer readings indicate groundwater to be present at approximately 8.4m (RL28.1m AHD) depth. Based on a basement floor level of 22.06m AHD, it is considered that the groundwater level is approximately 6.0m above the lower basement level and would be within extremely weathered, extremely low strength Shale bedrock.

Consideration needs to be given to specific geotechnical issues including excavation stability, foundation conditions and temporary shoring. Geotechnical commentary regarding these geotechnical constraints and recommendations for the proposed development is presented in the following sections.

7.2 Excavation Conditions

The observations made during the investigation indicate the existing buildings and associated concrete slabs cover approximately one third of the footprint of the proposed development. Excavation is expected to be through fill, residual soils and then into shale bedrock of generally extremely low and very low to medium strength.

Excavation within the soils and extremely low to low strength bedrock is expected to be readily achieved using a large hydraulic excavator down to the level of medium or stronger bedrock. However, localised use of rock breaking equipment or ripping may be required where high strength bands are encountered.

For medium or greater strength rock, excavation will require the use of heavy ripping and/or hydraulic rock hammers. Excavation for foundations or trenches will require the use of hydraulic hammers and possibly a rock saw. Both noise and vibration will be generated by the proposed excavation work within these bedrock materials.

The rock classification system in Table 1 should not be used to directly assess rock excavation characteristics. Contractors should refer to the engineering logs, core photographs and point load tests when assessing the suitability of their excavation equipment.

7.3 Vibration Control

It is recommended that a vibration monitoring plan is developed to monitor the potential vibration effects during excavation and from the demolition works, on existing buildings within adjoining properties and road reserves and carriageways along the site boundary.

To ensure vibration levels remain within acceptable levels and to minimise the potential effects of vibration, if required, excavation into medium strength bedrock or stronger should be complemented with saw cutting or other appropriate methods prior to excavation. Rock saw cutting should be carried out using an excavator mounted rock saw, or similar, so as to minimise transmission of vibrations to any adjoining properties that may be affected. Hammering is not recommended and should be avoided. However, if necessary, hammering should be carried out horizontally along bedding planes of (pre-cut) broken rock blocks or boulders where possible and at the required operational limit to ensure noise levels are restricted to limits acceptable to adjacent residents.

Recommended Maximum Peak Particle Velocity (PPV) for different types of building or structure is summarised in Table 2. Induced vibrations in structures adjacent to the excavation should not be exceeded.

Table 2: Recommended Maximum Peak Particle Velocity

Type of Building or Structure	Max. PPV (mm/sec)
Historical or structures in sensitive conditions	2
Residential and low rise buildings	5
Brick or unreinforced structures in good condition	10
Commercial and industrial buildings or structures of reinforced concrete or steel construction.	25

It is recommended that monitoring is carried out during excavation using a vibration monitoring instrument (seismograph) and alarm levels (being the appropriate PPV) selected in accordance with the type of structures present within the zone of influence of the proposed excavation.

If vibrations in adjacent structures exceed the above values or appear excessive during construction, excavation should cease and the project Geotechnical Engineer should be contacted immediately for appropriate reviews.

It is recommended a dilapidation survey of the existing buildings within adjoining properties and infrastructure is conducted. Preparation of dilapidation survey report and vibration monitoring plan together with vibration monitoring should constitute as "Hold Points".

7.4 Stability of Excavation

As excavation of the proposed basement will extend up to a maximum of approximately 14.0m depth (varying within the site) and due to the generally close proximity of the basement with the boundaries, the use of temporary batter slopes is unsuitable, and therefore temporary shoring should be provided. Shoring design should consider both short term (construction) and permanent conditions as well as the presence of adjacent buildings and roads.

Based on the ground conditions encountered and the requirements of the proposed development, we recommend a contiguous pile wall solution socketed into the underlying Class III Shale bedrock to at least 1.0m below basement level to prevent 'kick-out' of the pile toe. The use of contiguous pile walls may allow for small gasp between the piles, which could permit groundwater inflow during excavation. The use of strip drains behind the piles and shotcreting in weak areas susceptible to inflow during excavation can limit the amount of groundwater ingress. All vertical drains should be connected to a perimeter drain provided at the toe of the final excavation, which should discharge to the site stormwater system to provide long term drainage behind excavation walls.

Alternatively, a secant pile wall retaining system, which creates a near impervious barrier and significantly inhibits groundwater seepage during excavation, may also be considered. A suitably designed secant wall will therefore 'cut-off' any groundwater seepage flowing into the excavation, to provide a relatively dry working area. A sump and pump drainage system is still likely to be required to control surface water run-off and any minor inflow.

For the maximum retained height being considered, a temporary anchorage system is likely to be required to provide lateral support during construction. As two or more rows of anchors are likely to be required to support the shoring due to significant retained height or where significant lateral movements cannot be tolerated (e.g. due to adjacent infrastructure), the shoring/basement wall should be designed as a braced structure. Anchor designs should be based on allowing effective bonding to be developed behind an 'active zone' determined by drawing a line at 45° from the base of the wall to intersect the ground surface behind the excavated face. It is considered that basement floor slabs will provide permanent restraint to the retaining walls where these are incorporated into the permanent works. Anchors are therefore considered to be temporary but depending on the sensitivity of the adjacent infrastructure, it may be necessary to incorporate the temporary anchors into the permanent works to control deflections.

Anchor installation beyond the property boundaries will be subject to approval by owners of adjoining properties, roads and infrastructure. Where an anchorage system is shown to be impractical, consideration of other temporary support options would be necessary. These options include the following:

- Temporary solutions such as installation of props associated with staged excavation; and
- Staged excavations and temporary partial berms in front of walls.
- Top-down construction where floor slabs and beams are constructed at the top of shoring wall and at floor levels of the upper basement levels prior to excavation within the basement level underneath the floor slabs.

The shoring wall and anchors can be designed using the recommended parameters provided in Section 7.5 below.

Detailed design of anchored or propped retaining walls should utilise commercial software packages such as WALLAP or PLAXIS that can model the sequence of anchor installation and excavation to ensure deflections are within tolerable limits. The design of retaining structures should to take into account horizontal pressures due to surcharge loads from any adjacent infrastructure.

A dilapidation survey will be required prior to excavation for the existing buildings within the adjoining properties and the section of road carriageway and road reserve adjoining the site.

Detailed construction supervision, monitoring and inspections will be required during piling and subsequent bulk excavation and should be carried out by an experienced Geotechnical Engineer, in addition to inspection of the structural elements by the Project Structural Engineer. The inspections should constitute as “Hold Points”.

7.5 Earth Pressures

Earth retaining structures should be designed to withstand the lateral earth pressure, hydrostatic and earthquake (if applicable) pressures, and the applied surcharge loads in their zone of influence, including existing structures, traffic and construction related activities.

For the design of flexible retaining structures, where some lateral movement is acceptable, it is recommended the design should be based on active lateral earth pressure. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient “at rest” should be considered such as the case when the shoring wall is in the final permanent state and is restrained by the concrete slab in its final state.

Recommended parameters for the design of earth retaining structures in the soils and rock horizons underlying the site are presented in Table 3.

Table 3: Preliminary Geotechnical Design Parameters for Retaining Walls

Units	Unit Weight (kN/m^3)	Effective Cohesion c' (kPa)	Angle of Friction ϕ' (°)	Modulus of Elasticity E_{sh} (MPa)
Fill	19	0	26	8
Residual Soils	20	5	24	15
Class V Shale	22	25	27	75
Class IV Shale	22	50	28	250
Class III Shale	23	100	30	400

Table 4 below provides preliminary coefficients of lateral earth pressure for the soils and rocks encountered during the geotechnical investigation. The coefficients provided are based on horizontal ground surface and fully drained conditions.

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Table 4: Preliminary Coefficients of Lateral Earth Pressure

Units	Coefficient of Active Lateral Earth Pressure Ka	Coefficient of Active Lateral Earth Pressure at Rest Ko	Coefficient of Passive Lateral Earth Pressure Kp
Fill	0.39	0.56	2.56
Residual Soils	0.42	0.59	2.37
Class V Shale	0.3	0.5	3.0
Class IV Shale			
Class III Shale	0.2	0.5	5.0

- If present, adverse jointing systems in the rock may result in higher active earth pressures than those outlined above. Potential areas of block or wedge failure should therefore be identified during construction and appropriate stabilization measures adopted.
- As Sydney rocks can often experience high lateral pressures, consideration may be given to adopting an earth pressure at rest K_o = 2.0 in the shale bedrock as part of a sensitivity analysis during detailed design.
- Higher earth pressures (K=1) will apply for undrained (temporary) clay soils.
- Coefficient of active and passive lateral earth pressure K_a and K_p, respectively, can be calculated using Rankine's or Coulomb's equations, as appropriate.
- Coefficient of lateral earth pressure at rest K_o for soils, can be calculated using Jacky's equation.

The coefficients of lateral earth pressure should be verified by the project Structural Engineer prior to use in the design of retaining walls. Simplified calculations of lateral active (or at rest) and passive earth pressures can be carried out for cantilever walls using Rankine's equation shown below:

$P_a = K\,\gamma\,H - 2c\sqrt{K}$ For calculation of lateral active or 'at rest' earth pressure

$P_p = K_p\,\gamma\,H + 2c\sqrt{K_p}$ For calculation of passive earth pressure

For braced retaining walls, a uniform lateral earth pressure should be adopted as follows:

$P_a = 0.65\,K\,\gamma\,H$ For calculation of active earth pressure

Where;

P_a = Active (or at rest) Earth Pressure (kN/m^2)
P_p = Passive Earth Pressure (kN/m^2)
γ = Bulk density (kN/m^3)
K = Coefficient of Earth Pressure (K_a or K_o)
K_p = Coefficient of Passive Earth Pressure
H = Retained height (m)
c = Effective Cohesion (kN/m^2)

If adopted, temporary anchors will require embedment in bedrock. Preliminary allowable bond stresses may be adopted for temporary anchors, as detailed in Table 5 below.

Table 5: Preliminary Allowable Bond Stress for Temporary Anchors

Units	Allowable Bond Stress (kPa)
Class V Shale	50
Class IV Shale	100
Class III Shale	150

Anchors should undergo proof testing following installation. The anchors can be designed for the parameters recommended above providing:

- The bond (socket) length at least 3.0m; and
- Anchors are proof tested to 1.3 times the design working load specified by the Structural Engineer, before they are locked off at working load. Anchor testing should constitute as a "Hold Point".

7.6 Subgrade Preparation and Earthworks

The following general procedure is provided for site preparation of building platforms and pavements:

- Strip topsoil and remove any unsuitable material from site.
- Excavate fill, residual soils and rock stockpiling for re-use as engineered fill or remove to spoil.
- Where clayey soil is exposed at formation level, the exposed surface should be treated and moisture conditioned to within 2% of optimum moisture content (OMC) followed by proof rolling with a smooth drum roller. Soft or loose areas should be excavated and replaced with approved fill material.
- Where rock is exposed at footing level, it should be free of loose or softened material.

The suitability of imported materials for filling should be subject to the following criteria:

- The materials should be clean (i.e. free of contaminants, deleterious or organic material), free of inclusions of >120mm in size; high plasticity material and soft material be removed and suitably conditioned to meet the design assumptions where fill material is proposed to be used.
- Material with excessive moisture content should not be used without conditioning.
- The materials should satisfy the Australian Standard AS 3798-2007 (Reference 3).

The final surface levels of all cut and fill areas should be compacted in order to enable the subgrade to achieve adequate strength for the proposed building platforms.

For the fill construction, the recommended compaction targets should be the following:

- Moisture content of ±2% of OMC (Optimal Moisture Content);
- Minimum density ratio of 98% of the maximum dry density for the building platforms of the proposed dwellings;
- The loose thickness of layer should not exceed 300mm during the compaction.

Design and construction of earthworks should be carried out in accordance with Australian Standard AS 3798-2007 (Reference 3). Inspections by the project Geotechnical Engineer will be required during earthworks, subgrade preparation and proof rolling. The inspections should constitute as “Hold Points”.

7.7 Foundations

Bulk excavation is mainly likely to expose variable strength bedrock potentially comprising Class IV to Class III Shale. Suitable footings are therefore likely to comprise cast in-situ reinforced concrete raft foundation with thickened slab footings to support internal columns and walls, respectively. A stiffened raft slab would distribute the applied load of the building over the bedrock underlying the slab, with some of the stresses being distributed through the basement walls into the underlying bedrock.

However, given the potential for variable strength bedrock at bulk excavation level, it is recommended that all footings be founded on consistent bedrock. This could be achieved by strip or pad footings where suitable bedrock is exposed at bulk excavation level and pile foundations elsewhere. Installation of piles is expected to be required in cases where axial loads on columns and walls exceed the bearing pressure of the bedrock present at bulk excavation level.

Other cases where piles may be required include the need to increase the resistance against lateral seismic and wind loads. Design of shallow and pile foundations should be carried out in accordance with Australian Standards AS2870-2011 (Reference 4) and AS2159-2009 (Reference 5), respectively.

Table 6 provides geotechnical parameters recommended for design of shallow and piled foundations.

Table 6: Preliminary Geotechnical Foundation Design Capacities

Unit	Allowable Capacity Values (kPa)	
	End Bearing Pressure[1]	**Shaft Adhesion Compression (Tension)**[2]
Fill	N/A[3]	N/A[3]
Residual Soils	100	N/A[3]
Class V Shale	700	25 (15)
Class IV Shale	1,000	50 (25)
Class III Shale[4]	2,500	200 (100)

[1] With a minimum embedment depth of 0.5m for deep foundations and 0.4m for shallow foundations.
[2] Clean rock socket of roughness of at least grooves of depth 1mm to 4mm and width greater than 5mm at spacing of 50mm to 200mm.Shaft Adhesion in Tension is 50% of Compression, applicable to piles only.
[3] N/A, Not Applicable, not recommended for the proposed building of this development.
[4] The actual depth of the underlying Class III Shale should be confirmed during construction if required.

Shaft adhesion may be applied to socketed piles adopted for foundations provided socket shaft lengths conform to appropriate classes of Shale and accepted levels of shaft sidewall

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cleanliness and roughness. The rock socket sidewalls should be free of soil and/or crushed rock to the extent that natural rock is exposed over at least 80% of the socket sidewall. Shaft adhesion should be reduced or ignored within socket lengths that are smeared and fail to satisfy cleanliness requirements. Additional attention to cleanliness of socket sidewalls may be required where presence of clay seams and weathered Shale bands is evident over socket lengths. Where the piles penetrate soils that are susceptible to shrinkage and swelling, we recommend that the shaft adhesion be ignored in the zone of seasonal moisture variations due to the potential of shrinkage cracking.

Due to expected groundwater levels, bored piles may require dewatering. Some over break and fretting should be allowed for. Continuous flight auger (CFA) piles may be considered as a suitable alternative to bored piles in the case of elevated groundwater levels.

The excavations should be dewatered prior to concrete pouring if groundwater seepages or surface runoff be encountered within foundation excavations. Any loose debris and wet soils should also be removed from excavations.

An experienced Geotechnical Engineer should review footing designs to ensure compliance with the recommendations in the geotechnical report and assess foundation excavations to ensure suitable materials of appropriate bearing capacity have been reached. The presence of water within foundation excavations may negate satisfactory examination of founding surfaces and certification of founding materials quality. Foundation inspections should only be undertaken under conditions satisfying WHS requirements.

Verification of the capacity of the shallow and pile foundations by inspections would be required and inspections should constitute as “Hold Points”.

7.8 Groundwater Management

As the proposed excavation is expected to be approximately 6.0m below groundwater level with the potential for higher groundwater levels resulting from heavy rainfall, flooding or damaged services, etc., consideration should be given to seepage flows through soils and weathered bedrock during excavation or in the long term during the design life of the building. It would therefore be prudent to give consideration to precautionary drainage measures in the design and construction of the proposed development. Such measures could include the following:

- Strip drains or drainage materials should be installed behind the shoring/retaining walls in conjunction with collection trenches or pipes and pits connected to the building stormwater system. A temporary storage tank and pump system may be required.
- Depending on the groundwater inflow rate during excavation, groundwater seepage and surface water infiltration may be controlled by a sump and pump methods during construction.
- Consideration may also be given to waterproofing of basement slabs and walls with appropriate allowance for nominal hydrostatic uplift.

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It should be noted that groundwater behaviour may be influenced by the seasonal variations in groundwater level resulting from heavy rainfall, flooding, damaged services, etc.

7.9 Stoney Creek Road and King Georges Road – RMS Requirements

Construction of the proposed development will require excavation adjacent to Stoney Creek Road and King Georges Road which are both Roads and Maritime Services (RMS) Assets. In accordance with the requirements of the 'RMS Technical Direction "Geotechnology" GTD 2012/001' dated 27th April 2012, the following items, which is not exhaustive, should be considered as part of the overall geotechnical strategy in satisfying the requirements of the RMS and providing reliable geotechnical data for detailed design purposes:

- The RMS may require a dilapidation survey of any assets located within the zone of influence of the investigation and may include road pavement, subsurface drainage, traffic signal structures and other road assets.
- Instrumentation and monitoring plan may be required to monitor any movements that may occur as a result of the excavation. This would include trigger levels and action to be taken when trigger levels are exceeded.
- Comprehensive Geotechnical investigation and laboratory testing should be undertaken to provide geotechnical material parameters for detailed design. This should also include an assessment of the rock stress state and its effect on the excavation. Boreholes will be required adjacent to Stoney Creek and King Georges Road for subsequent analysis.
- Geotechnical analysis using appropriate geotechnical software (WALLAP or PLAXIS) is required for the prediction of wall deflections for each stage of the construction.

7.10 Preliminary Site Earthquake Classification

The results of the site investigation indicate the presence of fill and residual soil extending to a maximum depth of about 3.2m (varying within the site), and underlain by extremely low to very low strength Class V Shale, low to medium strength Class IV Shale and medium strength Class III Shale. In accordance with Australian Standard AS 1170.4-2007 (Reference 2) the site may be classified as a "Shallow soil site" (Class C_e) for design of foundations and retaining walls embedded in the underlying soils and weathered Shale. The Hazard Factor (Z) for Sydney in accordance with AS 1170.4-2007 is considered to be 0.08.

8. LIMITATIONS

The geotechnical assessment of the subsurface profile and geotechnical conditions within the proposed development area and the conclusions and recommendations presented in this report have been based on available information obtained during the work carried out by Aargus and in the provided documents listed in Section 2 of this report. Inferences about the nature and continuity of ground conditions away from and beyond the locations of field exploratory tests are made, but cannot be guaranteed.

It is recommended that should ground conditions including subsurface and groundwater conditions, encountered during construction and excavation vary substantially from those presented within this report, Aargus Pty Ltd be contacted immediately for further advice and any necessary review of recommendations. Aargus does not accept any liability for site conditions not observed or accessible during the time of the inspection.

This report and associated documentation and the information herein have been prepared solely for the use of **Cuzeno Pty Ltd** and any reliance assumed by third parties on this report shall be at such parties' own risk. Any ensuing liability resulting from use of the report by third parties cannot be transferred to Aargus Pty Ltd, directors or employees.

For and on behalf of

Aargus Pty Ltd

Joe Nader
BE (Civil - Construction), Dip.Eng.Prac., GradIEAust
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National Engineering Manager

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APPENDIX A

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

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IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include the general nature of the structure involved, its size and configuration, the location of the structure on the site and its orientation, physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program.

To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should NOT be used:*

- when the nature of the proposed structure is changed: for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an un-refrigerated one,
- when the size or configuration of the proposed structure is altered,
- when the location or orientation of the proposed structure is modified,
- when there is a change of ownership, or

for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

Geotechnical reports present the results of investigations carried out for a specific project and usually for a specific phase of the project. The report may not be relevant for other phases of the project, or where project details change.

The advice herein relates only to this project and the scope of works provided by the Client.

Soil and Rock Descriptions are based on AS1726-1993, using visual and tactile assessment except at discrete locations where field and/or laboratory tests have been carried out. Refer to the attached terms and symbols sheets for definitions.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact. For this reason, most experienced owners retain their geotechnical consultants through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions, and thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

Subsurface conditions can change with time and can vary between test locations. Construction activities at or adjacent to the site and natural events such as flood, earthquake or groundwater fluctuations can also affect the subsurface conditions.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems.

No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professional develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

The interpretation of the discussion and recommendations contained in this report are based on extrapolation/interpretation from data obtained at discrete locations. Actual conditions in areas not sampled or investigated may differ from those predicted

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimise the likelihood of boring log misinterpretation, give contractors ready access in the complete geotechnical engineering report prepared or authorized for their use. Those who do not provide such access may proceed under mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are not exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

FURTHER GENERAL NOTES

Groundwater levels indicated on the logs are taken at the time of measurement and may not reflect the actual groundwater levels at those specific locations. It should be noted that groundwater levels can fluctuate due to seasonal and tidal activities.

This report is subject to copyright and shall not be reproduced either totally or in part without the express permission of the Company. Where information from this report is to be included in contract documents or engineering specifications for the project, the entire report should be included in order to minimise the likelihood of misinterpretation.

APPENDIX B

SITE PLAN (FIGURE 1)

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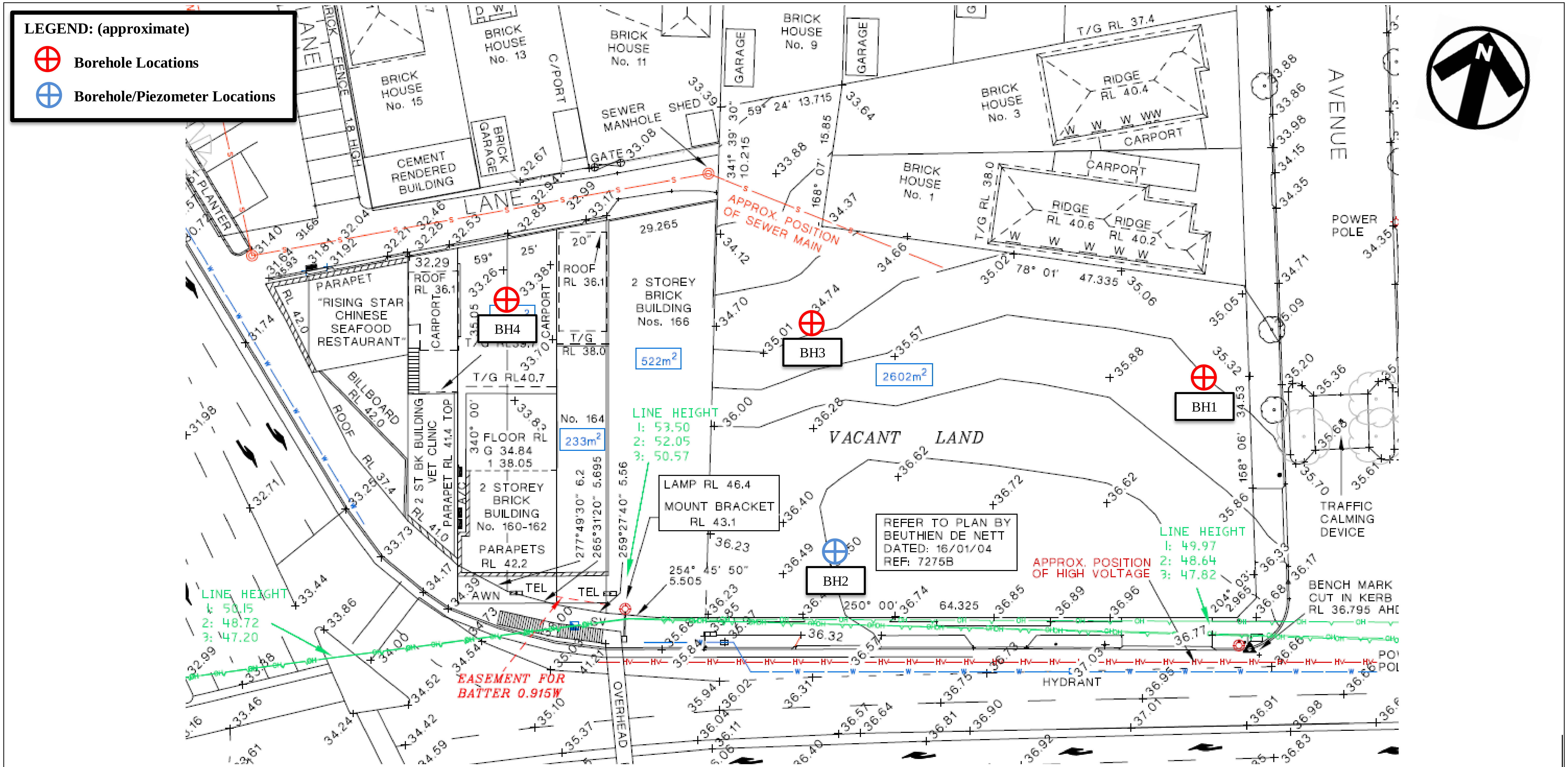

Image Source: Site Survey Plan for "160-178 Stoney Creek Road Beverly Hills" prepared by Stuart De Nett Land Surveyors, and referenced No. 11025C.

Aargus ENVIRONMENTAL - ENGINEERING - DRILLING - LABORATORIES - ASBESTOS

Drawn	RB	**Cuzeno Pty Ltd** **Geotechnical Investigation** **160-178 Stoney Creek Road, Beverly Hills, NSW 2209**	Aargus	Figure 1
Checked	JN			Title: Site Plan
Date	24/01/2017			
Scale @ A3	NTS			Job No: GS6759-1A

APPENDIX C

ENGINEERING BOREHOLE LOGS

Aargus Pty Ltd
446 Parramatta Road
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Telephone: 1300 137 038

BOREHOLE NUMBER BH1

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 16/1/17 **COMPLETED** 16/1/17 **R.L. SURFACE** 35.4 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig **HOLE LOCATION** Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	NOT ENCOUNTERED	35.0, 34.5, 34.0	0.0 – 1.5 (0.5, 1.0, 1.5)			Sandy Gravelly CLAY, low to medium plasticity, brown, fine to corase grained gravel, fine grained sand, moist.		FILL
		33.5, 33.0, 32.5	1.5 – 3.2 (2.0, 2.5, 3.0)		CIS	Sandy CLAY, medium plasticity, brown, some fine grained rounded gravel, fine grained sand, tree rootlets, moist, firm.	SPT 2, 3, 4 N=7 SPT 3, 5, 8/33mm bouncing	
		32.0, 31.5, 31.0, 30.5	3.2 – 5.0 (3.5, 4.0, 4.5, 5.0)			SHALE, pale grey to brown, extremely weathered, extremely low strength, moist.		BEDROCK

BOREHOLE / TEST PIT GS6759.GPJ GINT STD AUSTRALIA.GDT 17/1/23

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Telephone: 1300 137 038

BOREHOLE NUMBER BH1

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 16/1/17 **COMPLETED** 16/1/17
R.L. SURFACE 35.4 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT						SHALE, pale grey to brown, extremely weathered, extremely low strength, moist. *(continued)* becoming hard at 5.0m bgl.		
		30.0	5.5					
		29.5	6.0					
		29.0	6.5			becoming dark grey at 6.5m bgl.		
		28.5	7.0					
		28.0	7.5					
		27.5	8.0					
		27.0	8.5					"TC" Bit Refusal at 8.6m bgl
						Borehole BH1 terminated at 8.6m		
		26.5	9.0					
		26.0	9.5					
		25.5	10.0					

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BOREHOLE NUMBER BH2

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 16/1/17 **COMPLETED** 16/1/17
R.L. SURFACE 36.5 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	NOT ENCOUNTERED	36.0	0.5		Sandy Gravelly CLAY, low plasticity, brown to grey, fine to coarse grained gravel, fine grained sand, moist.		FILL
		35.5	1.0				
		35.0	1.5	CI-CH	Silty CLAY, medium to high plasticity, brown to grey, mottled red, some fine grained ironstone gravel, moist, firm.	SPT 2, 3, 3 N=6	RESIDUAL SOILS
		34.5	2.0				
		34.0	2.5	CI	Silty CLAY, medium plasticity, pale grey, some fine to coarse grained ironstone gravel, interbedded with shale, moist, firm to stiff.	SPT 4, 4, 6 N=10	
		33.5	3.0		SHALE, pale grey to brown, extremely weathered, extremely low strength, moist.		BEDROCK
		33.0	3.5				
		32.5	4.0				
		32.0	4.5				High "TC" Bit resistance at 4.5m bgl.
					Borehole BH2 continued as cored hole		
		31.5	5.0				

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BOREHOLE NUMBER BH2

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 16/1/17 **COMPLETED** 16/1/17 **R.L. SURFACE** 36.5 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig **HOLE LOCATION** Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength (EL VL L M H VH EH)	$Is_{(50)}$ MPa (D- diametral, A- axial)	RQD %	Defect Spacing mm (30 100 300 1000 3000)	Defect Description
			36.0	0.5								
			35.5	1.0								
			35.0	1.5								
			34.5	2.0								
			34.0	2.5								
			33.5	3.0								
			33.0	3.5								
			32.5	4.0								
			32.0	4.5		Continued from non-cored borehole						
NMLC			31.5	5.0		SHALE, grey to brown, occassional pale grey to brown laminations.	EW/HW			13		4.50m, Fractured Zone (FZ), 200mm 4.70m, Extremely Weathered (EW), Clay (CL), 600mm

BOREHOLE NUMBER BH2

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CLIENT Cuzeno Pty Ltd | **PROJECT NAME** Geotechnical Investigation
PROJECT NUMBER GS6759-1A | **PROJECT LOCATION** 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 16/1/17 **COMPLETED** 16/1/17 | **R.L. SURFACE** 36.5 | **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd | **SLOPE** 90° | **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig | **HOLE LOCATION** Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter | **LOGGED BY** JN | **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	RL (m)	Depth (m)	Material Description	Weathering	Is(50) MPa (D-diametral, A-axial)	RQD %	Defect Description
NMLC			SHALE, grey to brown, occassional pale grey to brown laminations. *(continued)*	EW/HW		13	5.14m, Joint (J), R, UN, 5 deg
	31.0	5.5					5.42m, Bedding (B), Rough (R), Undulating (UN), 5 deg 5.44m, B, R, U, 5 deg 5.52m, EW, 80mm 5.60m, FZ, 110mm
	30.5	6.0			D 0, A 0	28	5.76m, J, R, UN 5.81m, J, R, UN 5.84m, B, Smooth (S), Curved (CU) 5.90m, J, R, UN 5.93m, FZ, 150mm 6.05m, B, R, UN 6.10m, J, R, UN 6.14m, B, R, CU 6.19m, J, R, UN 6.23m, B, R, UN 6.30m, J, R, UN 6.46m, J, R, UN
	30.0	6.5					6.58m, FZ, 340mm
			SHALE, dark grey, occassional pale grey laminations.	HW/MW			
	29.5	7.0					6.92m, J, R, UN 6.97m, J, R, UN 7.12m, FZ, 100mm 7.22m, J, R, UN
	29.0	7.5					7.35m, J, R, UN 7.41m, J, R, CU 7.48m, FZ, 180mm
	28.5	8.0				18	7.66m, J, R, UN 7.69m, B, R, UN 7.75m, B, R, UN 7.86m, B, R, UN 7.95m, J, R, UN 7.96m, J, R, UN 8.00m, J, R, CU 8.04m, EW, CL, 100mm 8.14m, J, R, UN 8.22m, J, R, UN 8.31m, B, R, UN 8.41m, FZ, 310mm
	28.0	8.5					
	27.5	9.0			D 0.16, A 0.27		8.72m, J, R, CU 8.75m, B, S, CU 8.80m, B, S, CU 8.87m, FZ, 90mm 8.96m, J, R, UN 9.07m, EW, CL, 100mm 9.17m, FZ, 230mm
	27.0	9.5	BH2 terminated at 9.4m				
	26.5	10.0					

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BOREHOLE NUMBER BH3

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 17/1/17 **COMPLETED** 17/1/17
R.L. SURFACE 34.8 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	NOT ENCOUNTERED	34.5, 34.0, 33.5	0.5, 1.0, 1.5			Sandy Gravelly CLAY, low plasticity, brown to grey, fine to corase grained gravel, fine grained sand, moist.		FILL
		33.0	2.0		CIS	Sandy CLAY, medium plasticity, brown, some fine grained rounded gravel, fine grained sand, moist, firm to stiff.	SPT 3, 4, 4 N=8	
		32.5	2.5		CI	Silty CLAY, medium plasticity, brown, mottled red, some fine grained ironstone gravel, shale laminations, moist.		RESIDUAL SOILS
		32.0, 31.5, 31.0, 30.5, 30.0	3.0, 3.5, 4.0, 4.5, 5.0			SHALE, brown to grey, extremely weathered, extremely low strength, moist. very hard layer to auger at 2.8m bgl. becoming brown to pale grey at 4.5m bgl.		BEDROCK

BOREHOLE / TEST PIT GS6759.GPJ GINT STD AUSTRALIA.GDT 17/1/23

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BOREHOLE NUMBER BH3

CLIENT Cuzeno Pty Ltd
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PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 17/1/17 **COMPLETED** 17/1/17
R.L. SURFACE 34.8 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		29.5	5.5			SHALE, brown to grey, extremely weathered, extremely low strength, moist. *(continued)*		
		29.0	6.0					
		28.5	6.5					"TC" Bit Refusal at 6.5m bgl
		28.0	7.0			Borehole BH3 terminated at 6.5m		
		27.5	7.5					
		27.0	8.0					
		26.5	8.5					
		26.0	9.0					
		25.5	9.5					
		25.0	10.0					

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BOREHOLE NUMBER BH4

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 17/1/17 **COMPLETED** 17/1/17
R.L. SURFACE 33.4 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	NOT ENCOUNTERED					CONCRETE PAVEMENT 100mm.		PAVEMENT
		33.0				Gravelly Silty CLAY, medium plasticity, dark brown to grey, fine to coarse grained gravel, moist.		FILL
		32.5	0.5 1.0		CI	Silty CLAY, medium plasticity, brown, moist, firm to stiff.		RESIDUAL SOILS
		32.0	1.5		CI-CH	Silty CLAY, medium to high plasticity, pale grey, brown laminations, mottled red, some fine grained ironestone gravel, shale laminations, moist, very stiff.	SPT 3, 7, 9/10mm bouncing	
		31.5 31.0 30.5 30.0 29.5 29.0 28.5	2.0 2.5 3.0 3.5 4.0 4.5 5.0			SHALE, grey to brown, extremely weathered, extremely low strength, moist.		BEDROCK

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BOREHOLE NUMBER BH4

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DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT						SHALE, grey to brown, extremely weathered, extremely low strength, moist. *(continued)*		
		28.0	5.5			becoming dark grey from 5.5m bgl		
		27.5	6.0					
		27.0	6.5					
		26.5	7.0			hard layer at 6.8m bgl.		
		26.0	7.5					"TC" Blt Refusal at 7.7m bgl
		25.5	8.0			Borehole BH4 continued as cored hole		
		25.0	8.5					
		24.5	9.0					
		24.0	9.5					
		23.5	10.0					

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DATE STARTED 17/1/17 **COMPLETED** 17/1/17
R.L. SURFACE 33.4 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength (EL VL L M H VH EH)	$Is_{(50)}$ MPa D- diametral A- axial	RQD %	Defect Spacing mm (30 100 300 1000 3000)	Defect Description
		28.0	5.5								
		27.5	6.0								
		27.0	6.5								
		26.5	7.0								
		26.0	7.5		Continued from non-cored borehole						
NMLC		25.5	8.0		CORELOSS 300mm.						7.77m-8.00m, CORELOSS
		25.0	8.5		SHALE, dark grey, pale grey laminations.	MW			48		8.02m, Joint (J), Rough (R), Undulating (U), 85 deg 8.05m, J, R, U, 5 deg 8.08m, J, R, U, 5 deg 8.16m, B, R, U, 10 deg 8.31m, J, Smooth (S), Planar (P), 45 deg 8.60m, J, R, Curved (C), 15 deg 8.68m, J, R, C, 15 deg 8.73m, J, R, P, 0-5 deg
		24.5	9.0					D 0.02 A 0.98			8.92m, J, R, Stepped (ST), 5 deg 9.00m, Crushed Zone (CZ), 80mm
		24.0	9.5								9.19m, B, S, U, 0-5 deg 9.21m, B, S, U, 0-5 deg 9.39m, B, R, ST, 10 deg 9.49m, B, R, ST, 10 deg 9.54m, J, R, C, 10 deg
		23.5	10.0						9		9.72m, J, S, P, 0 deg 9.79m, J, R, P, 0 deg 9.89m, J, R, U, 5 deg 9.90m, J, R, U, 5 deg

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Petersham NSW 2049
Telephone: 1300 137 038

BOREHOLE NUMBER BH4

CLIENT Cuzeno Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER GS6759-1A
PROJECT LOCATION 160-178 Stoney Creek Rd, Beverly Hills, NSW 2209

DATE STARTED 17/1/17 **COMPLETED** 17/1/17
R.L. SURFACE 33.4 **DATUM** m AHD
DRILLING CONTRACTOR IVAN Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Truck Mounted Drilling Rig
HOLE LOCATION Refer to Site Plan Figure 1
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** MM
NOTES RL to the top of borehole and depths of the subsurface conditions are approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength (EL VL L M H VH EH)	$Is_{(50)}$ MPa (D- diametral, A- axial)	RQD %	Defect Spacing mm (30 100 300 1000 3000)	Defect Description
NMLC					SHALE, dark grey, pale grey laminations. *(continued)*	MW		D 0, A 0.44			9.95m, J, R, C, 5 deg 10.00m, Extremely Weathered (EW), 50mm 10.11m, J, R, P, 5 deg 10.18m, J, R, P, 5 deg 10.20m, CZ, 70mm
		23.0 22.5 22.0	10.5 11.0 11.5		CORELOSS 1470mm.				9		10.27m-11.74m, CORELOSS
		21.5 21.0 20.5 20.0 19.5	12.0 12.5 13.0 13.5 14.0		SHALE, dark grey, pale grey laminations.			D 0.48, A 0.96	10		11.79m, J, R, P, 5 deg 11.81m, J, R, P, 5 deg 11.85m, J, R, ST, 10 deg 11.92m, J, R, U, 5 deg 11.96m, J, R, U, 5 deg 12.00m, EW, 40mm 12.10m J, R, ST, 10 deg 12.15m, J, R, ST, 10 deg 12.21m, J, S, P, 5 deg 12.24m, J, R, C, 10 deg 12.27m, CZ, 60mm 12.38m, J, R, U, 5 deg 12.43m, J, R, U, 5 deg 12.47m, CZ, 150mm 12.64m, CZ, 60mm 12.72m, EW, 50mm 12.84m, J, R, U, 90 deg 12.91m, J, R, U, 5 deg 13.05m, J, R, U, 5 deg 13.12m, CZ, 10mm 13.20m, CZ, 60mm 13.30m, J, R, U, 5 deg 13.33m, J, R, U, 5 deg 13.50m, Fractured Zone (FZ), 180mm 13.75m, CZ, 150mm 13.95m, CZ, 180mm
		19.0 18.5	14.5 15.0		BH4 terminated at 14.12m						

APPENDIX D

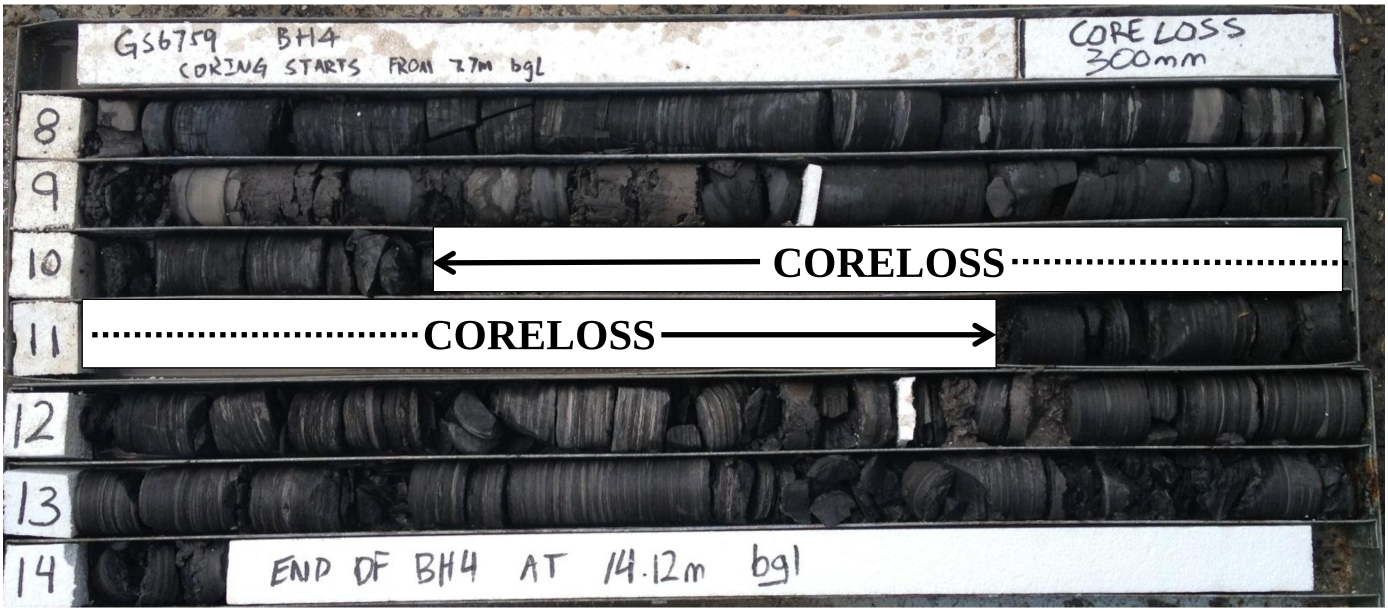
ROCK CORE PHOTOGRAPHS

Aargus

Core Box Photographs

BH2: 4.5m to 9.4m

BH4: 7.7 to 14.12m



APPENDIX E

POINT LOAD TEST RESULTS

Aargus POINT LOAD STRENGTH INDEX REPORT

Client:	Cuzeno Pty Ltd	**Date Tested:**	19/01/2017
Address:	160–178 Stoney Creek Road, Beverly Hills, NSW 2209	**Job No:**	GS6759-1A

Borehole ID	Depth (m)	Sample Description	Test Type	Point Load Index $Is_{(50)}$	UCS (MPa)	Notes
BH2	5.83	Shale	Diametral	0.0	0.0	Sample Dry
			Axial	0.02	0.4	Sample Dry
BH2	8.77	Shale	Diametral	0.16	2.8	Sample Dry
			Axial	0.27	4.8	Sample Dry
BH4	8.87	Shale	Diametral	0.02	0.3	Sample Dry
			Axial	0.98	17.6	Sample Dry
BH4	10.14	Shale	Diametral	0.0	0.0	Sample Dry
			Axial	0.44	7.9	Sample Dry
BH4	12.38	Shale	Diametral	0.5	8.6	Sample Dry
			Axial	0.96	17.3	Sample Dry

Comments: UCS –Unconfined Compressive Test. Multiplication Factor of 18 was used to calculate UCS.	Sheet 1 of 1	Tested By:	RB
		Checked By:	JN