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M K GEO ENG

In Cooperation With

PTC CONSULTING ENGINEERS

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Proposed Mixed Use Development at

Nos. 13 to 19 Canberra Avenue St Leonards, NSW 2065

Prepared for

Hyecorp Property Group

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REFERENCES

1. Australian Standard – AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake actions in Australia.
2. Australian Standard – AS 1726-1993 Geotechnical Site Investigation.
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1. INTRODUCTION

In accordance with the agreed brief, a preliminary geotechnical site investigation was carried out at nos. 13 to 19 Canberra Ave, St Leonards, NSW 2065. The site investigation was followed by interpretation of the results and assessment of the geotechnical conditions of the site.

The purpose of the investigation was to assess the existing ground conditions and geotechnical design requirements of the site for a proposed mixed use development.

This report presents results of the geotechnical site investigation, interpretation and assessment of the site existing geotechnical conditions, together with recommendations for design and construction of ground structures for the proposed development.

2. AVAILABLE INFORMATION

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

- Architectural drawings titled “13 to 19 Canberra Ave, St Leonards”, by SJB Architects, consisting of fifty sheets, referenced 6429, dated 13th October 2021;
- Survey plan titled “Nos.13 to 19 Canberra Avenue, St Leonards, Topographical Detail Survey of Property and Surrounds”, by DSP Surveyors and Engineers, sheets 1 and 2, referenced 6071-DET, dated 18th December 2020; and
- A service protection report titled “13-19 Canberra Ave, St Leonards” by Qalcheck Pty Ltd, dated 13th August 2021.

3. SCOPE OF WORK

Fieldwork for the preliminary geotechnical site investigation was carried out by a Geotechnical Engineer from Geosense Drilling following in general, the guidelines provided in the Australian Standard AS1726-1993 “Geotechnical Site Investigation” (Reference 2).

The fieldwork tasks comprised the following:

- Collection and review of Dial-Before-You-Dig (DBYD) plans;
- Service locating using electromagnetic detection equipment to ensure the borehole locations are positioned away from underground services;
- Machine drilling of two boreholes identified as BH01 to BH06, inclusive, using a rotary drilling rig owned and operated by Geosense ;
- Collection of soil samples and rock cores during drilling;
- Reinstatement of the boreholes with the soil cuttings; and
- Installation of a standpipe piezometer in boreholes BH01.

Following the site investigation, the following tasks were carried out:

- Laboratory testing consisting of Point Load Index Test was carried out on sixty rock cores recovered from the two boreholes; and
- Pumping test and monitoring of groundwater level in the BH01 piezometer.

The approximate locations of the boreholes completed during this investigation are depicted on a plan of the site, which is reproduced from the NSW Six Maps and presented as "Figure 1 - Site Borehole Location Plan" attached as Appendix A.

The boreholes were drilled with flight auger to Tungsten Carbide (TC) bit refusal, drilling thence continued with coring using NMLC technique to depths varying from approximately 29.4 to 30.5m below ground level (bgl).

The site investigation was followed by interpretation of the results, assessment of the main geotechnical aspects that may be associated with the proposed development and preparation of this geotechnical report. This preliminary geotechnical investigation report summarises the results of the geotechnical site investigation, interpretation, assessment and recommendations for design and construction of the proposed development.

4. SITE DESCRIPTION

The site consists of the amalgamation of four residential properties being nos. 13, 15, 17 and 19 Canberra Ave, St Leonards. The site is located within a well-developed residential suburb and is bordered by the following road and properties:

- Canberra Ave carriageway and road reserve to the east;
- The property at No. 11 Canberra Ave to the north;
- The property at No. 21 Canberra Ave to the south; and
- The properties at Nos. 10, 12, 14 and 16 Holdsworth Ave to the west.

The adjoining properties are occupied by residential dwellings which are located at setbacks varying from 2.0 to 15.0m to the common boundaries with the site.

At the time of the site investigation, the properties at nos. 13 to 19 Canberra Ave were occupied by residential dwellings. Several mature trees are present at scattered locations within the site and alongside the site boundaries.

The survey plan indicates the ground surface within the site is slightly to moderately sloping from approximate reduced level (RL) 63.5m in the vicinity of the western boundary to approximately RL 58.0m in the vicinity of the eastern boundary. The topography of the surrounding lands indicate slight to moderate sloping towards the east towards the north shore rail corridor at approximately 87m to the east of the site.

The service protection report indicates existing sewer mains are running alongside the eastern boundary within the Canberra Ave carriageway, and alongside the western boundary within the adjoining properties at No. 10 to 16 Holdsworth Ave, inclusive. The sewer main to the east is located at approximately 10m from the eastern boundary and the invert level varies in depth from 2.0 to 3.28m bgl. The sewer main to the west is located at approximately 4.1m from the western boundary and the invert level is at approximately 1.74m bgl.

Other underground services such as water mains, stormwater, cables and similar may be running within the road reserve alongside the eastern boundary.

5. PROPOSED DEVELOPMENT

The architectural drawings referenced in Section 2 indicate that the currently proposed development consists of demolition of the existing dwellings and construction of a fifteen storey above ground building with four to six basement levels for underground carparking.

The drawings indicate the following setbacks are proposed for the basement walls to the site boundaries:

- No setback to the northern boundary;
- 2.0m to the western boundary;
- 1.0 to 3.0m to the eastern boundary; and
- The 15.0m wide land at No. 19 Canberra Ave adjacent to the basement to the south will be for a deep soil zone.

The drawings indicate the finished floor level (FFL) of the proposed lower basement level will vary from RL 44.9m AHD.

The approximate depths of the proposed excavation to the bulk excavation level (i.e. to the underside of the floor slab of the lower basement level) below the existing ground levels is estimated to vary from approximately 13.4m bgl within the eastern side to approximately 19.3m bgl on the western side.

Vehicular access to the basement will be provided via a ramp from Canberra Ave. The basement will be provided with staircases and lift shafts.

Further details are shown on the drawings referenced in Section 2.

6. LOCAL GEOLOGY

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 (Reference 8), indicates the site is located in the vicinity of the geological boundary between the Hawkesbury Sandstone formation (Rh) to the south and the Ashfield Shale formation (Rwa) to the north.

The Hawkesbury Sandstone formation is described as "*medium to coarse-grained quartz sandstone, very minor shale and laminate lenses*". The Ashfield Shale formation is described as "*black to dark-grey Shale and laminite*".

7. INVESTIGATION RESULTS

7.1 Surface Conditions

At the time of the site investigation, apart from the existing dwellings, concrete patios, driveways and footpaths, the site surface was covered with topsoil and associated grass.

7.2 Subsurface Conditions

The subsurface conditions encountered within the boreholes are recorded on the Borehole Logs attached in Appendix B. Photographs of the rock cores recovered during drilling of the boreholes, and results of laboratory Point Load Index testing, also are attached in Appendix B.

Subsurface conditions encountered during drilling of boreholes BH01 and BH02, of this investigation, using auger drilling consisted of the following:

- Fill materials consisting of gravelly Sand, fine to medium grained, extending to approximately 0.3m bgl; overlying
- Residual soils consisting of silty Clay, red and orange brown, mottled grey, stiff to very stiff, extending to refusal of drilling using Tungsten Carbide (TC) bit depths varying from approximately 0.7 to 0.8m bgl.

The observations made on the recovered rock cores from the boreholes using NMLC technique indicated the site is underlain by the following rock horizons (classes):

- Class II Sandstone, interbedded reddish brown, yellow and white, moderately to slightly weathered, medium strength, with moderately dipping bedding planes and laminations, and occasional clay seams, with horizons of moderate strength dark grey mudstone (i.e. upper Class II Shale), extending to approximately 7.5 and 8.0m bgl in boreholes BH02 and BH01, respectively. An approximately 200mm thick dark grey shale was encountered at approximately 10.8 and 10.2m bgl in boreholes BH01 and BH02, respectively; overlying

- Class I Sandstone, whitish grey, slightly weathered to fresh, high strength, with horizons of high strength dark grey mudstone (i.e. lower Class I Shale) from approximately 14.5 to 16.5m bgl, with slightly to moderately dipping bedding planes and grey laminations.

The rock horizons above were classified with reference to the guidelines provided in a paper by Pells et al (Reference 11).

It should be noted that although the stratification of the rock horizons were reasonably consistent, the depths of the shale horizons encountered in the two boreholes varied by approximately 0.5m in the upper shale horizons and approximately 2.0m in the lower shale horizons. Based on the 16.0m distance between the two boreholes, these depth differences indicate dipping towards the north by approximately 2° within the upper shale horizons and 8° within the lower shale horizons.

The recovered rock cores indicated several natural defects consisting of predominantly sub-horizontal, rough and clean joints, fractures, bedding partings, and occasional irregular and diagonal rough fractures, joints, and clay seams within the Class II Sandstone horizons. Within the Class I Sandstone horizons, the natural defects consisted of occasional horizontal and sub horizontal joints, and a crushed zone with clay seams of approximately 200mm in thickness was encountered at approximately 27.5m bgl in borehole BH01.

7.3 Hydrology and Groundwater

The site survey plan, and contour lines provided in the Six Maps (Reference 12) indicate the local topography is slightly to moderately sloping towards the east towards the North Shore Line rail corridor. The local groundwater and surface water are expected to flow towards the east.

Search for information on existing water bores available in the groundwater database of the NSW Office of Water (Reference 9) indicates no water bores exist within an area of 500m radius from the site.

A standpipe piezometer was installed in boreholes BH01 to approximately 18.0m bgl, with screening of the pipes from approximately 6.0 to 18.0m bgl. The borehole was backfilled with sand and a bentonite plug was installed from approximately 4.5 to 5.5m bgl.

Following the fieldwork the standing water level measured in the piezometer was at approximately 7.5m bgl. Therefore, it is inferred that the natural groundwater level is present in the form of seepage through joints and bedding planes within the sandstone horizons below 7.5m bgl.

Pump testing (i.e. raising head test) was carried out on 19th July 2021. The results of the pump testing are attached in Appendix B. Interpretation of the results is provided in Section 8.10.

It should be noted that seepage levels may be subject to seasonal fluctuations influenced by factors such as rainfalls, and future development of the surrounding lands.

8. GEOTECHNICAL ASSESSMENT

8.1 Overview

The results of the site investigation carried out at this site indicate the main geotechnical aspects associated with the proposed development include the following:

- Excavation conditions and vibration control;
- Stability of basement excavation and retaining walls;
- Foundations;
- Drainage and dewatering; and
- Site earthquake classification.

Assessment of the geotechnical aspects above and recommendations for design and construction of the proposed development are provided in the following sections.

8.2 Excavation Conditions

Based on the available information on the proposed development and the site survey plan excavation will be required for more than two thirds of the site. Excavation to below the bulk excavation level is inferred to be predominantly within moderately weathered to fresh sandstone (i.e. Class II and Class I Sandstone).

Subject to confirmation by inspection during excavation, Table 1 overleaf provides the Characteristic Geological Strength Index (GSI) and Rock Mass Rating (RMR), for the Class II and Class I Sandstone, which are described with reference to the classification provided in papers by Bertuzzi et al (Reference 6 and Reference 7). Table 1 also provides assessment of the rock rippability based on Weaver's Rippability Rating Chart (Weaver 1975) inferred for the rock horizons underlying the site.

Table 1: Preliminary GSI, RMR and Rippability

Rock Class	Characteristic GSI	RMR	Rippability
Class II Sandstone	65 to 90	57 to 69 Fair to Very Good Rock	Extremely Hard Ripping
Class I Sandstone	75 to 100	60 to 77 Good Rock	Extremely Hard Ripping/ Blasting(1)
(1) For classification only, blasting is not recommended for this site.			

The classification above indicates that heavy ripping, rock breaking equipment in conjunction with vibratory rock breaking equipment are likely to be required for excavation within the rock horizons to the bulk excavation level for this development, as the site is underlain by Class I and Class II Sandstone horizons.

The rippability rating of the rock horizons underlying the site indicates a combination of some of the following options should be considered:

- Medium to high energy generating methods such as heavy ripping and rock breaking such as Class 300/400C dozers (Cat D10 or equivalent); and
- Medium energy generating methods such as rock saw or grinder, alongside the boundaries, to aid breaking and trimming; or
- Other medium energy generating methods such as Penetrating Cone Fracture (PCF) for the massive unfractured Class I Sandstone; or
- Low energy generating methods such as: Line drilling and splitting.

Excavation contractors should be provided with a copy of this report. The contractors should make their own review and assessment as to the selection of most appropriate excavation methods and machinery, productivity, or bulking factors, taking into consideration vibration and noise aspects associated with the excavation. It is recommended that only excavation contractors with appropriate insurances and experience on similar projects, should be engaged to carry out the rock excavation at this site.

8.3 Vibration Control

Vibration levels should be maintained within acceptable levels to minimise the potential effects of vibration that would be generated during excavation and appropriate methods should be planned and appropriate machinery used in order to minimise transmission of vibrations to the proposed new sewer pipe, the adjoining properties and road.

Induced vibrations in existing structures within the adjoining properties should not exceed a Peak Particle Velocity (PPV) of 10mm/sec for brick or unreinforced structures that are in good conditions, 5mm/sec for residential and low rise buildings or 2mm/sec for historical or structures that are in sensitive conditions.

Table 2 below provides preliminary vibration limits and distances to ordinary structures related to jack and rock hammers, which are typically adopted for similar developments in NSW. It is recommended that detailed assessment based on a monitoring trial is carried out prior to construction in order to confirm the preliminary recommended operating limits.

Table 2: Vibration Limits and Distances to Ordinary Structures and Type of Plant

Distance to Nearest Structure (m)	Plant	Operating Limit (% of Maximum Capacity) to achieve 5mm/sec PPV	Plant	Operating Limit (% of Maximum Capacity) to achieve 10mm/sec PPV
1.5 to 2.5	Hand operated Jack Hammer	100	300kg Rock Hammer	50
2.5 to 5.0	300kg Rock Hammer	50	300kg Rock Hammer	100
			600kg Rock Hammer	50
5.0 to 10.0	300kg Rock Hammer	100	600kg Rock Hammer	100
	600kg Rock Hammer	50	900kg Rock Hammer	50

As vibration and noise are restricted to low levels, the use of low to moderate energy generating equipment such as rock saw or rotary grinder is recommended. These should be used at least near the site boundaries, to aid in breaking and trimming, in order to minimise transmission of vibrations and noise.

The excavation should start from the middle portion of the site and continue progressively towards the site boundaries in stages. The use of rock hammers should be avoided, particularly within the areas alongside the site boundaries. If necessary, hammering should be carried out based on confirmed operating limits, preferably horizontally along bedding planes of rock horizons or pre-cut rock boulders/blocks, away from the site boundaries with noise limits restricted to those acceptable to the adjoining properties and school. The rock hammers should be operated only for short durations/bursts in order to reduce the potential for amplification of vibrations.

If high energy generating equipment are used, then the measures overleaf are recommended prior to and during construction (including excavation):

- Undertaking dilapidation survey of the existing structures within the influence zone of the proposed excavation, within the adjoining properties and roads.
- Vibration monitoring during construction should be carried out using a vibration monitoring instrument (i.e. seismograph). The alarm levels should be set based on the appropriate PPV selected in accordance with the type of structures present within the zone of influence of the proposed construction works.
- If the vibrations exceed the alarm levels construction activities should cease immediately and the Geotechnical Engineer for this development should be contacted for assessment and modification of the construction methodology if necessary.

8.4 Noise

Noise generated during excavation and construction should be restricted to the appropriate limits specified in the "Interim Construction Noise Guideline" by the NSW Department of Environment and Climate Change (Reference 13). The guideline indicates that the affected parties within the adjoining properties should be consulted to schedule the project's work hours to achieve a reasonable noise outcome.

8.5 Ground Horizontal Movement

For excavation through the medium to high strength sandstone horizons (i.e. Class II and Class I Sandstone), there will be a potential for some ground horizontal movement along bedding planes and partings due to rock relaxation induced by the release of locked-in horizontal stresses within the rock mass. Typical horizontal movements for basement excavations in Sydney sandstone reported in the published literature vary from 1 to 2 mm per metre of excavation depth, extending to a distance of up to 2 x the excavation depth, and movements ranging from 10 to 20mm have been reported. For a 13.4 to 19.3m deep excavation, the potential horizontal movement may be within such range, or slightly more, extending to a distance of 20 to 40.0m away from the excavation.

Design of an appropriate retention system to restrain the lateral stresses equivalent to such movements is impracticable. It is therefore common in Sydney to allow for a gap between the rock face and basement structures to avoid the adverse effects of the horizontal movement. However, such movement can be detrimental to existing structures within the zone of influence of the excavation. It is recommended that monitoring is carried out during the basement excavation and that the effects of movement should be mitigated or repaired.

The field stresses for the Class II and Class I Sandstone horizons underlying this site were estimated with reference to the published literature with some adjustment made due to the limited depth of the proposed basement excavation.

Below are equations recommended for the Class II and Class I Sandstone horizons:

$\sigma_H = 2.5 + 2.0\ \sigma_V$ $\sigma_h = 0.7\ \sigma_H$

where; σ_V = Insitu vertical stress

σ_H = Insitu horizontal stress in plane direction

σ_h = Insitu horizontal stress in out of plane direction

All stresses are in MPa

8.6 Stability of Basement Excavation during Construction

Temporary excavation within the fill and residual soils, which extend to approximately 0.8m bgl may be carried out with a cut batter of 1V:2H, where V denotes "Vertical" and H denotes "Horizontal".

As the Class II and Class I Sandstone horizons are typically self-supporting, no temporary shoring is anticipated to be required. As discussed in Section 8.5, it would be impracticable to design appropriate retention system to restrain the potential lateral stresses induced by release of locked-in insitu horizontal stresses.

For fractured horizons of the sandstone, shotcrete and pattern bolting in about 2.0m grid may be adopted. For the unfractured horizons, spot bolting may be used where required.

The recommendations above should be confirmed during construction by inspection, which should be carried out by the Geotechnical Engineer. During basement excavation, observations and recording of the condition of exposed soil and rock faces should be carried out so any local softening or weakening of material resulting from possible seepage or the presence of any loosening of soil or rock wedges or the presence of adversely orientated defects can be identified and treated. If adverse defects are identified the parameters and recommendations provided in this report should be reviewed. The inspections should constitute "Hold Points".

The underground services within the adjoining road and properties are expected to be embedded into the sandstone. Due to the setback of the proposed basement to the site boundaries, no significant impacts are expected on the adjoining underground services.

Undertaking dilapidation survey of the existing structures within the influence zone of the basement excavation, within the adjoining properties and road is recommended to be carried out prior to commencing of excavation. Existing underground services within the zone of influence, within the site, adjoining properties and road should be identified and protected during construction. All excavations should be carried out in accordance with the "NSW WorkCover: Code of Practice – Excavation work" (Reference 10).

Monitoring of the ground movement along the site boundaries is recommended to be carried out until completion of the basement structure to the ground level. During construction, regular visual monitoring of the ground in the vicinity of the site boundaries would be required.

Installation of inclinometers at the site boundaries of the site together with monitoring during construction is recommended. Alternatively, monitoring can be carried out by surveying of markers installed on the vertical cuts at different depths.

If significant movement occurs during monitoring, excavation and construction must be ceased and the Geotechnical Engineer should be immediately contacted for assessment and modification of the shoring methodology if necessary.

Any cracks that may develop on the ground surface in the vicinity of the site boundaries must be immediately sealed to prevent percolation of surface water into the cracks and reduce the potential subsequent effects on the adjoining properties and road.

With the recommended retention options above, construction of the proposed basement in the short and long terms is expected to have low effects on the adjoining properties and road.

8.7 Retaining Walls

Depending on the integrity of the rock mass of the Class II and Class I Sandstone horizons and the amount of seepage below approximately 7.5m bgl permanent retaining walls may be required for the upper first or two basement levels.

The basement walls should be designed to withstand the lateral earth pressures and the applied surcharge loads within the zones of influence of the walls. The surcharge loads typically include existing and future proposed structures, traffic and construction traffic loads. Earthquake and hydrostatic lateral pressures based on the results of this investigation should be considered. If relevant, lateral stresses induced by compaction of backfill materials of the basement walls should also be considered in the design.

Where minor lateral movement is acceptable, retaining walls are typically considered as flexible structures. The design of flexible retaining walls should be carried out based on "active" lateral earth pressures. If it is critical to limit the lateral movement of the walls, the design should be carried out based on "at rest" lateral earth pressures. Typically, the "at rest" lateral pressure design is considered for cases when the retaining walls are restrained by concrete slabs of buildings, or by pre-stressed ground anchors in their permanent state.

Table 3 below provides recommended preliminary parameters for the design of the shoring and retaining walls retaining or embedded within the relevant soils and rock horizons encountered in the boreholes drilled during this investigation.

Table 3: Preliminary Geotechnical Design Parameters for Retaining Walls

Layer/ Unit	**Unit Weight** γ kN/m^3	**Effective Cohesion** ***c'*** kPa	**Effective Internal Friction Angle** ϕ' degrees	**Elastic Modulus** Horizontal E_{sh} MPa	**Poisson's Ratio** ν
Fill	17	0	26	8	0.35
Residual silty Clay	19	8	28	24	0.3
Class II Sandstone	24	500	37	1000	0.2
Class I Sandstone	24	1000	37	2000	0.2
Engineered Fill (Proposed)	18	3	30	20	0.3
The strength parameters should be confirmed during construction. The strength parameters of the rock horizons depend on the integrity of the rock mass and the presence of defects and bedding planes.					

- The coefficient of active and passive lateral earth pressure Ka and Kp, respectively, can be calculated using Coulomb's equations, or the charts by Caquot and Kerisel.
- The coefficient of lateral earth pressure at rest Ko, can be calculated using Jacky's equation (Ko= 1 – Sinϕ').

Preliminary coefficients of lateral earth pressure for the relevant soils and rock horizons encountered during the geotechnical site investigation are provided in Table 4 overleaf. The coefficients provided are based on horizontal ground surface behind and in front of the retaining walls with fully drained conditions.

The preliminary coefficients of lateral earth pressure should be verified prior to use in the design of the retaining walls.

Table 4: Preliminary Coefficients of Lateral Earth Pressure

Layer/ Unit	Coefficient of Active Lateral Earth Pressure Ka	Coefficient of At Rest Lateral Earth Pressure Ko	Coefficient of Passive Lateral Earth Pressure Kp
Fill	0.39	0.56	2.56
Residual silty Clay	0.36	0.53	2.77
Class II Sandstone	0.25	0.40	4.02
Class I Sandstone	0.25	0.40	4.02
Engineered Fill (Proposed)	0.33	0.50	3.00
The coefficients should be confirmed during construction. The coefficients of active and passive pressures assume some wall movement will be mobilised.			

Simplified calculations of lateral active (or at rest) and passive earth pressures can be carried out using the Rankine equations below:

For Flexible walls;

$Pa = K\,\gamma\,H - 2c'\sqrt{K}$ For calculation of Lateral Active or At Rest Earth Pressure for flexible walls

$Pa = 0.65\,K\,\gamma\,H$ For calculation of Lateral Active or At Rest Earth Pressure for braced walls

$Pp = K_p\,\gamma\,H + 2c'\sqrt{K_p}$ For calculation of Passive Earth Pressure

where,

P_a = Active (or at rest) Earth Pressure (kN/m^2)
P_p = Passive Earth Pressure (kN/m^2)
γ = Unit Weight (kN/m^3)
K = Coefficient of Lateral Earth Pressure (K_a or K_o)
Kp = Coefficient of Passive Lateral Earth Pressure
H = Retained Height (m)
c' = Effective Cohesion (kN/m^2)

The lateral pressures generated by the rock horizons should be confirmed during construction by inspection. The passive resistance of the rock horizons, may be calculated based on blocks of passive pressure equivalent to 4000 and 6000kPa for the Class I and Class II Sandstone, respectively.

Anchors and rock bolts socketed into the Class II Sandstone or Class I Sandstone horizons can be designed based on an allowable bond stress of 700kPa and 800kPa respectively. The anchors can be designed based on these capacities and parameters above subject on the following conditions:

- The bond (socket) length is at least 3.0m;
- 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; and
- Anchor testing should constitute a “Hold Point”.

8.8 Backfilling

Design and construction of the backfill for the excavation should be carried out in accordance with Australian Standard AS 3798-2007 “Guidelines on Earthworks for Commercial and Residential Developments” (Reference 5).

The following general recommendations for the backfilling:

- Filling using engineered fill in layers accompanied by adequate compaction.
- The filling should be carried out using clean materials to engineering standards accompanied by testing under supervision of the Geotechnical Engineer.
- Suitability of the excavated materials for reuse or imported materials for filling should be subject to satisfying the following criteria:
    - The materials should be clean (i.e. free of contaminants, deleterious or organic material), free of inclusions of >120mm in size.
    - Materials with excessive moisture content should not be used without conditioning.
    - The materials should satisfy AS 3798-2007 requirements.
- 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 building platforms;
    - The loose thickness of layer should not exceed 150mm; and
    - For driveways and footpaths, minimum density ratio of 95% of the maximum dry density for general fill and 98% for the subgrade to 0.3m depth.

8.9 Foundations

The architectural drawings indicate the FFL of the lower basement level of the proposed building is proposed to be at 44.9m AHD.

The results of the site investigation indicates foundations of the building will be founded onto predominantly Class I Sandstone. Excavation for the basement will result in removal of an overburden pressure equivalent to the unit weight of the removed materials times the excavation depth.

Based on the ground profile encountered in the boreholes, a foundation system consisting of cast insitu reinforced concrete shallow spread foundations, such as pad footings under columns and strip footings under walls may be applicable for the proposed building if the footings are sufficiently embedded onto the Class I Sandstone.

Installation of piles is not expected to be required for the building at the basement level, which is typically required if the axial loads on columns and walls exceed the allowable bearing pressure of the bearing stratum. Other cases where piles may be required include the need to reduce the potential effects of differential settlement, to increase the stiffness of the founding rock, or increase the resistance against lateral earthquake loads. Cast insitu reinforced concrete bored piles would be suitable for this site.

It should be noted that the structural elements outside the footprint of the basement will be underlain by the insitu residual soils or engineered fill, depending on the shoring option that will be adopted. The permanent basement walls should be designed taking into consideration the structural loading of the foundations within the zone of influence of the walls and the dynamic forces due to compaction of any backfill materials.

Table 5 overleaf provides preliminary geotechnical ultimate and allowable capacities, recommended for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) design, respectively, and elasticity parameters for shallow and piled foundations embedded onto the relevant soil and rock horizons underlying the site, subject to inspection during construction.

Table 5: Preliminary Geotechnical Foundation Design Capacities and Parameters

Layer/ Unit	Ultimate/*Allowable* End Bearing Pressure[1,3] kPa	Ultimate/ *Allowable* Shaft Adhesion Compression[2] kPa	Elastic Modulus Vertical E_{sv} MPa
Residual silty Clay	ULS 300	ULS 50 (25)	30
	SLS 100	*SLS 25 (12)*	
Class II Sandstone	ULS 60000	ULS 1500 (750)	1000
	SLS 8000	*SLS 750 (375)*	
Class I Sandstone	ULS 120000	ULS 3000 (1500)	2000
	SLS 12000	*SLS 1500 (750)*	

[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. The rock socket sidewalls should be free of soil and/or crushed rock, with at least 80% of the socket sidewall consisted of solid rock. Shaft adhesion should be reduced or ignored within sockets lengths that are smeared and fail to satisfy cleanliness requirements.
[3] Bearing capacity for shallow and pile foundations in rock are based on Pells et al (Reference 11).

As a rule of thumb the uplift (tension) side or shaft, adhesion for rock is usually as 50% of compression side adhesion.

It should be noted that the allowable bearing pressures (SLS) provided in Table 5 are based on Factor of Safety (FoS) against bearing capacity failure of ≥ 3.0. Based on the information available for this project and results of the site investigation, typical strength reduction factor values for verification of the geotechnical capacity for the ULS case, range from 0.4 to 0.5. However, the strength reduction factors should be selected based on the risk rating of the design in accordance with the relevant engineering standards such as the Australian Standard AS 2870-2011 "Residential slabs and footings" (Reference 4) and AS 2159-2009 "Piling - Design and installation" (Reference 3).

To minimise the effects of differential vertical ground deformation under the building loads, it is recommended all foundations of the proposed building should be founded on consistent rock horizons of similar class. Typically, the compressibility of the weathered shale and sandstone is very low. According to Pells et al (Reference 11) for foundations founded onto the weathered sandstone and shale horizons within the Sydney region, settlement of up to 1% of the minimum footing dimension can occur under the allowable (SLS) end bearing pressures provided in Table 5 above. Under the ultimate (ULS) bearing pressures settlement can exceed 5% of the minimum footing dimension.

For shallow footings constructed on the existing ground level, the footings should be embedded to at least 0.4m into the bearing stratum. Shallow footings should not be founded within topsoil or existing fill. If there is a need to reduce the effects of shrinkage and swelling of the insitu soils, the footings should be embedded further to a minimum of 0.6m.

For bored piles, shaft adhesion may be applied to socketed piles adopted for foundations provided socket shaft lengths conform to appropriate classes of sandstone, and accepted levels of shaft sidewall 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 sandstone bands is evident over socket lengths. For piled foundations embedded in soils with potential for shrinkage and swelling, shaft adhesion should be ignored in the zones of seasonal moisture variations due to the potential of cracks developed by cycles of moisture variations.

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

Shallow foundations in rock at the lower basement level will require spoon testing carried out on at least 1/3 of the total number of footings in conjunction with Point Load index testing on the rock cuttings for determination of rock strength. Pile foundations will require at least inspection of the rock socket visually by methods such as downhole camera in conjunction with inspection and testing of the cuttings.

Verification of the capacity of the exposed rock at the foundation base excavation by inspection should constitute a "Hold Point".

8.10 Drainage and Dewatering

The standing groundwater level was measured in the standpipe piezometer at approximately 7.5m bgl.

Based on the results of raising head test, the equivalent hydraulic conductivity of the joints of the rock horizons was estimated using a simplified equation that was developed by Hvorslev, which is summarised below:

$$K = \frac{Ln\left[\frac{L_e}{r_w}\right].r_c}{2.L_e.t_0}$$

Where,

K = Hydraulic Conductivity (m/sec)

L_e = Screen length (or difference in water levels between the start and end of the test) (m)

r_c = Radius of piezometer (m)

r_w = Radius of borehole (m)

t_0 = duration of the test (seconds)

Based on the test results and using the above equation, the equivalent hydraulic conductivity is estimated to be approximately 1.072 x 10^{-8} m/sec.

For preliminary assessment purpose, the flow rate of groundwater into the basement excavation assuming seepage will occur along the entire perimeter of the basement was estimated based on the Dupuit-Thiem equation for steady state unconfined flow as described in the diagram and equation in Figure 2 below.

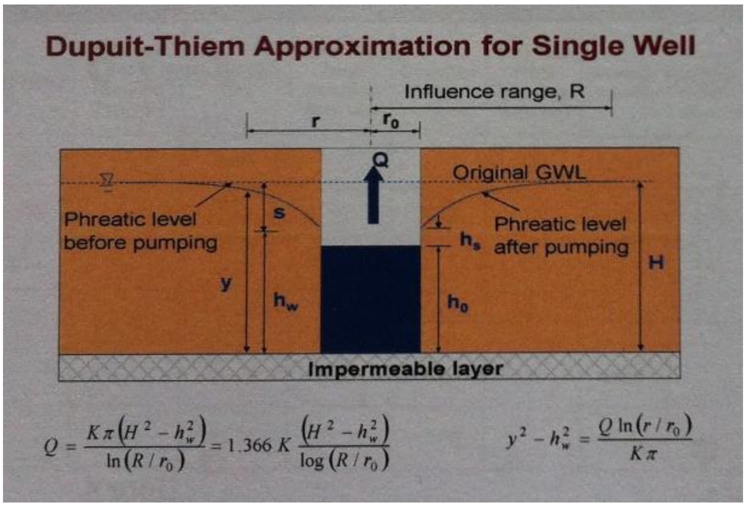

Figure 2 – Dupuit-Thiem diagram and equation for steady state unconfined flow

Where;

Q = groundwater inflow (m^3/s)

K = hydraulic conductivity (m/s)

H = Height of water table above assumed impermeable basement (m)

h_w = height of dewatering above assumed impermeable basement (m)

R = Radius of drawdown (m)

r_0 = equivalent radius of excavation (m)

Based on the raising head test results for the approximately 1781m^2 basement area, the groundwater inflow is estimated to be in the order of 0.34 and 1.6ML/year for assumed radius of drawdown of 30 and 25m, respectively.

Therefore, it would be prudent to allow for precautionary drainage measures in the design and construction of the proposed development for the potential of seepage during construction and in the long term. These measures may include the following:

- Sealing of the seepage joints with appropriate structural impervious measures.
- Strip drains or similar behind the basement perimeter walls to reduce hydrostatic pressures on the basement walls. The drains should be installed in conjunction with collection trenches or pipes and pits connected to the stormwater system of the building.
- The basement walls and lower basement floor should be constructed with appropriate construction joints.
- The capacity of the permanent basement walls should be verified with respect to the potential for hydrostatic pressures.
- Seepage or surface runoff inside excavated foundation excavations should be removed prior to concrete pouring.

It is recommended that the above measures should be reviewed during construction based on the exposed ground conditions during the basement excavation.

Dewatering during construction, would typically require a conventional sump and pump. Toe drains at the base of the basement walls and sump pits would be required within the excavations to collect surface water or seepage and a pump to discharge water to the public stormwater system subject to approval by the Council.

With the recommended procedures and measures described above, the potential effects on the proposed development, adjoining properties and road are expected to be low.

8.11 Site Earthquake Classification

The results of the site investigation indicate the presence of minor fill overlying stiff residual cohesive soils extending to approximately 0.7 to 0.8m bgl, overlying horizons of moderately to slightly weathered rock horizons of the Hawkesbury Sandstone formation. In accordance with Australian Standard AS 1170.4-2007 “Structural Design Actions” (Reference 1) the site may be classified, a “Strong rock site” (Class A_e). The Hazard Factor (Z) for Sydney in accordance with AS 1170.4-2007 is considered to be 0.08.

8.12 Additional Geotechnical Site Investigation and Analysis

Due to the constraints imposed by the existing dwellings within the site, it was not possible to undertake the drilling of more than two boreholes. It is our opinion that due to the potential of varying ground conditions throughout the site, the depth and classification of the rock horizons, and the groundwater levels, that the existing geotechnical data is not adequate for detailed design of the ground structures. It is recommended that additional site investigation is carried out consisting of at least another two deep machine borehole drilling. It is also recommended that additional standpipe piezometers are installed at two locations. At each location two to three nested standpipe piezometers are recommended to be installed to different depth. Additional pump (raising head) testing is also recommended together with monitoring.

Following the additional site investigation, geotechnical analysis consisting of finite element stress-deformation and groundwater analyses are recommended to be carried out for the staged excavation of the proposed basement.

It is expected with the recommended additional geotechnical investigation and detailed geotechnical analyses, a safe and an economical design can be achieved.

9. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The results of the geotechnical investigation and assessment for the site at nos. 13 to 19 Canberra Ave, St Leonards, NSW 2065, indicate the ground conditions in general are suitable for the proposed development subject to adoption of the recommendations made in this report. The following is a summary of the conclusions of the geotechnical assessment and recommendations for design and construction of the proposed development.

The site is underlain by minor fill and residual soils, overlying horizons of slightly weathered to fresh sandstone belonging to the Hawkesbury Sandstone formation, consisting of predominantly Class II sandstone extending to approximately 7.5 and 8.0m bgl, overlying horizons of Class I Sandstone. The standing groundwater level was measured at approximately 7.5m bgl in a standpipe piezometer installed to approximately 18.0m bgl. It is inferred that groundwater occurs in the form of seepage through natural fissures and fractures in the underlying weathered sandstone horizons. Based on the results of a raising head permeability testing for the approximately 1781m^2 basement area, the groundwater inflow into the open excavation is estimated to be in the order of 0.34 and 1.6ML/year for assumed radius of drawdown 30 and 25m, respectively. It would be prudent to allow for precautionary drainage measures in the design and construction of the proposed development for the potential of seepage during construction and in the long term.

Excavation to below the FFL of the lower basement level is inferred to be predominantly within horizons of Class II and Class I Sandstone. Heavy ripping, rock breaking equipment and vibratory rock breaking equipment are expected to be required for excavation through the Class II and Class I Sandstone. As vibration and noise are restricted to low levels due to the adjoining residential properties and road, the use of low to moderate energy generating machinery will be required, particularly near the site boundaries.

Depending on the integrity of the rock mass of the Class II and Class I Sandstone horizons and the amount of seepage below approximately 7.5m bgl permanent retaining walls may be required for the upper first or two basement levels.

For excavation through the Class II and Class I Sandstone, there will a potential for some ground horizontal movement along bedding planes partings due to rock relaxation induced by the release of locked-in horizontal stresses within the rock mass. For a 13.4 to 19.3m deep excavation, the potential horizontal movement may be in the range of 10 to 20mm, or slightly more, extending to a distance of 20 to 40.0m away from the excavation.

Based on the subsurface conditions encountered during the site investigation consideration may be given to the options summarised:

- Temporary excavation within the fill and residual soils, which extend to approximately 0.8m bgl may be carried out a cut batter of 1V:2H.
- The Class II and Class I Sandstone horizons are typically self-supporting and no temporary shoring is anticipated to be required. It would be impracticable to design appropriate retention system to restrain the potential lateral stresses induced by release of locked-in insitu horizontal stresses.
- For fractured horizons of the sandstone, shotcrete and pattern bolting in about 2.0m grid may be adopted. For the unfractured horizons, spot bolting may be used where required.

Alternative options may be considered for the basement perimeter shoring and permanent walls, subject to assessment by the Structural Engineer in consultation with the Geotechnical Engineer.

A foundation system consisting of cast insitu reinforced concrete shallow spread foundations, such as pad footings under columns and strip footings under walls is assessed applicable for the proposed development provided the footings are sufficiently embedded onto the bearing stratum being Class I Sandstone. The design should be verified with reference to the recommended preliminary geotechnical capacities and parameters provided in this report.

Additional geotechnical site investigation consisting of drilling at least two boreholes within the site will be required to confirm the assessment and recommendations provided in this report. Detailed stress-deformation analysis and groundwater analysis are also required as part of the design of the proposed development.

The design of the foundations, retaining walls, shoring measures, backfilling and drainage measures should take into consideration the geotechnical aspect discussed in this report. It is recommended the final architectural and structural design drawings should be reviewed by Mark Kiryakos - Geotechnical Engineer for further assessment and confirmation of the conclusions and recommendations provided in this report.

It is assessed that the depths and elevations of the soil and rock horizons may vary across the site and therefore the stratification of the rock horizons should be confirmed during excavation of the proposed basement and the foundations. Inspections by the project

Geotechnical Engineer during the basement and foundation excavations are required. The inspections should constitute "Hold Points".

The summary of the conclusions and recommendations should be read in conjunction with the entire report.

10. LIMITATIONS

The geotechnical assessment of the subsurface profile and geotechnical conditions within the proposed development area and the conclusions and recommendations provided in this report have been based on available information obtained during the work carried out by *Mark Kiryakos – Geotechnical Engineer* and in the provided documents listed in Section 2 of this report.

Inferences about the nature and continuity of ground conditions within the site are made, but cannot be guaranteed. It is possible that the nature of the exposed subsurface soils and rock will require further investigation and modification of the design based upon this report. It is recommended that the ground conditions within the site should be inspected during construction by *Mark Kiryakos - Geotechnical Engineer* to assess if the conditions are compatible with the assumptions made in this report and/or referenced reports. In all circumstances, if the ground conditions differ from those described or assumed to exist, *Mark Kiryakos – Geotechnical Engineer* should be consulted for further advice and review of the conclusions and recommendations provided for this site. *Mark Kiryakos – Geotechnical Engineer* does not accept any liability for site conditions not observed or accessible during the time of the investigation or inspection.

This report and associated documents have been prepared for the particular purpose described to the author, and for the sole use of the client, ***Hyecorp Property Group***. Any reliance assumed by third parties on this report shall be at such parties' own risk. No responsibility is accepted for the use of any part of this report in any other context or for any other purposes. Any ensuing liability resulting from use of the report by third parties cannot be transferred to *Mark Kiryakos – Geotechnical Engineer*.

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Yours faithfully,

M.D. Kiryakos

Mark Kiryakos
BScEng, MEngSt
Consulting Geotechnical Engineer

APPENDIX A

Site Borehole Location Plan

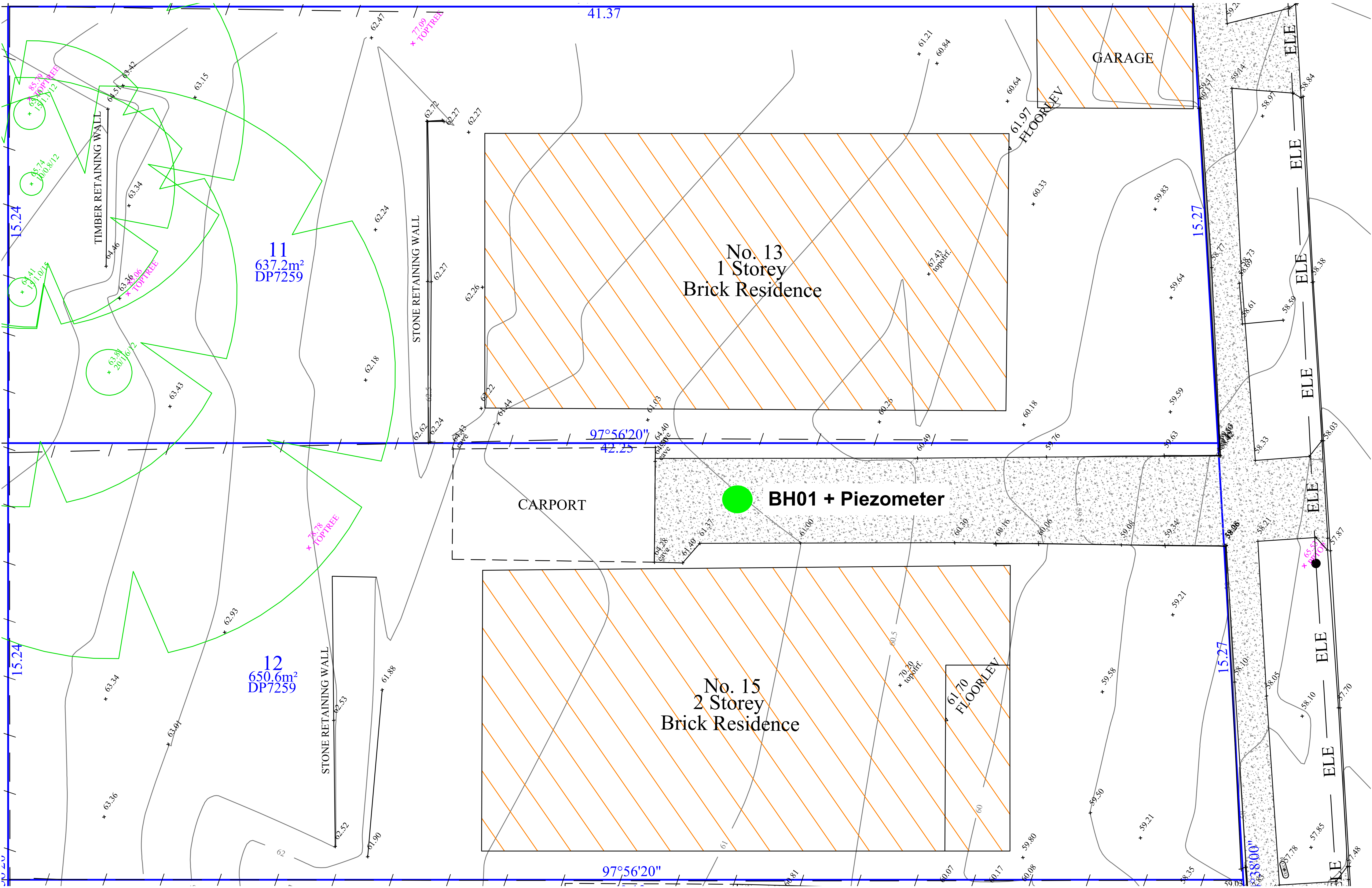
Image Source: NSW Six Maps

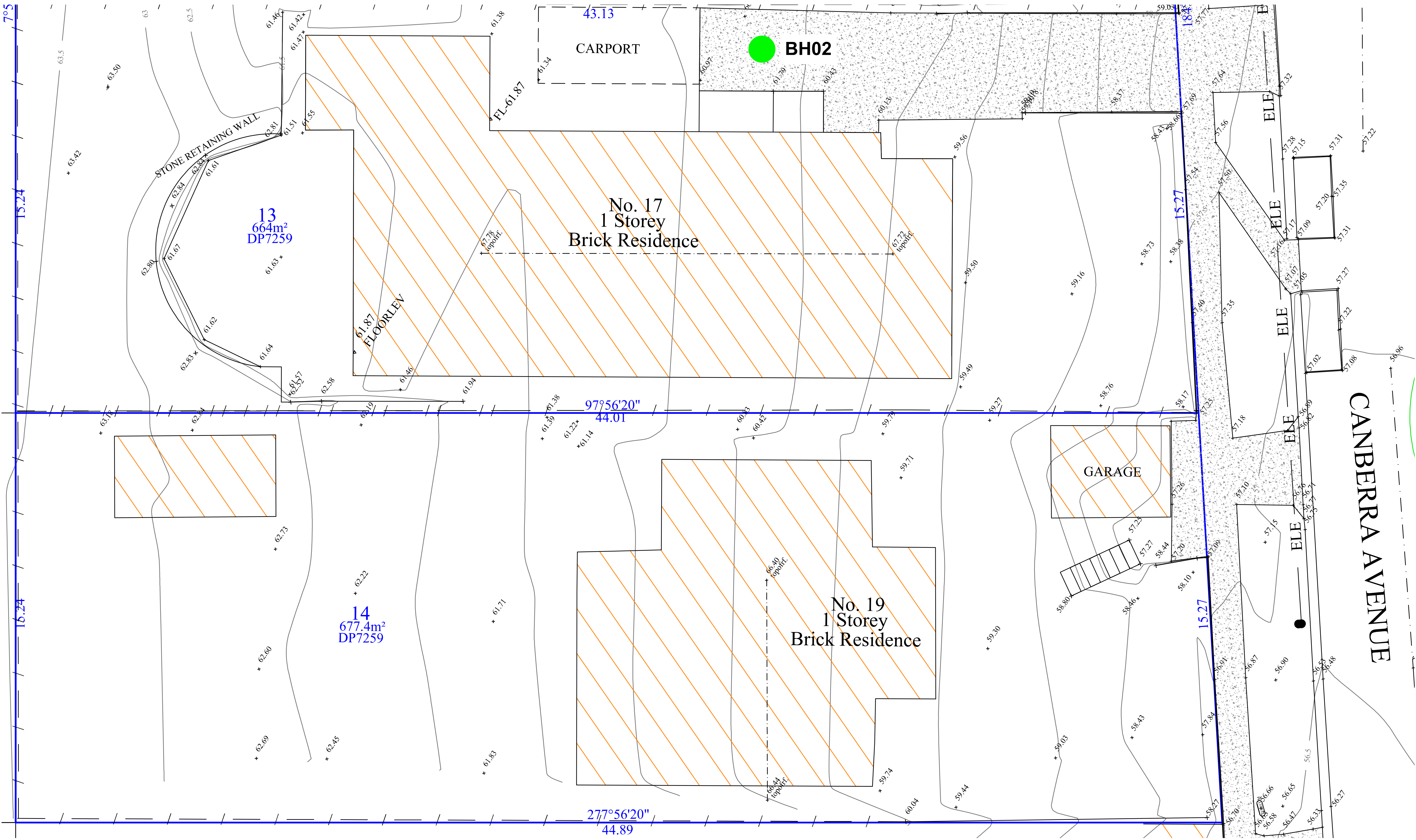
⊕ Approximate locations of boreholes

				MK GEO ENG / PTC Consulting Engineers	Drawn By:	MK	**Hyecorp Property Group** **Proposed Mixed Use Development** **Nos 13-19 Canberra Ave, St Leonards NSW 2065** **Preliminary Geotechnical Site Investigation**	Project No.: G2020-22A
					Checked By:	MK		Figure Title: Site Borehole Location Plan
				PO Box 474 Broadway NSW 2007	Date:	26/07/2021		
Rev.	Details	By	Date		Scale:	NTS		Figure No.: 1 / Rev.: 0

APPENDIX B

Fieldwork Factual Report


41.37
GARAGE
61.97 FLOOR LEV
No. 13
1 Storey
Brick Residence
11
637.2m²
DP7259
15.24
15.27
TIMBER RETAINING WALL
STONE RETAINING WALL
97°56'20"
42.25
CARPORT
BH01 + Piezometer
12
650.6m²
DP7259
No. 15
2 Storey
Brick Residence
61.70 FLOOR LEV
ELE


BH02
CARPORT
No. 17
1 Storey
Brick Residence
13
664m²
DP7259
STONE RETAINING WALL
No. 19
1 Storey
Brick Residence
14
677.4m²
DP7259
GARAGE
CANBERRA AVENUE
97°56'20"
44.01
277°56'20"
44.89
43.13
15.27
15.24
ELE

BOREHOLE LOG

GEOSENSE DRILLING & ENGINEERING

BH NO. BH01

Project	Proposed Development	**Sheet**	1 of 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By**	JZ **Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By**	MK **Date** 08/07/2021

Drilling Contactor	GeoSense Drilling & Engineering	**RL**	**61.0**m AHD
Drill Rig	Comacchio GEO205	**Inclination**	-90°

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	*DEPTH* RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	GROUP SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY REL. DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/T	L-M	GWNE	0	*61.0m*				GP	CONCRETE, 120mm	M	-	PAVEMENT / FILL
									Fill; Gravelly SAND; fine to medium grained, dark brown			
									0.3m			
				60.7m				CL	Silty CLAY; low to medium plasticity, mottled grey, red-brown and orange brown	w> PL		RESIDUAL
			0.5									
				60.2m					*Start coring at 0.8m*			TC-bit refusal on bedrock
			1									

GEOSENSE DRILLING & ENGINEERING

CORED BOREHOLE LOG

BH NO. BH01

Project	Proposed Development	**Sheet**	2 OF 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By** JZ	**Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By** MK	**Date** 08/07/2021

Drilling Contactor	GeoSense Drilling & Engineering	**RL**	m AHD
Drill Rig	Comacchio GEO205	**Inclination**	-90°

Drilling | Field Material Description | Defect Information

METHOD	WATER	TCR	RQD	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)
				0 – 0.8			Start coring at **0.8m**				
NMLC		1.11m	0.88m (79%)	0.8 – 6.05	*60.2m*		SANDSTONE; fine grained, mottled pale grey, red-brown and orange-brown	MW - SW		1.17 JT 10 PR RF CN 1.67 BP 0 UN RF CN 1.75-1.77 DS, clay infill 1.81 BP 5 PR RF CN 1.85-1.88 DS, clay infill 1.90 DB (Drilling Break) 2.01 DB 2.07 JT 10 PR RF CN 2.37 BP 10 CU RF ST 2.53 BP 15 CU RF ST 2.88 BP 0 UN RF CN 2.93 DB 3.11-3.12 DS, clay infill 3.48 BP 10 PR RF CN 3.68-3.70 DS, clay infill 3.93 BP 0-5 CU RF CN 4.05 DB 4.12 DB 5.22 DB 5.62 JT 10 UN RF CN 5.71 DB	
		2.22m	1.97m (89%)								
		3.04m	2.82m (93%)								
				6.05 – 8.06	*54.95m*		*6.05m* SHALE, interbedded with SANDSTONE, dark grey, well developed	SW		6.05 BP 0 PR RF CN 6.09-6.12 DS 6.25 BP 10 CU SM CN 6.56 BP 10 CU SM CN 6.81 DB 7.03 JT 45 CU SM CN 7.16 DB 7.83 DB 7.97-8.06 DS	
		3.09m	2.92m (94%)	8.06 – 10	*52.94m*		*8.06m* SANDSTONE; fine grained, pale grey	SW		8.16 JT 10 PR RF CN 9.26 BP 0 UN RF CN 9.88 DB	

CORED BOREHOLE LOG

BH NO. BH01

Project	Proposed Development	**Sheet**	3 OF 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By** JZ	**Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By** MK	**Date** 08/07/2021

Drilling Contactor GeoSense Drilling & Engineering **RL** **61.0**m AHD
Drill Rig Comacchio GEO205 **Inclination** -90°

Drilling: METHOD	WATER	TCR	RQD	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)
NMLC				10	*51.0m*		SANDSTONE; fine grained, pale grey	SW		10.00-10.05 DB 10.25 DB 10.59-10.60 DS/CS 10.78 BP 0-5 UN RF CN 10.87-10.88 DS	
							10.78-10.88m: SHALE; dark grey				
		3.04m	2.93m (96%)	11						11.18 BP 0 PR RF CN	
				12						12.77 BP 0 PR RF CN	
				13						13.00-13.02 DB 13.08 BP 5 PR RF CN 13.29 DB	
		3.04m	3.04m (100%)	14						14.54 BP 0 PR RF CN 14.78 BP 0 PR RF CN	
				15							
				16			*16.73m*			16.33 DB	
					44.27m		SHALE, interbedded with SANDSTONE, dark grey, well developed	SW		16.73 BP 0 PR SM CN 16.98 DB	
		3.06m	3.06m (100%)	17							
				18			*18.38m*			17.85 DB 18.02 DB 18.22 BP 0 PR SM CN 18.38 BP 0 PR SM CN	
					42.62m		SANDSTONE; fine grained, pale grey	SW			
				19						19.29 DB	
				20	*41.0m*						

Strength scale: VL 0.1 L 0.3 M 1 H 3 VH 10 EH. Defect spacing scale: 30 100 300 1000 3000

CORED BOREHOLE LOG

GEOSENSE DRILLING & ENGINEERING

BH NO. BH01

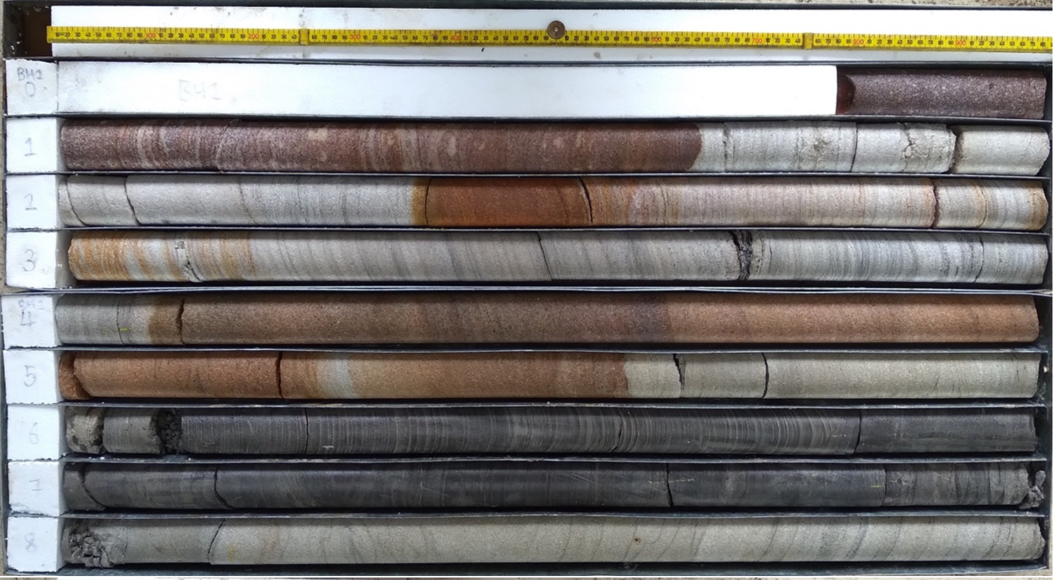
Project	Proposed Development	**Sheet**	4 OF 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By** JZ	**Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By** MK	**Date** 08/07/2021

Drilling Contactor GeoSense Drilling & Engineering **RL** **61.0**m AHD
Drill Rig Comacchio GEO205 **Inclination** -90°

METHOD	WATER	TCR	RQD	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations
NMLC		3.08m	3.08m (100%)	20	41.0m		SANDSTONE; fine grained, pale grey	SW		20.70 DB 21.98 DB 22.37 DB
		3.00m	2.98m (99%)	23–25						24.05 BP 0 PR RF CN 24.28 BP 0 PR RF CN 25.37 DB
		3.04m	2.82m (93%)	26–28			26.2m: interbedded with SHALE laminite			26.26 JT 15 PR RF CN 26.71 BP 10 UN RF CN 27.34 BP 5 PR RF CN 27.50-27.51 DS, clay infill 27.53-27.66 CS/JTs 27.66-27.78 DS, clay infill 28.14 DB 28.24-28.25 DS 28.41 DB 28.65 DB 28.95 DB
		2.09m	2.09m (100%)	29–30	31.0m		Hole Terminated at **30.5m**			29.12 BP 10 UN RF CN

Inferred strength scale: VL 0.1, L 0.3, M 1, H 3, VH 10, EH. Average Defect Spacing (mm): 30, 100, 300, 1000, 3000.

This borehole log should be read in conjunction with Geosense's accompanying explanatory notes.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BH1
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2
3
BH1
4
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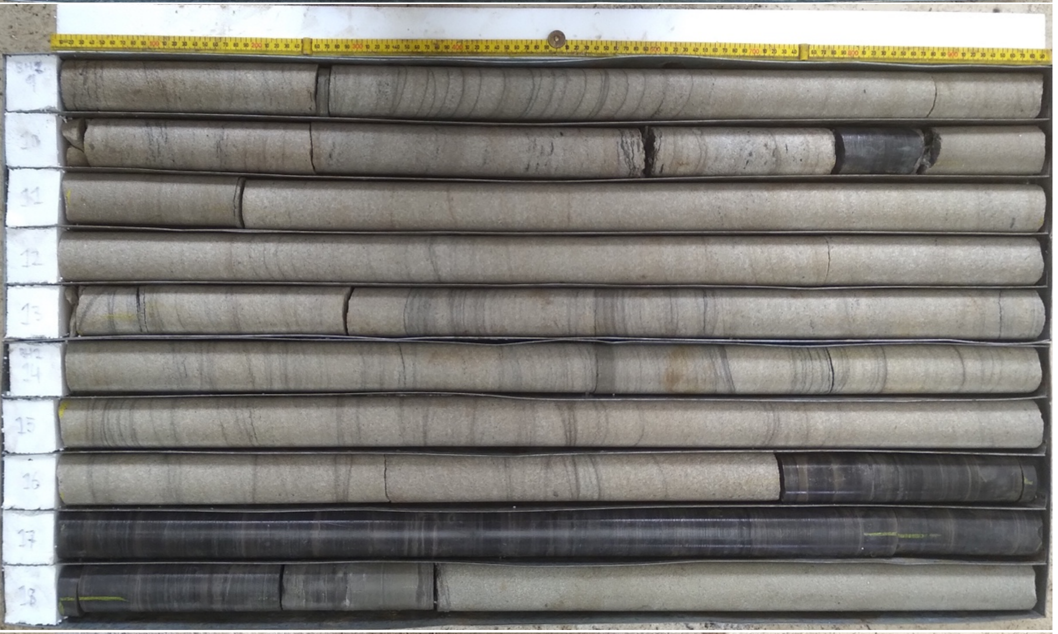
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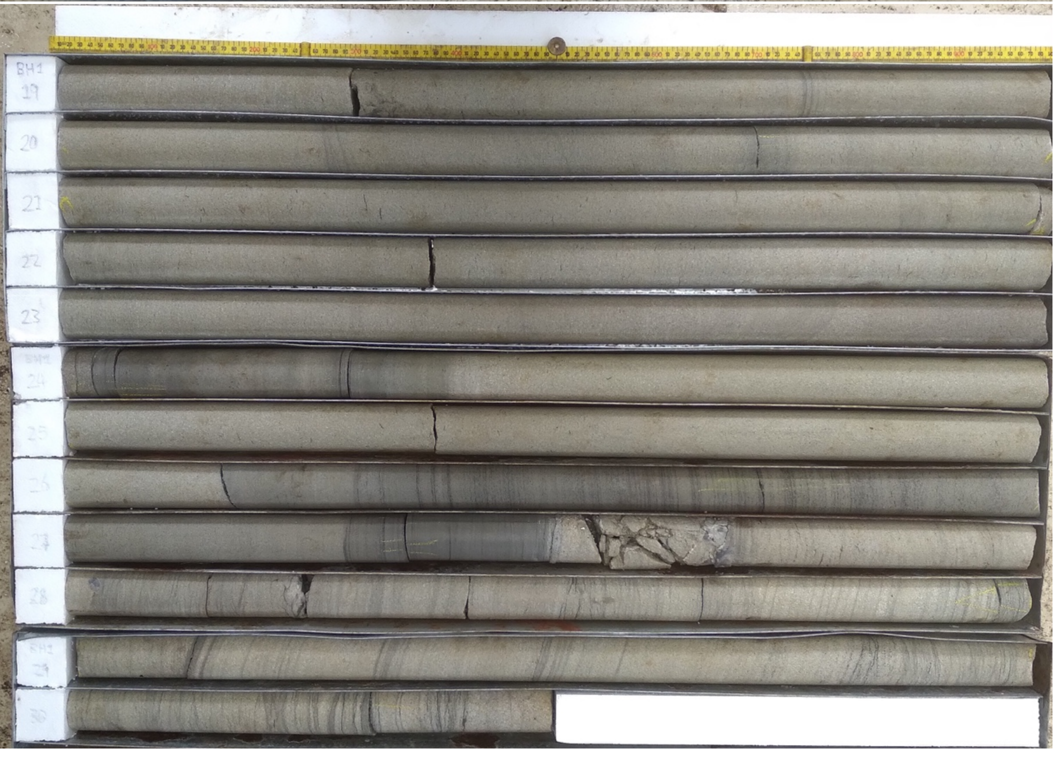
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BH1
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26
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28
BH1
29
30

BH01 Location

BOREHOLE LOG

BH NO. BH02

Project	Proposed Development	**Sheet**	1 of 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By**	JZ **Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By**	MK **Date** 08/07/2021

Drilling Contactor GeoSense Drilling & Engineering **RL** **60.6**m AHD

Drill Rig Comacchio GEO205 **Inclination** -90°

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	*DEPTH* RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	GROUP SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY REL. DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/T	L-M	GWNE	0	*60.6m*				GP	BRICK PAVER/CONCRETE, 150mm ~~Fill; Gravelly SAND; fine to medium grained, dark brown~~	M	-	PAVEMENT / FILL
									0.3m			
				60.3m				CL	Silty CLAY; low to medium plasticity, mottled grey, red-brown and orange brown	w> PL		RESIDUAL
			0.5									
				59.93m					*Start coring at 0.67m*			TC-bit refusal on bedrock
			1									

This borehole log should be read in conjunction with GeoSense's accompanying explanatory notes.

CORED BOREHOLE LOG

BH NO. BH02

Project	Proposed Development	**Sheet**	2 OF 4
Location	13-19 Canberra Avenue, St Leonards NSW	**Date Started**	28/06/2021
Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By** JZ	**Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By** MK	**Date** 08/07/2021

Drilling Contactor	GeoSense Drilling & Engineering	**RL**	**60.6**m AHD
Drill Rig	Comacchio GEO205	**Inclination**	-90°

Drilling (METHOD, WATER, TCR, RQD, DEPTH (metres)): METHOD NMLC

TCR	RQD
1.27m	1.07m (85%)
2.31m	2.12m (92%)
3.06m	2.45m (80%)
2.86m	2.62m (92%)

Field Material Description

DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa (VL 0.1 L 0.3 M 1 H 3 VH 10 EH)
		Start coring at **0.67m**		
59.93m		SANDSTONE; fine grained, mottled pale grey, red-brown and orange-brown	MW	
		5.41m		
55.19m		SHALE, interbedded with SANDSTONE, dark grey, well developed	SW	
		6.40m		
54.2m		SANDSTONE; fine grained, pale grey	SW	
50.6m				

Defect Information

DEFECT DESCRIPTION & Additional Observations

0.82-0.85 DS, clay infill
0.90 BP 0 UN RF CN
0.95 JT 10 CU RF CN
1.18 BP 10 PR RF CN
1.35 BP 10 PR RF CN
1.46-1.47 DS, clay infill
1.59 BP 0 UN RF CN
2.25 BP 10 PR RF CN
2.34 BP 10 PR RF CN
2.44-2.45 DS, clay infill
2.49 BP 10 PR RF CN
2.70-2.71 DS
3.09-3.12 DS
3.37 DB
4.25 DB
4.55-4.60 CS
4.67 JT 30 UN RF CN
5.06 JT 5 UN RF CN
5.12-5.13 DS
5.41 BP 0-5 UN RF CN
5.43 BP 0-5 UN RF CN
5.47 BP 5 UN RF CN
5.65 DB
5.75 BP 10 PR RF CN
5.91-5.95 JT/DS, 45 CU RF CN
6.02 JT 30 UN RF CN
6.12 JT 30 UN RF CN
6.50 BP 0 UN RF CN
6.56-6.58 JT/DS 30 UN RF CN
6.66-6.67 DS
6.88 DB
7.12 DB
7.23 BP 5 UN RF CN
7.31 DB
7.77 JT 5 UN RF CN
7.93 JT 5 UN RF CN
8.05 JT 5 UN RF CN
8.09-8.12 DS, clay infill
9.97 JT 10 UN RF CN

Average Defect Spacing (mm): 30 100 300 1000 3000

CORED BOREHOLE LOG

BH NO. BH02

Project	Proposed Development	**Sheet**	3 OF 4
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Position	Refer to attached location plan	**Date Completed**	29/06/2021
Job No.		**Logged By** JZ	**Date** 29/06/2021
Client	PTC Consulting Engineers	**Reviewed By** MK	**Date** 08/07/2021

Drilling Contactor	GeoSense Drilling & Engineering	**RL**	**60.6**m AHD
Drill Rig	Comacchio GEO205	**Inclination**	-90°

Drilling

METHOD	TCR	RQD	DEPTH (metres)
NMLC			10.0 – 10.25
NMLC	2.98m	2.98m (100%)	10.25 – 13.27
NMLC	3.10m	3.10m (100%)	13.27 – 16.30
NMLC	3.11m	3.09m (99%)	16.30 – 19.27
NMLC			19.27 – 20.0

Field Material Description

DEPTH (m)	RL	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa
10.00	50.6m	SANDSTONE; fine grained, pale grey 10.06-10.17m: SHALE; dark grey, with frequent joints	SW	M
14.51m	46.09m	SHALE, interbedded with SANDSTONE, dark grey, well developed	SW	M
16.50m	44.1m	SANDSTONE; fine grained, pale grey	SW	M
20.00	40.6m			

Defect Information

DEFECT DESCRIPTION & Additional Observations

- 10.96 JT 0-10 CN RF CN
- 11.08 DB
- 11.85 JT 5 UN RF ST
- 11.96 DB
- 12.14 DB
- 13.15 DB
- 13.56 BP 10 UN RF CN
- 13.93 DB
- 14.05 DB
- 14.49 JT 30 CU RF CN
- 14.66 BP 0 PR RF CN
- 15.18 BP 0 PR RF CN
- 15.88 BP 0 PR RF CN
- 15.96 BP 0 PR RF CN
- 16.25 DB
- 16.47-16.49 DS, clay infill
- 17.12 DB
- 19.36 DB

Average Defect Spacing (mm): 30 / 100 / 300 / 1000 / 3000

This borehole log should be read in conjunction with Geosense's accompanying explanatory notes.

CORED BOREHOLE LOG

BH NO. BH02

Project	Proposed Development	**Sheet**	4 OF 4
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Drilling Contactor GeoSense Drilling & Engineering **RL** **60.6**m AHD
Drill Rig Comacchio GEO205 **Inclination** -90°

METHOD	WATER	TCR	RQD	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)
NMLC		2.99m	2.99m (100%)	20–22.3	*40.6m*		SANDSTONE; fine grained, pale grey	SW		20.02 DB 20.18 DB 22.35 DB 22.56 BP 0 PR RF CN	
		3.10m	3.10m (100%)	22.3–25.4						23.26 BP 10 PR RF CN 23.58 BP 10 PR RF CN 24.16 BP 10 PR RF CN 24.54 JT 0-10 UN RF CN 25.45 DB	
		3.05m	2.75m (90%)	25.4–28.5						25.54 JT 5 UN RF CN 25.57-25.60 DS, clay infill 25.85 BP 10 PR RF CN 26.12 JT 0-5 CU RF CN 26.16-26.18 DS, clay infill 26.27 BP 0 UN RF CN 27.23 BP 0 UN RF CN 27.86-27.91 DS 28.27 BP 0-5 UN RF CN 28.33 BP 0-5 UN RF CN 28.50 DB	
		0.90m	0.90m (100%)	28.5–29.4	*31.2m*					28.73 BP 10 PR RF CN	
				30			Hole Terminated at **29.4m**				

Inferred strength scale: VL 0.1, L 0.3, M 1, H 3, VH 10, EH
Average defect spacing scale: 30, 100, 300, 1000, 3000

This borehole log should be read in conjunction with Geosense's accompanying explanatory notes.


17 CANBERRA AVE ST LEONARDS
BH02
29 JUN 2021
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BH2 4m
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BH2 9
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BH2 14
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BH2 19
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BH2 24
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BH2 29

BH02 Location

EXPLAINATION OF NOTES, ABBREVIATIONS & TERMS USED ON BOREHOLE LOGS

DRILLING/EXCAVATION METHOD

HA	Hand Augering	**PT**	Push Tube	**NQ**	Diamond Core - 47 mm
DT	Diatube Coring	**RC**	Reverse Circulation	**NMLC**	Diamond Core - 52 mm
NDD	Non-destructive digging	**JET**	Jetting	**HQ**	Diamond Core - 63 mm
ADS	Solid Flight Auger	**V**	V-Bit	**HMLC**	Diamond Core - 63 mm
ADH	Hollow Flight Auger	**TC**	Tungsten Carbide Bit	**EX**	Tracked Hydraulic Excavator
RM	Rotary Mud	**T**	Tricone Bit	**EE**	Existing Excavation
RA	Rotary Air	**DTH**	Rock Hammer	**HAND**	Excavated by Hand Methods

PENETRATION RESISTANCE

L	**Low Resistance**	Rapid penetration/ excavation possible with little effort from equipment used.
M	**Medium Resistance**	Penetration/ excavation possible at an acceptable rate with moderate effort from equipment used.
H	**High Resistance**	Penetration/ excavation is possible but at a slow rate and requires significant effort from equipment used.
R	**Refusal/Practical Refusal**	No further progress possible without risk of damage or unacceptable wear to equipment used.

These assessments are subjective and are dependent on many factors, including equipment power and weight, condition of excavation or drilling tools and experience of the operator.

WATER

Water level at date shown

Partial water loss

Water inflow

Complete Water Loss

GWNO GROUNDWATER NOT OBSERVED - Observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave-in of the borehole/ test pit.

GWNE GROUNDWATER NOT ENCOUNTERED - Borehole/ test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/ test pit been left open for a longer period.

SAMPLING AND TESTING

SPT	Standard Penetration Test to AS1289.6.3.1-2004
6,8,8 N=16	6,8,8 = Blows per 150mm. N = Blows per 300mm penetration following a 150mm seating drive
30/80mm	Where practical refusal occurs, the blows and penetration for that interval are reported
RW	Penetration occurred under the rod weight only
HW	Penetration occurred under the hammer and rod weight only
HB	Hammer double bouncing on anvil
Sampling	
DS	Disturbed Sample
ES	Sample for environmental testing
CBR	Bulk disturbed Sample used for Californian Bearing Ratio testing
GS	Gas Sample
WS	Water Sample
U50	Thin walled tube sample - number indicates nominal sample diameter in millimetres
In-situ Testing	
FP	Field Permeability test over section noted
FVS	Field Vane Shear test expressed as uncorrected shear strength (sv= peak value, sr= residual value)
PID	Photoionisation Detector reading in ppm
PP	Pocket Penetrometer test expressed as instrument reading in kPa
WPT	Water Pressure tests
DCP	Dynamic Cone Penetrometer test
CPT	Static Cone Penetration test
CPTu	Static Cone Penetration test with pore pressure (u) measurement

ROCK CORE RECOVERY

TCR=Total Core Recovery (%)

$$= \frac{Length\ of\ core\ recovered}{Length\ of\ core\ run} \times 100$$

SCR=Solid Core Recovery (%)

$$= \frac{\sum Length\ of\ cylindrical\ core\ recovered}{Length\ of\ core\ run} \times 100$$

RQD = Rock Quality Designation (%)

$$= \frac{\sum Axial\ lengths\ of\ core > 100mm}{Length\ of\ core\ run} \times 100$$

GEOLOGICAL BOUNDARIES

——————— = Observed Boundary (position known)

– ––––––––– = Observed Boundary (position approximate)

– –?– –?– –?– – = Boundary (interpreted or inferred)

METHOD OF SOIL DESCRIPTION USED ON BOREHOLE LOGS

GEOSENSE DRILLING & ENGINEERING

FILL	ORGANIC SOILS (OL, OH or Pt)	CLAY (CL, CI or CH)
COUBLES or BOULDERS	SILT (ML or MH)	SAND (SP or SW)
GRAVEL (GP or GW)	Combinations of these basic symbols may be used to indicate mixed materials	

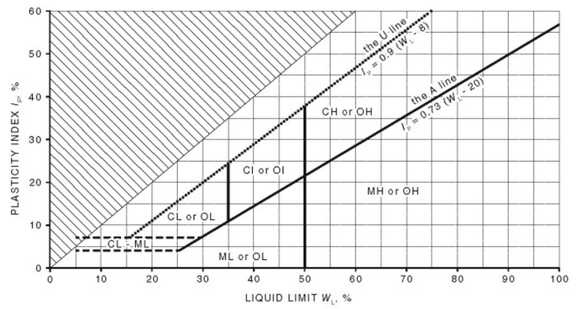
CLASSIFICATION AND INFERRED STRATIGRAPHY

Soil is broadly classified and described in Borehole Logs using the preferred method given in AS 1726:2017, Section 6.1 – Soil description and classification.

PARTICLE SIZE CHARACTERISTICS

Fraction	Components	Sub Division	Size mm
Oversize	BOULDERS		>200
	COBBLES		63 to 200
Coarse grained soil	GRAVEL	Coarse	19 to 63
		Medium	6.7 to 19
		Fine	2.36 to 6.7
	SAND	Coarse	0.6 to 2.36
		Medium	0.21 to 0.6
		Fine	0.075 to 0.21
Fine grained soil	SILT		0.002 to 0.075
	CLAY		<0.002

PLASTICITY PROPERTIES



GROUP SYMBOLS

Major Divisions		Symbol	Description
COARSE GRAINED SOILS More than 65% of soil excluding oversize fraction is greater than 0.075mm	**GRAVEL** More than 50% of coarse fraction is >2.36mm	GW	Well graded gravel and gravel-sand mixtures, little or no fines.
		GP	Poorly graded gravel and gravel-sand mixtures, little or no fines.
		GM	Silty gravel, gravel-sand-silt mixtures.
		GC	Clayey gravel, gravel-sand-clay mixtures.
	SAND More than 50% of coarse fraction is <2.36 mm	SW	Well graded sand and gravelly sand, little or no fines.
		SP	Poorly graded sand and gravelly sand, little or no fines.
		SM	Silty sand, sand-silt mixtures.
		SC	Clayey sand, sandy-clay mixtures.
FINE GRAINED SOILS More than 35% of soil excluding oversized fraction is less than 0.075mm	Liquid Limit less < 50%	ML	Inorganic silts of low plasticity, very fine sands, rock flour, silty or clayey fine sands.
		CL, CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL	Organic silts and organic silty clays of low plasticity.
	Liquid Limit > than 50%	MH	Inorganic silts of high plasticity.
		CH	Inorganic clays of high plasticity.
		OH	Organic clays of medium to high plasticity.
	Highly Organic soil	PT	Peat muck and other highly organic soils.

MOISTURE CONDITION

Symbol	Term	Description
D	Dry	Non- cohesive and free-running.
M	Moist	Soils feel cool, darkened in colour. Soil tends to stick together.
W	Wet	Soils feel cool, darkened in colour. Soil tends to stick together, free water forms when handling.

Moisture content of cohesive soils shall be described in relation to plastic limit (PL) or liquid limit (LL) for soils with higher moisture content as follows: Moist, dry of plastic limit (w < PL); Moist, near plastic limit (w ≈ PL); Moist, wet of plastic limit (w < PL); Wet, near liquid limit (w ≈ LL), Wet, wet of liquid limit (w > LL),

CONSISTENCY

Symbol	Term	Undrained Shear Strength (kPa)	SPT "N" #
VS	Very Soft	≤12	≤2
S	Soft	>12 to ≤25	>2 to ≤4
F	Firm	>25 to ≤50	>4 to 8
St	Stiff	>50 to ≤100	>8 to 15
VSt	Very Stiff	>100 to ≤200	>15 to 30
H	Hard	>200	>30
Fr	Friable	-	

DENSITY

Symbol	Term	Density Index %	SPT "N" #
VL	Very Loose	≤15	0 to 4
L	Loose	>15 to ≤35	4 to 10
MD	Medium Dense	>35 to ≤65	10 to 30
D	Dense	>65 to ≤85	30 to 50
VD	Very Dense	>85	Above 50

In the absence of test results, consistency and density may be assessed from correlations with the observed behaviour of the material.
SPT correlations are not stated in AS1726:2017, and may be subject to corrections for overburden pressure and equipment type.

MINOR COMPONENTS

Term	Assessment Guide	Proportion by Mass
Trace	Presence just detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: ≤ 5% Fine grained soil: ≤15%
With	Presence easily detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: 5 - 12% Fine grained soil: 15 - 30%
Prefix	Presence easily detectable by feel or eye in conjunction with the general properties of primary component	Coarse grained soils: >12% Fine grained soil: >30%

TERMS FOR ROCK MATERIAL STRENGTH AND WEATHERING

CLASSIFICATION AND INFERRED STRATIGRAPHY

Rock is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 2017, Section 6.2 – Rock identification, description and classification.

ROCK MATERIAL STRENGTH CLASSIFICATION

Symbol	Term	Point Load Index, $Is_{(50)}$ (MPa) #	Field Guide
VL	Very Low	0.03 to 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 30 mm can be broken by finger pressure.
L	Low	0.1 to 0.3	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of pick point; has dull sound under hammer. A piece of core 150 mm long by 50 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
M	Medium	0.3 to 1	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.
H	High	1 to 3	A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow; rock rings under hammer.
VH	Very High	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
EH	Extremely High	>10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

 # Rock Strength Test Results

▼ Point Load Strength Index, $Is_{(50)}$, Axial test (MPa)

● Point Load Strength Index, $Is_{(50)}$, Diametral test (MPa)

Relationship between rock strength test result ($Is_{(50)}$) and unconfined compressive strength (UCS) will vary with rock type and strength, and should be determined on a site-specific basis. However UCS is typically 20 x $Is_{(50)}$.

ROCK MATERIAL WEATHERING CLASSIFICATION

Symbol		Term	Field Guide
RS		Residual Soil	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
XW		Extremely Weathered	Rock is weathered to such an extent that it has soil properties - i.e. it either disintegrates or can be remoulded, in water.
DW	HW	Distinctly Weathered	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores. In some environments it is convenient to subdivide into Highly Weathered and Moderately Weathered, with the degree of alteration typically less for MW.
	MW		
SW		Slightly Weathered	Rock slightly discoloured but shows little or no change of strength relative to fresh rock.
FR		Fresh	Rock shows no sign of decomposition or staining.

ABBREVIATIONS AND DESCRIPTIONS FOR ROCK MATERIAL AND DEFECTS

GEOSENSE DRILLING & ENGINEERING

CLASSIFICATION AND INFERRED STRATIGRAPHY

Rock is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 2017, Section 6.2 – Rock identification, description and classification.

ROCK MATERIAL DESCRIPTION

Layering

Term	Description
Massive	No layering apparent
Poorly Developed	Layering just visible; little effect on properties
Well Developed	Layering (bedding, foliation, cleavage) distinct; rock breaks more easily parallel to layering

Structure

Term	Spacing (mm)
Thinly laminated	<6
Laminated	6 – 20
Very thinly bedded	20 – 60
Thinly bedded	60 – 200
Medium bedded	200 – 600
Thickly bedded	600 – 2,000
Very thickly bedded	> 2,000

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT TYPES

Defect Type	Abbr.	Description
Joint	JT	Surface of a fracture or parting, formed without displacement, across which the rock has little or no tensile strength. May be closed or filled by air, water or soil or rock substance, which acts as cement.
Bedding Parting	BP	Surface of fracture or parting, across which the rock has little or no tensile strength, parallel or sub-parallel to layering/ bedding. Bedding refers to the layering or stratification of a rock, indicating orientation during deposition, resulting in planar anisotropy in the rock material.
Foliation	FL	Repetitive planar structure parallel to the shear direction or perpendicular to the direction of higher pressure, especially in metamorphic rock, e.g. Schistosity (SH) and Gneissosity.
Contact	CO	The surface between two types or ages of rock.
Cleavage	CL	Cleavage planes appear as parallel, closely spaced and planar surfaces resulting from mechanical fracturing of rock through deformation or metamorphism, independent of bedding.
Sheared Seam/ Zone (Fault)	SS/SZ	Seam or zone with roughly parallel almost planar boundaries of rock substance cut by closely spaced (often <50 mm) parallel and usually smooth or slickensided joints or cleavage planes.
Crushed Seam/ Zone (Fault)	CS/CZ	Seam or zone composed of disoriented usually angular fragments of the host rock substance, with roughly parallel near-planar boundaries. The brecciated fragments may be of clay, silt, sand or gravel sizes or mixtures of these.
Decomposed Seam/ Zone	DS/DZ	Seam of soil substance, often with gradational boundaries, formed by weathering of the rock material in places.
Infilled Seam	IS	Seam of soil substance, usually clay or clayey, with very distinct roughly parallel boundaries, formed by soil migrating into joint or open cavity.
Schistocity	SH	The foliation in schist or other coarse grained crystalline rock due to the parallel arrangement of platy or prismatic mineral grains, such as mica.
Vein	VN	Distinct sheet-like body of minerals crystallised within rock through typically open-space filling or crack-seal growth.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT SHAPE AND ROUGHNESS

Shape	Abbr.	Description	Roughness	Abbr.	Description
Planar	Pl	Consistent orientation	Polished	Pol	Shiny smooth surface
Curved	Cu	Gradual change in orientation	Slickensided	SL	Grooved or striated surface, usually polished
Undulating	Un	Wavy surface	Smooth	S	Smooth to touch. Few or no surface irregularities
Stepped	St	One or more well defined steps	Rough	RF	Many small surface irregularities (amplitude generally <1mm). Feels like fine to coarse sandpaper
Irregular	Ir	Many sharp changes in orientation	Very Rough	VR	Many large surface irregularities, amplitude generally >1mm. Feels like very coarse sandpaper

Orientation: **Vertical Boreholes –** The dip (inclination from horizontal) of the defect.
Inclined Boreholes – The inclination is measured as the acute angle to the core axis.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT COATING

Coating	Abbr.	Description
Clean	CN	No visible coating or infilling
Stain	SN	No visible coating but surfaces are discoloured by staining, often limonite (orange-brown)
Veneer	VNR	A visible coating of soil or mineral substance, usually too thin to measure (< 1 mm); may be patchy

DEFECT APERTURE

Aperture	Abbr.	Description
Closed	CL	Closed.
Open	O	Without any infill material.
Infilled	-	Soil or rock i.e. clay, talc, pyrite, quartz, etc.

STS GEOTECHNICS PTY LTD
CONSULTING GEOTECHNICAL ENGINEERS

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NATA Accredited for Compliance with ISO/IEC 17025 - Testing No. 2750

Point Load Strength Index Report

Project: Canberra Avenue, St Leonards, NSW	Project No.: 31302/5317D-L
Client: GEOSENSE DRILLING AND ENGINEERING	Report No.: 21/2064
Address: 32 THIRD AVENUE, BERALA	Report Date: 8/07/2021
Test Method: AS4133.4.1	Page: 1 of 2

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 05/07/2021

Borehole No. 1

Depth	Test Type	Is(50) (Mpa)	Rock Type	Failure Type	Moisture
1.43	A	**1.80**	SS	3	M
2.64	A	**0.56**	SS	3	M
3.32	A	**1.00**	SS	3	M
4.33	A	**0.85**	SS	3	M
5.47	A	**0.87**	SS	3	M
6.61	A	**0.68**	SH	3	M
7.41	A	**0.48**	SH	3	M
8.46	A	**1.90**	SS	3	M
9.42	A	**1.50**	SS	3	M
10.71	A	**2.50**	SS	3	M
11.32	A	**2.00**	SS	3	M
12.58	A	**2.10**	SS	3	M
13.43	A	**1.90**	SS	3	M
14.32	A	**1.80**	SS	3	M
15.60	A	**1.50**	SS	3	M

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 05/07/2021

Borehole No. 1

Depth	Test Type	Is(50) (Mpa)	Rock Type	Failure Type	Moisture
16.52	A	**1.40**	SS	3	M
17.50	A	**1.10**	SH	3	M
18.51	A	**0.96**	SS	3	M
19.40	A	**1.00**	SS	3	M
20.32	A	**1.30**	SS	3	M
21.57	A	**1.30**	SS	3	M
22.44	A	**1.30**	SS	3	M
23.53	A	**1.50**	SS	3	M
24.48	A	**1.20**	SS	3	M
25.66	A	**1.10**	SS	3	M
26.41	A	**2.20**	SS	3	M
27.35	A	**1.90**	SS	3	M
28.51	A	**1.30**	SS	3	M
29.37	A	**1.30**	SS	3	M
30.14	A	**1.30**	SS	3	M

FAILURE TYPE
1= FRACTURE THROUGH BEDDING OR WEAK PLANE
2= FRACTURE ALONG BEDDING
3= FRACTURE THROUGH ROCK MASS
4= FRACTURE INFLUENCED BY NATURAL DEFECT OR DRILLING
5= PARTIAL FRACTURE OR CHIP (INVALID RESULT)

TEST TYPE
A= AXIAL
D= DIAMETRAL
I= IRREGULAR
C= CUBE

MOISTURE CONDITION
W= WET
M= MOIST
D= DRY

ROCK TYPE
SS= SANDSTONE
ST= SILTSTONE
SH= SHALE
YS= CLAYSTONE
IG= IGNEOUS

Remarks:

Approved Signatory...........................

Technician: FV

STS GEOTECHNICS PTY LTD
CONSULTING GEOTECHNICAL ENGINEERS

STS Geotechnics Pty Ltd
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Phone: (02)9756 2166 | Email: enquiries@stsgeo.com.au

NATA Accredited for Compliance with ISO/IEC 17025 - Testing No. 2750

Point Load Strength Index Report

Project: Canberra Av, St Leonards, NSW	Project No.: 31302/5317D-L
Client: GEOSENSE DRILLING AND ENGINEERING	Report No.: 21/2064
Address: 32 THIRD AVENUE, BERALA	Report Date: 8/07/2021
Test Method: AS4133.4.1	Page: 2 of 2

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 05/07/2021

Borehole No. 2

Depth	Test Type	**Is(50) (Mpa)**	Rock Type	Failure Type	Moisture
0.70	A	**1.70**	SS	3	M
1.77	A	**0.78**	SS	3	M
2.60	A	**0.59**	SS	3	M
3.50	A	**1.10**	SS	3	M
4.20	A	**0.66**	SS	3	M
5.51	A	**0.76**	TS	3	M
6.46	A	**2.00**	SH	3	M
7.50	A	**1.30**	SS	3	M
8.50	A	**1.70**	SS	3	M
9.50	A	**2.40**	SS	3	M
10.50	A	**1.90**	SS	3	M
11.05	A	**2.40**	SS	3	M
12.50	A	**2.50**	SS	3	M
13.15	A	**2.40**	SS	3	M
14.63	A	**0.96**	SH	3	M

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 05/07/2021

Borehole No. 2

Depth	Test Type	**Is(50) (Mpa)**	Rock Type	Failure Type	Moisture
15.39	A	**1.10**	SH	3	W
16.70	A	**1.10**	SS	3	W
17.12	A	**1.20**	SS	3	W
18.50	A	**1.40**	SS	3	W
19.50	A	**1.50**	SS	3	W
20.50	A	**1.40**	SS	3	W
21.50	A	**1.40**	SS	3	M
22.52	A	**1.40**	SS	3	W
23.58	A	**1.40**	SS	3	W
24.60	A	**1.30**	SS	3	W
25.20	A	**1.70**	SS	3	W
26.50	A	**1.80**	SS	3	W
27.60	A	**1.50**	SS	3	W
28.32	A	**1.50**	SS	3	M
29.00	A	**1.50**	SS	3	M

FAILURE TYPE
1= FRACTURE THROUGH BEDDING OR WEAK PLANE
2= FRACTURE ALONG BEDDING
3= FRACTURE THROUGH ROCK MASS
4= FRACTURE INFLUENCED BY NATURAL DEFECT OR DRILLING
5= PARTIAL FRACTURE OR CHIP (INVALID RESULT)

TEST TYPE
A= AXIAL
D= DIAMETRAL
I= IRREGULAR
C= CUBE

MOISTURE CONDITION
W= WET
M= MOIST
D= DRY

ROCK TYPE
SS= SANDSTONE
ST= SILTSTONE
SH= SHALE
YS= CLAYSTONE
IG= IGNEOUS

Remarks:

Approved Signatory..........................

Technician: ZW

PUMPOUT TEST - No.15 Canberra Avenue, St Leonards
19 Jul 2021

Monitoring well construction details:

- Bottom of well = 18.0m Below Existing Ground Level (BEGL)
- Well screened from 6.0m to 18.0m BEGL, entirely within Sandstone bedrock
- Top of screening sand = 5.5m BEGL
- Top of bentonite plug = 4.5m BEGL
- Grout filled the rest of the annulus and a steel gatic cover level with the ground

Initial groundwater level measured prior to start of test = 7.50m BEGL (*RL 53.50m*)

Groundwater pumped out from inside the monitoring well, aprox. 35L of water

Test results:

Time of measurement	**Duration (seconds)**	**Depth to Groundwater BEGL (m)**	**Groundwater RL (m AHD) – existing ground level = RL 61.0m**
12:00	0	16.03	*44.97*
12:05	300	15.73	*45.27*
12:10	600	15.39	*45.61*
12:20	1200	14.63	*46.37*
12:30	1800	14.05	*46.95*
13:00	3600	12.16	*48.84*
14:00	7200	9.88	*51.12*
15:00	10800	8.50	*52.50*
16:00	14400	8.05	*52.95*
17:00	18000	7.76	*53.24*