SPECIALIST ADVICE TO

EDSGEAR PTY LTD

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Date: 27 March 2025
Ref: 37122Lrptrev1

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1 INTRODUCTION

This report presents the results of a preliminary and limited scope geotechnical investigation for the proposed mixed-use development at 28-38 Pacific Highway, St Leonards, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Wolfgang Ripberger of Tonkin Zulaikha Greer Architects on behalf of the clients Edsgear Pty Ltd. The commission was on the basis of our fee proposal, Ref: P70198L, dated 19 September 2024.

We have been provided with the following documents;

- Architectural drawings prepared by Tonkin Zulaikha Greer Pty Ltd (Job No. 20019, Drawings No's. A100-A113, A200-A203, and A300-A301, , Issue For DA, all dated 19 March 2025),
- A site survey plan prepared by Axiom Spatial Pty Ltd (Drawing No. 23623007, Revision 1, dated 24 September 2024).
- Survey Drawings of the Basement 2 carpark for 38 Pacific Highway, drawn by Axiom Spatial Pty Ltd, Drawing Number 23623002, dated 12 January 2024.

Based on the provided information, we understand that the proposed development will include demolition of the existing buildings within 28-38 Pacific Highway, and construction of a new multi-level mixed-use development overlying two basement car parking levels. The lowest basement car parking level will have a finished floor level at RL68.9m. Along the north-western (Pacific Highway) side of the site, excavation to achieve the basement 2 level will be to maximum depths of about 10m below the Pacific Highway footpath level, while at the south-eastern (Marshall Lane) side of the site excavation will be to a maximum of about 7m below the Marshall Lane level. We also note that the existing commercial building that occupies 38 Pacific Highway has basement levels, with the lower level at approximately RL72.7m whereas the buildings that occupy 28-32 Pacific Highway are constructed at ground level approximately RL75.5m.

The purpose of this investigation was to obtain preliminary subsurface information on the subsurface conditions in order to provide a preliminary geotechnical report assessing the site's geotechnical suitability for the proposed development. This report also provides our preliminary comments and recommendations on geotechnical aspects of the proposed development, including excavation conditions, shoring methods, groundwater management, footings and basement slabs.

Access to the site for subsurface investigations was limited and therefore only a single geotechnical borehole could be drilled. Therefore additional geotechnical investigations will be required once demolition is completed to provide more suitable site coverage and more specific geotechnical advice for detailed design.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 28 October 2024, and comprised the drilling of one borehole (BH1) using our track-mounted JK308 drill rig. The borehole was initially advanced through soil and the upper extremely weathered bedrock using a spiral auger with a Tungsten Carbide (TC) bit. The borehole was then advanced to a total depth of 18.06m below existing surface levels using rotary diamond coring techniques with an NMLC core barrel and water flush.

The borehole location, is shown on Figure 2, was it was set out by tape measurements from surface features shown on the site survey plan by Axiom Spatial Pty Ltd (Drawing No. 23623007, Revision 1, dated 24 September 2024). An approximate surface level was interpolated from spot levels on the above referenced survey plan. The height datum is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength of the residual clay was assessed from Standard Penetration Tests (SPT) 'N' values, augmented by hand penetrometer tests on cohesive samples recovered from the SPT split spoon sampler. The strength of the upper weathered bedrock in the augered portion of the borehole was assessed from observation of the drilling resistance using a Tungsten Carbide 'TC' drill bit attached to the augers, examination of rock cuttings and correlation with the results of subsequent laboratory moisture contents. It should be noted that rock strength assessments in this way are approximate and variances of at least one strength order should not be unexpected.

For the cored portion of the bedrock, the recovered core was returned for photographing and Point Load Strength Index ($Is_{(50)}$) testing. Established correlations were used to estimate the unconfined compressive strength (UCS) of the bedrock. Photographs of the core are presented with the borehole logs, while the Point Load Strength Index test results are summarised on the borehole logs and presented in Table B.

During drilling, groundwater observations were made in the borehole during augering and immediately upon completion. As water is used during core drilling, water levels observed post-coring were not recorded, as they may reflect an artificially elevated level. At completion of drilling, strata-pack drilling was used to 'ream' the borehole to provide a sufficient anulus for installation of a 50mm Class 5 PVC groundwater monitoring standpipe. After installation, water was pumped out of the standpipe to clear any drilling water. We returned to site on 5 November 2024 to take a measurement of the groundwater level.

The fieldwork was completed under the full-time supervision of our geotechnical engineer, Mr. Salvatore Wedde, who managed borehole setup, in-situ testing, sampling, groundwater monitoring well installation and borehole log preparation. The borehole logs, including colour photographs of the recovered core, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for Atterberg Limit testing on cohesive soil samples and Moisture Content testing on recovered rock chips, the results of which are presented in the attached STS Table A.

Selected soil samples were sent to Envirolab Services Pty Ltd, a NATA-registered laboratory, for testing, including soil pH, sulfate, chloride, and resistivity. Results are detailed in Section 3.3 below and in the attached Envirolab Certificate of Analysis No. 365177.

No contamination testing was conducted, as this was outside the scope of the current investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within a region of undulating topography, located towards the toe of a south-west sloping hillside that slopes down at around 9°, with an elevation change of around 4m between the north-west and south-east site boundaries. The site is approximately trapezoidal in shape and bounded by Pacific Highway to the north-west, Marshall Lane to the south-east, and neighbouring properties to the north-east and south-west. Marshall Lane grades down to the north-east at around 5° and levels out in line with No. 22 Pacific Highway, before continuing to slope up to the north-east at around 1-2°.

At the time of investigation, the eastern portion of the site comprised 2-storey commercial brick buildings (No. 28-32 Pacific Highway). These buildings abut the north-western (Pacific Highway) boundary and are setback around 12m to 15m from the south-east boundary of Marshall Lane, with concreted carparks at the rear. No. 30 and No. 32 are adjoining terraces, and No. 28 adjoins the Neighbouring No. 26. The rear carpark sloped up from Marshall Lane towards the buildings at around 5°. The brick buildings and concrete carparks were in good external condition, based upon a cursory inspection.

The western portion of the site comprises a 5-storey rendered brick commercial building (No. 38 Pacific Highway). Based upon a provided survey plan prepared by Axiom Spatial Pty Ltd (Drawing No. 23623002, Revision 00, dated 12 January 2023), along with observations made on site, the building at No. 38 contained two basement carpark levels, the upper Basement 1 level is at around RL75.5m (close to street level at Marshall Lane), and the lower Basement 2 level is at about RL72.7m (around 3m below the street level at Marshall Lane). The basement levels at No. 38 appear to extend to the lot boundaries, although we were unable to access the basement levels to confirm this during the fieldwork period. The building appeared in good condition, based upon a cursory inspection.

To the south-west, No. 46 Pacific Highway, is a multi-storey brick commercial building, with a basement carpark with street level access off Marshall Lane. Access was not possible into the basement to confirm if there were any other basement levels. The building appeared in good external condition, based upon a cursory inspection.

To the north-east of the site, No.26 Pacific Highway, is a two-storey brick commercial building adjoining No.28 via a common party wall. It is also set back approximately 12m to 15m from the rear Marshall Lane boundary and has a concrete carpark at the rear of the property. Along the shared boundary at the rear there is a concrete curb that is cracked. A small tree is located along the boundary, on the side of No.28.

3.2 Subsurface Conditions

The NSW Seamless Geology version 2.4 indicates that the site is underlain by Ashfield Shale, but is close to the geological boundary with Hawkesbury Sandstone.

The single borehole has shown a generalised subsurface profile comprising silty clay fill and residual silty clay soil underlain by sandstone bedrock at relatively shallow depth. While sandstone bedrock has been encountered in the single borehole drilled, it is quite possible based on available geological maps that the site may contain variable bedrock conditions with weathered siltstone bedrock encountered in other areas of the site. A summary of the encountered subsurface conditions in BH1 is presented below. However, reference should be made to the attached borehole log for a detailed description of the subsurface conditions.

Pavement and Fill

A 90mm thick concrete pavement was initially encountered at BH1. Below the concrete pavement, fill comprising silty clay of medium plasticity was encountered and it extended to a depth of 1.3m. Fine grained igneous and ironstone gravels and fine to medium grained sand were observed within the fill. Based on the SPT N values and Hand Penetrometer Readings, the silty clay fill was assessed to be poorly compacted.

Residual Clays

Residual silty clay of medium plasticity was encountered below the fill and extended to the underlying sandstone bedrock. The residual clays were initially of stiff strength becoming hard at a depth of 1.8m. The strength of the clays was assessed by hand penetrometer testing on recovered SPT samples.

Sandstone Bedrock

The top of sandstone bedrock was encountered at a depth of 3.0m. The upper 0.4m of sandstone was extremely weathered and of hard soil strength. We note that extremely weathered sandstone is a material with soil like properties. Below this initial extremely weathered sandstone layer the sandstone was assessed to be of medium and then low strength down to 4.78m, where a further layer of extremely weathered sandstone underlain by a 0.62m thick core loss zone (which is likely an extremely weathered rock or clay seam washed out by drill flush water) was encountered. Very low to low strength sandstone with a further 0.43m thick core loss was then encountered between 5.86m and 9.23m. At 9.23m, the rock strength improved to low to medium strength, with better quality medium to high strength rock encountered below 10.66m which continued to the borehole termination depth.

Groundwater

We returned to site on the 5 November 2024 and measured the groundwater within the installed monitoring well at 4.55m below current surface levels or at approximate RL70.8m. No longer term groundwater monitoring has been carried out.

3.2.1 Subsurface Conditions on Adjoining Site to the South

JK Geotechnics has undertaken previous investigations on a site immediately to the south. That investigation also encountered a relatively shallow soil profile comprising residual silty clays overlying sandstone bedrock. Some of the residual silty clays on the adjoining site to the south were of medium to high plasticity. The sandstone bedrock on the site to the south was generally consistent with BH1, showing poorer quality and lower strength rock down to about 9.2m where medium and high strength sandstone was encountered. However one of the boreholes at the south-western end, just beyond where the subject site extends encountered poorer quality and lower strength rock to at least 11m depth. Therefore it is possible that poorer quality rock may be encountered over the western portions of the site.

Groundwater levels on the adjoining site were measured at reduced levels ranging from about RL72.7m to RL66.7m with a general drop in groundwater level to the south-east.

3.3 Laboratory Test Results

The results of the Atterberg Limits tests completed on samples of silty clay fill and residual silty clay are presented in Table A, and these tests confirmed the silty clay fill and residual silty clays to be of medium plasticity, and therefore they will have moderate shrink/swell potential with changes in moisture content.

The results of the Point Load Strength Index tests and moisture content test results correlated well with the field assessed rock strengths. The results of the point load strength index tests on the sandstone indicate that the calculated Unconfined Compressive Strength (UCS) of the upper poorer quality and lower strength weathered sandstone ranges from 2MPa to 12MPa, while the better quality sandstone below about 10.6m ranges from 10MPa to 32MPa. The estimated UCS's, are based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (ie. UCS = 20 x $I_{S(50)}$).

The results of soil aggression testing for the silty clay fill and residual silty clay are tabulated below:

Borehole	Sample Depth (m)	pH	Chloride (mg/kg)	Sulfate (mg/kg)	Resistivity (ohm.cm)
BH1	0.2-0.4 (FILL)	5.9	<10	22	34000
	0.5-0.95 (FILL)	6.0	<10	<10	40000
	2.5-2.7 (RESIDUAL)	5.0	<10	37	35000

Based on the soil aggression test results we consider that the soils should be designed using a 'Mild' exposure classification for concrete and a 'Non-Aggressive' exposure classification for steel piles, in accordance with Table 6.4.2(C) and Table 6.5.2(C) of AS2159-2009 *'Piling – Design and Installation'*.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Considerations

Our investigations have been limited to a single borehole due to existing buildings on the site. Notwithstanding, based on the results of that borehole, subsurface investigations on adjoining sites, and review of geological maps, we consider that from a geotechnical perspective, the site is suitable for the proposed development. Further geotechnical investigations will be required once access is possible following demolition. The comments and recommendations below are of a preliminary nature and may be used for preliminary concept designs. The comments and recommendations below will need to be updated once additional geotechnical investigations are carried out. We consider that there are a number of geotechnical considerations for this site, and these will need to be addressed as detailed design is developed and following further geotechnical investigations. These are discussed briefly below and in more detail in the following sections of this report.

- The investigations to date include only a single borehole. The borehole encountered a weathered sandstone bedrock. As discussed in Section 3.2, the geological maps indicate that the site should be within an area underlain by Ashfield Shale, although it is relatively close to the boundary with the Hawkesbury Sandstone. Therefore it is quite possible that additional geotechnical investigations will encounter siltstone bedrock as part of the Ashfield Shale geological unit within other areas of the site.
- Groundwater has been encountered within the depth of excavation. Therefore dewatering will need to be carried out in accordance with the requirements of Water NSW, and further investigations and groundwater monitoring in accordance with the Water NSW document "*Minimum Requirements for Building Site Groundwater Investigations and Reporting*" dated October 2022 will be required.
- The site is located adjacent to the Pacific Highway and therefore Transport for NSW will also require investigations to be carried out in accordance with their Technical Direction - Geotechnology GTD2020/001 Version 01, dated 2020.
- The borehole indicated the presence of an upper soil profile and then quite poor quality and lower strength sandstone bedrock down to a depth of about 10.6m (RL64.7). Therefore the excavation for the basement excavation will require full height shoring systems to be installed to below bulk excavation level.
- The existing commercial building on the site (NO. 38 Pacific Highway) has basement levels. It appears like the basement levels extend right up to the site boundaries of No. 38. Therefore consideration will need to be given to the type and location of the existing retaining walls around the perimeter and how these will impact on construction of new shoring walls to support the proposed excavations. Further details of existing retaining walls will need to be obtained.
- The adjoining building to the south-west, No. 46 Pacific Highway, is a multi-storey brick commercial building, with at least one basement level, but possibly more. Further assessment of the depth and extent of any basements below this adjoining building should be determined as it will also impact on shoring designs.
- The site is located some 100m or so from the North Shore train line and about 250m from the North-West Metro tunnel. Therefore the proposed development works on this site will have no impact on these assets.

4.2 Dilapidation Reports

Prior to the commencement of any site work, we recommend that detailed dilapidation survey reports be compiled on the neighbouring buildings to the north-east (No. 26 Pacific Highway), and the south-west (46 Pacific Highway). The dilapidation reports can be used as a benchmark against which to set vibration limits for trafficking of plant and rock excavation, and for assessing possible future claims for damage arising from the works due to the excavation generally. Dilapidation reports may also be required by the authorities of assets adjacent to the site such as the Pacific Highway footpath (Transport for NSW), Marshall Lane (Council) and sensitive water bearing assets (Sydney Water).

The respective owners of the neighbouring properties should be asked to confirm in writing that the dilapidation reports present a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly by reputable companies with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc) and photographed. The dilapidation survey reports should be reviewed by JK Geotechnics (JKG).

4.3 Excavation Conditions

The following recommendations should be read in conjunction with the latest version of *'Excavation Work – Code of Practice'* prepared by SafeWork NSW.

The proposed Basement 2 level will have a finished floor level at RL68.9m, which will require excavation to depths ranging from about 10m below the Pacific Highway footpath level, and about 7m below the Marshall Lane level. Although, based on the existing basement levels within 38 Pacific Highway (which is approximately RL72.7m), the actual excavation depth in this portion of the site will probably only be about 4m deep. Locally deeper excavations may be required for proposed lift pits or services trenches.

Based on the investigation results, excavation to these depths will encounter the fill, residual soils and weathered sandstone bedrock. The weathered sandstone in the excavation profile is generally anticipated to comprise very low to low strength rock, with some medium strength bands and with a general increase in strength with depth. Excavation of the soils and any extremely weathered sandstone should be readily achievable using the buckets of medium to large sized hydraulic excavators. Any very low to low and low strength sandstone will require rock excavation techniques, such as ripping tynes fitted to medium sized dozers or ripping tynes fitted to hydraulic excavators. Where sandstone bedrock of medium or higher strength is encountered this will present 'harder' excavation conditions which will require excavation using equipment such as hydraulic impact hammers or ripping tynes on heavy excavators or larger dozers.

Rock excavation using hydraulic impact hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to adjoining structures. We note that the adjoining terrace structure to the north-east (No. 26 Pacific Highway) is likely to be quite sensitive to vibrations. Therefore allowance should be made for some full-time quantitative vibration monitoring on adjoining structures to both the north-east and south-west. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation. If during excavation it is confirmed that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a smaller rock hammers, rock saws or rock grinders. Such techniques will almost certainly be required immediately adjacent to the adjoining structures, particularly No. 26 Pacific Highway. Reference should be made to the attached Vibration Emission Design Goals for further details.

Where percussive excavation techniques are used, the vibration limits that should be adopted on this site are presented in the Vibration Emission Design Goals which is attached to the rear of this report. The limits are dependent on the frequency of the vibrations and the type of structure. Subject to review of the dilapidation reports, we recommend that vibrations, measured as Peak Particle Velocity (PPV), on the neighbouring buildings, be limited to no higher than 5mm/sec.

We recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. Excavation contractors should be provided with a copy of this geotechnical report, including the borehole logs and point load strength test results, so that they can make their own assessment of suitable excavation equipment.

4.4 Retention Systems

Based on the current basement general arrangement drawings, the basements will extend up to the existing site boundaries. Therefore temporary batter slopes will not be feasible and all excavations will need to be supported by properly designed insitu retention systems installed prior to excavation commencing.

We consider that anchored soldier pile walls with reinforced shotcrete infill panels will be suitable at least along the Pacific Highway and Marshall Lane property boundaries, unless there are particularly sensitive services in the roads or footpath areas. Adjacent to No. 26 Pacific Highway, and possibly also adjacent to the adjoining structure at 46 Pacific Highway, more rigid continuous piled walls (anchored or propped) may be required to reduce the risk of damage to these adjoining structures from shoring wall deflections. As discussed above further information on the adjoining basement extent and depth will need to be obtained to confirm the most suitable shoring system along this boundary.

The site at 38 Pacific Highway currently has two existing basement levels that will be supported by an existing retaining wall. Given this, the new shoring system for the proposed development must be designed in a way that does not interfere with the existing system. Any damage or demolition of the current retaining system prior to installing the new shoring wall may lead to instability along the property boundaries. One approach could be to place the proposed shoring system inside the existing retaining system (further within the proposed footprint), though this may result in a smaller basement footprint which may not be preferred. Alternatively, careful staged demolition of the existing retaining walls and construction of the new shoring walls may be feasible. Notwithstanding it will be essential that further investigations are undertaken to determine the nature of the existing basement retaining walls so that a considered construction methodology plan can be developed. The existing basement walls and any portion of the existing structure providing the existing walls with lateral support, should not be demolished without approval from the structural engineers.

Shoring walls will need to extend a minimum of at least 1.5m below the bulk excavation level, including allowances for localised excavations within the basement. Temporary lateral support should be provided by anchors or internal props, with lateral support provided progressively as each restraining point is uncovered. Permission will need to be obtained from the owners of the adjoining properties and roadways before installation of anchors below those properties. Such permission can take time to obtain and we recommend that the permission be sought as early as possible to allow time for negotiation. Permanent lateral support would be provided by the floor slabs inside the basement. We note that anchors may not be feasible along the boundary with No. 46 Pacific Highway where basements exist.

Drainage should be provided behind all retaining walls in the form of strip drains behind shotcrete panels at not greater than 1.5m intervals, or weep holes at approximately 1.5m horizontal and vertical centres through contiguous piled walls. Weep holes would comprise 50mm diameter PVC tubes which are protected at the rear by geofabric to reduce the risk of loss of material through the sweep holes.

4.4.1 Retaining Wall Design Parameters

Propped or anchored retaining walls may be preliminarily designed using a trapezoidal earth pressure distribution of 6H kPa or 8H kPa, where H is the retained height of soils and weathered sandstone of lower than medium strength. A pressure of 8H kPa should be used adjacent to movement sensitive buildings and services, while a pressure of 6H kPa may be used where some movement of the shoring system can be tolerated. The trapezoidal pressure distribution should comprise a pressure of either 6H or 8H kPa (depending on the amount of deflection permissible, as discussed above) over the middle 50% that then tapers off to zero over the upper and lower 25% of the pressure distribution.

The above earth pressures assume horizontal backfill surfaces and where inclined backfill is proposed the earth pressures should be increased or the inclined backfill taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the wall.

The passive toe resistance for piled walls embedded at least 1.5m below bulk excavation level and through at least very low strength sandstone bedrock, may be estimated based on an allowable lateral resistance of 150kPa. The passive resistance should be ignored to at least 0.5m below bulk excavation level, and to the depth of any footing/ lift pit and service trench excavations (whichever is the deepest) due to the potential for fracturing of the sandstone during bulk excavation.

Following further geotechnical investigations, we recommend that more detailed retaining wall analysis be carried out using more advanced Finite Element (FEM) software, such as PLAXIS or similar. These programs also predict the movements behind the basement walls. The more frequently used retaining wall analysis program, WALLAP, is considered inappropriate for the design of the piled walls through rock, as it cannot predict movements behind the basement walls (only of the wall itself) and cannot model potential rock defects. Due to the numerous geotechnical engineering inputs required to drive and rationalise FEM programs, the analyses should only be carried out by engineers with a good understanding of retaining wall design, and soil and rock mechanics. We caution against software which treats the soil and bedrock profiles as ‘equivalent springs’ as these are not geotechnical parameters.

4.4.2 Temporary Anchors

Rock anchors bonded at least 3m into bedrock beyond a 45° line inclined up from bulk excavation level (including nearby footings and service trenches) and with a minimum free length of 4m may be tentatively designed for an allowable bond stress of:

- Very Low or Low strength sandstone = 150kPa
- Medium strength sandstone= 400kPa

All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors about 4 days following locking off to confirm that the anchors are holding their load. The testing may allow an upgrade of the above bond stress. We recommend that only experienced contractors be considered for the anchor installations. We have assumed that permanent lateral support of the basement walls will be provided by the proposed structure, after which time the rock anchors can be de-stressed.

For temporary anchors, permission must be sought from the neighbouring property owners, including Council and Transport for NSW prior to installation. We recommend that requests for permission commence early in the design process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping.

4.5 Hydrogeological Considerations

Based on the results of the preliminary investigations to date, we expect that groundwater will be encountered within the depth of the basement excavation. Groundwater seepage will likely occur along the soil-rock interface and through rock defects, particularly during and shortly after rainfall. Additional groundwater monitoring will be required so that further discussion can be provided on longer term groundwater levels and likely groundwater inflows.

During excavation, groundwater due to seepage and rainfall will need to be progressively pumped out as levels deepen. Seepage volumes into the excavation are expected to be controllable by conventional sump and pump discharge systems. Desilting and possibly chemical treatment may be needed prior to discharge and should be further assessed by the environmental/contamination consultant. Piped discharge from the drainage system into the stormwater system can only be completed once relevant approvals have been obtained. The excavation should be monitored as it progresses by the hydraulic engineer to confirm the drainage requirements. Furthermore, given the expected relatively low groundwater inflows, we consider that from a geotechnical perspective, a drained basement should be feasible for the site, however this is subject to approvals through WaterNSW, who may insist on the need for a tanked basement. In our opinion, due to the plan extent of the basement excavation we consider that there is a high probability that Water NSW will require a tanked basement for this site.

Construction of a basement that intersects the groundwater, which can include seepage, is considered to be an aquifer interference activity. Such activities are subject to the Water Management Act 2000 and NSW Aquifer Interference Policy and are regulated by the Department of Planning and Environment (DPE), WaterNSW and Natural Resource Access Regulator (NRAR). The DPE's policy on basements is that ongoing or frequent dewatering of basements over their life is inconsistent with the principals of sustainable development and, where such dewatering is required, basements should be tanked. Dewatering during construction is permitted but is regulated through licencing which must either be obtained from WaterNSW or NRAR.

The DPE's document, "Minimum Requirements for Building Site Groundwater Investigations and Reporting", dated October 2022 outlines the minimum scope of investigation required where a basement is proposed and may intersect the groundwater table. This scope is quite involved and broadly requires the following:

- Boreholes drilled to a minimum depth, which is defined by the proposed number of basements.
- The installation of a minimum of three groundwater wells installed throughout the site in a triangulated fashion.
- Permeability testing to define the coefficient of permeability of the various soil and bedrock layers.
- Groundwater monitoring for a minimum period of three months in the six months prior to the submission of documentation to the relevant authority.
- Groundwater modelling to predict the groundwater take, groundwater drawdown behind the retention system and potential impact on nearby structures and other groundwater users.
- Chemical analysis of the groundwater to determine its quality.

The above scope of works will need to be carried out to satisfy the WaterNSW requirements so that dewatering can be carried out. Where dewatering is required, potentially two approvals are required from WaterNSW. These are:

- A Water Access Licence (WAL).
- A Water Supply Works (WSW) approval.

A WAL is a licence that provides an allocation of a certain volume of water in the aquifer to a user. However, it does not provide the right to extract this water. To extract or pump water from an aquifer, such as is required during basement dewatering, a WSW approval is required. The WAL is required where extraction of water from the aquifer exceeds 3ML/annum, where a water year coincides with a financial year. Where extraction volumes are less than this value, a WAL is not required, but a WSW approval is still required to remove any water from a site.

4.6 Footings

Following bulk excavation, we expect that sandstone bedrock will be exposed at bulk excavation level. Based on BH1, very low to low strength sandstone will be encountered at bulk excavation level, however at the Pacific Highway end of the site (where excavation depths will be greater), better quality medium strength sandstone bedrock may be exposed. Footings may include pad/strip footings founded at shallow depth on the sandstone bedrock exposed at bulk excavation level, or possibly piled footings founded at greater depth on better quality medium or higher strength sandstone bedrock. As recommended in Section 4.1 above, additional geotechnical investigations will be required to provide more detailed advice on the subsurface conditions across the site, and particularly in relation for footing bearing pressures.

For preliminary design we recommend that pad/strip footings founded on sandstone bedrock of at least very low strength may be designed for a maximum allowable bearing pressure of 1000kPa. If medium strength bedrock is exposed at the Pacific Highway end of the site and/or if piled footings founded on the better quality medium strength sandstone bedrock at depth is preferred, then higher allowable end bearing pressures appear feasible. As a guide (but subject to further detailed proving), end bearing pressures in the order of 3500kPa and possibly higher appear feasible for pad/strip footings or piles founded on the medium strength sandstone bedrock. Where piled footings are adopted we consider that bored piles will be feasible. Some groundwater seepage may occur into the bored piers and therefore we recommend that piles be drilled, inspected, and poured with minimal delay. Where seepage does occur it should be pumped from the pier holes prior to pouring of concrete. Tremie pouring techniques may be required.

The above allowable bearing pressures are 'serviceability' parameters and are based on settlement of less than 1% of the pile diameter or footing width. More efficient footing design based on the use of ultimate bearing pressures may be feasible but would require further proving.

All pad/strip footings and bored piles should be founded with a nominal socket of at least 0.3m into the appropriate strength of rock. For the design of pile sockets in compression an allowable shaft adhesion of 100kPa may be adopted for sockets into at least very low strength sandstone. The shaft adhesion should be ignored within the 0.3m nominal socket. For the design of piles in uplift, shaft adhesions of half the shaft adhesions in compression may be used. The shaft adhesion values assume that adequate socket roughness and cleanliness is maintained.

All footing excavations and the drilling of bored piles should be inspected by a geotechnical engineer to confirm that a suitable founding stratum has been achieved.

4.7 Basement Slabs

For the proposed basement slab, we expect that bedrock will be uniformly exposed across the basement footprint and therefore a slab-on-ground should be feasible. Where basement on-grade floor slabs are poured directly over bedrock no particular subgrade preparation is required, although slabs should be provided with underfloor drainage and a granular debonding layer. The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as 'blue metal' gravel. The underfloor drainage should collect groundwater seepage and direct it to the stormwater system for disposal. Where required basement slabs may need to be designed to resist hydrostatic uplift pressures.

4.8 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Additional geotechnical investigation, including cored boreholes, to enable detailed design and to satisfy the specific requirements of Water NSW and Transport for NSW
- Additional groundwater monitoring.
- Obtaining details on the adjoining basement extent and level below No.46 Pacific Highway
- Further investigations and assessment of the existing retaining walls supporting the No. 38 Pacific Highway basement so that their impact on proposed shoring systems can be determined.
- FEM analysis of the basement retention system and excavation;
- Dilapidation survey reports on adjoining structures to the north-east and south-west and also as required on surrounding roads and footpaths, and possibly buried assets;
- Vibration monitoring;
- Inspection of perimeter pile wall drilling;
- Proof testing and lift-off testing of temporary rock anchors for the basement walls;
- Groundwater monitoring of seepage volumes;
- Internal footing inspections, as appropriate;

5 GENERAL COMMENTS

The recommendations presented in this report are preliminary only but include specific comments to be addressed during the detailed design and construction phases of the project. In the event that any of the advice presented in this report is not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions away from the completed borehole may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Report No.:**	37122L - A
Project:	Proposed Mixed Use Development	**Report Date:**	6/11/2024
Location:	28-38 Pacific Highway, St Leonards, NSW	**Page 1 of 1**	

AS 1289	**TEST METHOD**	**2.1.1**	**3.1.2**	**3.2.1**	**3.3.1**	**3.4.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
1	0.50 - 0.95	22.2	34	15	19	6.0
1	1.50 - 1.95	18.2	38	13	25	10.0
1	3.00 - 3.40	15.7	-	-	-	-
1	3.40 - 3.70	6.2	-	-	-	-

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 30/10/2024.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

06/11/2024
Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Edsgear Pty Ltd **Ref No:** 37122L

Project: Proposed Mixed Use Development **Report:** B

Location: 28 - 38 Pacific Highway, St Leonards **Report Date:** 30/10/24

Page 1 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	3.89 - 3.92	0.5	10	A
	4.12 - 4.15	0.3	6	A
	4.56 - 4.60	1	20	A
	6.24 - 6.29	0.2	4	A
	6.46 - 6.50	0.2	4	A
	6.73 - 6.76	0.1	2	A
	7.13 - 7.17	0.1	2	A
	7.49 - 7.52	0.3	6	A
	7.71 - 7.74	0.2	4	A
	8.16 - 8.19	0.4	8	A
	8.40 - 8.44	0.2	4	A
	9.13 - 9.17	0.1	2	A
	9.28 - 9.32	0.4	8	A
	9.75 - 9.79	0.3	6	A
	10.14 - 10.17	0.6	12	A
	10.50 - 10.54	0.4	8	A
	10.79 - 10.83	1	20	A
	11.12 - 11.14	1.5	30	A
	11.40 - 11.43	1.5	30	A
	11.86 - 11.89	0.8	16	A
	12.15 - 12.18	1.1	22	A
	12.76 - 12.79	1.1	22	A
	13.27 - 13.29	0.9	18	A
	13.69 - 13.71	0.9	18	A
	14.54 - 14.57	1	20	A

NOTE: SEE PAGE 2

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Edsgear Pty Ltd

Ref No: 37122L

Project: Proposed Mixed Use Development

Report: B

Location: 28 - 38 Pacific Highway, St Leonards

Report Date: 30/10/24

Page 2 of 2

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	15.06 - 15.08	0.6	12	A
	15.28 - 15.30	0.8	16	A
	15.65 - 15.69	1	20	A
	16.18 - 16.20	1.6	32	A
	16.85 - 16.89	0.9	18	A
	17.31 - 17.33	0.9	18	A
	17.75 - 17.78	0.6	12	A
	17.95 - 17.98	0.5	10	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 365177

Client Details

Client	JK Geotechnics
Attention	L Speechley
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**37122L, 28-38 Pacific Hwy St Leonards NSW**
Number of Samples	3 Soil
Date samples received	30/10/2024
Date completed instructions received	30/10/2024

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	07/11/2024
Date of Issue	07/11/2024

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Jenny He, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		365177-1	365177-2	365177-3
Your Reference	UNITS	1	1	1
Depth		0.2-0.4	0.5-0.95	2.5-2.7
Date Sampled		28/10/2024	28/10/2024	28/10/2024
Type of sample		Soil	Soil	Soil
Date prepared	-	30/10/2024	30/10/2024	30/10/2024
Date analysed	-	04/11/2024	04/11/2024	04/11/2024
pH 1:5 soil:water	pH Units	5.9	6.0	5.0
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	22	<10	37
Resistivity in soil*	ohm m	340	400	350

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			30/10/2024	[NT]	[NT]	[NT]	[NT]	30/10/2024	[NT]
Date analysed	-			04/11/2024	[NT]	[NT]	[NT]	[NT]	04/11/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	107	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	109	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

BOREHOLE LOG

Borehole No.
1
1 / 4

Client: EDSGEAR PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Job No.: 37122L **Method:** SPIRAL AUGER **R.L. Surface:** ~75.3 m
Date: 28/10/24 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** S.W./L.S.

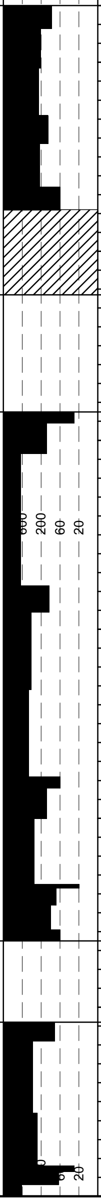
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										CONCRETE: 90mm.t				5mm DIA. REINFORCEMENT, 30mm BOTTOM COVER
						75			-	FILL: Silty clay, medium plasticity, brown, trace of fine grained igneous gravel, trace of fine to medium grained sand.	w>PL			APPEARS POORLY COMPACTED
					N = 3 1,2,1		1			FILL: Silty clay, medium plasticity, light brown and grey, trace of fine grained ironstone gravel and fine to medium grained sand.	w~PL		100 120 140	
					N = 13 4,4,9	74			CI	Silty CLAY: medium plasticity, light grey and red brown, trace of fine to medium grained sand.	w>PL	St	150 160 180	RESIDUAL
						73	2				w~PL	Hd	410 460 490	
					N > 28 6,14,14/ 100mm REFUSAL	72	3		-	Extremely Weathered sandstone: silty CLAY, medium plasticity, light grey and red brown.	XW	Hd	>600 >600 >600	HAWKESBURY SANDSTONE
										SANDSTONE: fine to medium grained, purple brown.	MW	M		MODERATE TO HIGH 'TC' BIT RESISTANCE
						71 70 69	4 5 6			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 16.36m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.36m TO 16.36m. CASING 0.06m TO 4.36m. 2mm SAND FILTER PACK 4.0m TO 16.36m. BENTONITE SEAL 2.0m TO 4.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

CORED BOREHOLE LOG

Borehole No.
1
2 / 4

Client: EDSGEAR PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Job No.: 37122L	**Core Size:** NMLC	**R.L. Surface:** ~75.3 m
Date: 28/10/24	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** S.W./L.S.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific General	Formation
		72			START CORING AT 3.70m					
ON 5/11/24		71	4		SANDSTONE: fine to medium grained, light grey and purple brown, bedded at 0-5°.	HW	L	0.50 0.30	(3.82m) Be, 5°, P, R, Cn (4.03m) Be, 0°, P, R, Cn (4.28m) Be, 5°, P, R, Cn (4.43m) J, 80°, Ir, R, Cn	Hawkesbury Sandstone
					as above, but bedded at 10-20°.			1.0	(4.66m) Be, 20°, P, R, Cn (4.72m) Be, 20°, P, R, Fe Sn	
			5		Extremely Weathered sandstone: silty CLAY, medium plasticity, light grey and red brown, with very low strength, dark grey siltstone laminae.	XW	Hd			
		70			NO CORE 0.62m					
90% RETURN		69 68	6 7		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-10°.	HW	VL - L	0.20 0.20 0.10 0.10	(5.89m) Be, 5°, P, R, Cb Ct (5.91m) XWS, 0°, 90 mm.t (6.07m) J, 70°, P, R, Clay Ct (6.77m) XWS, 0°, 14 mm.t (6.91m) XWS, 0°, 40 mm.t (7.32m) XWS, 0°, 150 mm.t	Hawkesbury Sandstone
		67	8		as above, but bedded at 0-20°.			0.30 0.20 0.40 0.20	(7.78m) Be, 20°, P, R, Cb Ct (7.84m) Be, 10°, P, R, Cn (8.00m) XWS, 0°, 90 mm.t (8.35m) Be, 0°, P, R, Fe Sn (8.37m) Be, 0°, P, R, Fe Sn (8.46m) CS, 0°, 100 mm.t (8.59m) CS, 0°, 60 mm.t	
			9		NO CORE 0.43m				~~(9.08m) CS, 0°, 10 mm.t~~	
		66			SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°.	HW SW	VL L - M	0.10 0.40 0.30	(9.18m) XWS, 0°, 70 mm.t (9.56m) XWS, 0°, 65 mm.t (9.84m) Be, 5°, P, R, Fe Sn (9.87m) CS, 0°, 45 mm.t	

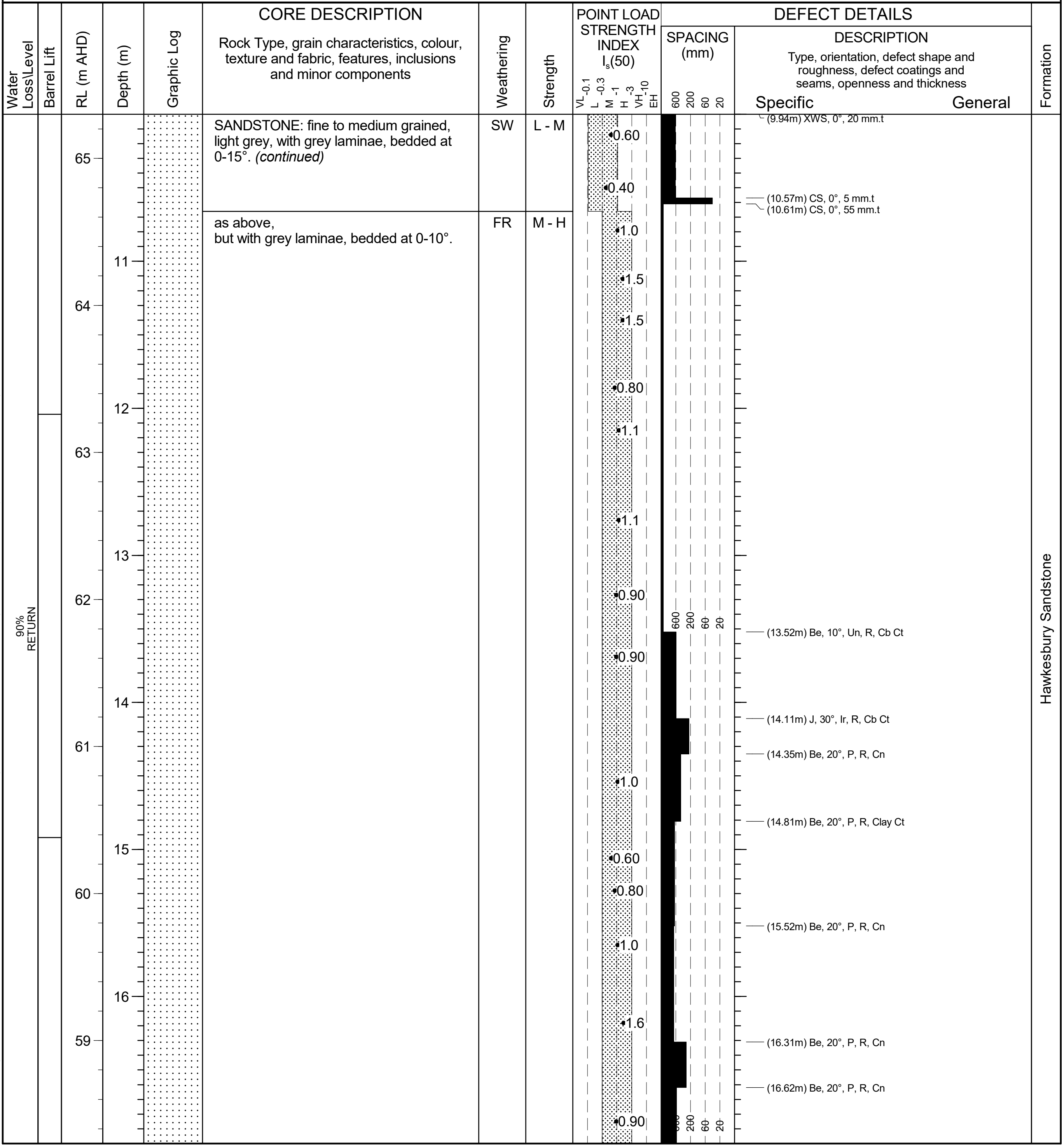
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
1
3 / 4

Client: EDSGEAR PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Job No.: 37122L
Core Size: NMLC
R.L. Surface: ~75.3 m
Date: 28/10/24
Inclination: VERTICAL
Datum: AHD
Plant Type: JK308
Bearing: N/A
Logged/Checked By: S.W./L.S.



CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength
SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-15°. *(continued)*	SW	L - M
as above, but with grey laminae, bedded at 0-10°.	FR	M - H

DEFECT DETAILS

DESCRIPTION
Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness
Specific General

(9.94m) XWS, 0°, 20 mm.t
(10.57m) CS, 0°, 5 mm.t
(10.61m) CS, 0°, 55 mm.t
(13.52m) Be, 10°, Un, R, Cb Ct
(14.11m) J, 30°, Ir, R, Cb Ct
(14.35m) Be, 20°, P, R, Cn
(14.81m) Be, 20°, P, R, Clay Ct
(15.52m) Be, 20°, P, R, Cn
(16.31m) Be, 20°, P, R, Cn
(16.62m) Be, 20°, P, R, Cn

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
4 / 4

Client: EDSGEAR PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Job No.: 37122L
Date: 28/10/24
Plant Type: JK308
Core Size: NMLC
Inclination: VERTICAL
Bearing: N/A
R.L. Surface: ~75.3 m
Datum: AHD
Logged/Checked By: S.W./L.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
90% RETURN		58	18		SANDSTONE: fine to medium grained, light grey, with grey laminae, bedded at 0-10°.	FR	M - H	0.90 0.60 0.50		(17.20m) Be, 0°, P, R, Clay Ct (17.39m) XWS, 0°, 65 mm.t (17.71m) J, 30°, Ir, R, Cn	Hawkesbury Sandstone
		57 56 55 54 53 52	19 20 21 22 23		END OF BOREHOLE AT 18.06 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 37122L

Borehole No: BH1

Depth: 3.70m to 12.00m

Job No: 37122L
Borehole No: BH1
Depth: 12.00m to 18.06m

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title:

SITE LOCATION PLAN

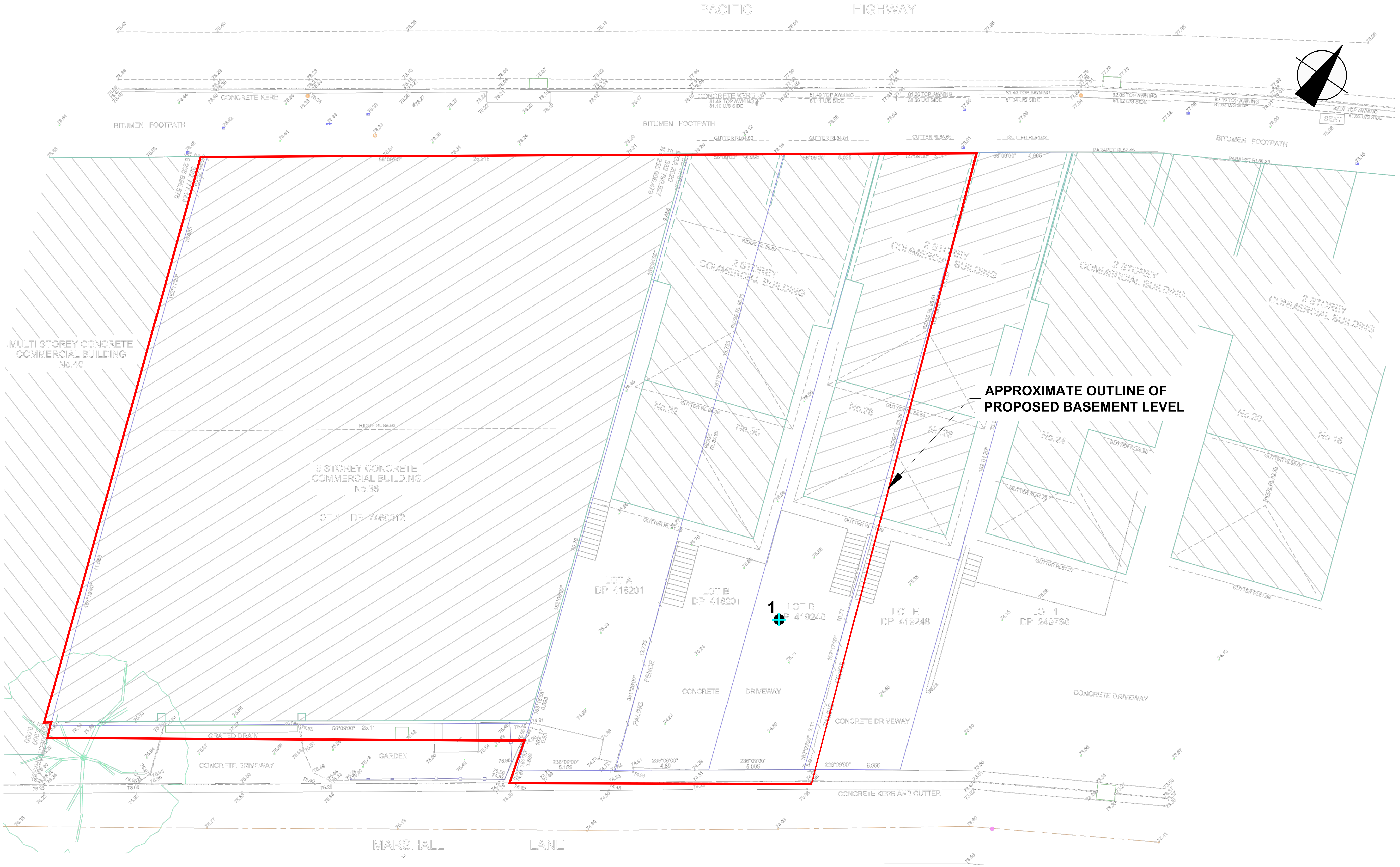
Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Report No: 37122L

Figure No: 1

JKGeotechnics

PLOT DATE: 30/10/2024 5:55:17 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\37000'S\37122L ST LEONARDS\CAD\37122L.DWG



LEGEND

⊕ BOREHOLE AND WELL

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **BOREHOLE LOCATION PLAN**

Location: 28-38 PACIFIC HIGHWAY, ST LEONARDS, NSW

Report No: 37122L

Figure No: 2

JKGeotechnics

PLOT DATE: 30/10/2024 5:54:51 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\37000'S\37122L ST LEONARDS\CAD\37122L.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

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

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- ▩ FILL
- ≋ TOPSOIL
- ▨ CLAY (CL, CI, CH)
- ▥ SILT (ML, MH)
- ░ SAND (SP, SW)
- ⁘ GRAVEL (GP, GW)
- ▨ SANDY CLAY (CL, CI, CH)
- ▨ SILTY CLAY (CL, CI, CH)
- ▨ CLAYEY SAND (SC)
- ░ SILTY SAND (SM)
- ▨ GRAVELLY CLAY (CL, CI, CH)
- ▨ CLAYEY GRAVEL (GC)
- ▥ SANDY SILT (ML, MH)
- ⁂ PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- ⁘ CONGLOMERATE
- ░ SANDSTONE
- ▤ SHALE/MUDSTONE
- ▥ SILTSTONE
- ▤ CLAYSTONE
- ■ COAL
- ▥ LAMINITE
- ▦ LIMESTONE
- ≋ PHYLLITE, SCHIST
- ☰ TUFF
- ⁖ GRANITE, GABBRO
- ✢ DOLERITE, DIORITE
- ⌵ BASALT, ANDESITE
- ≈ QUARTZITE

OTHER MATERIALS

- ▦ BRICKS OR PAVERS
- △ CONCRETE
- ■ ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

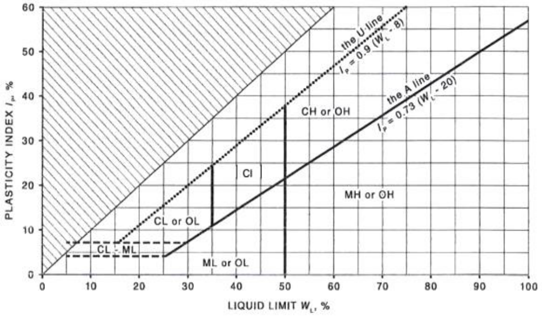
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition		
Groundwater Record	─▼─	Standing water level. Time delay following completion of drilling/excavation may be shown.		
	─C─	Extent of borehole/test pit collapse shortly after drilling/excavation.		
	▶─	Groundwater seepage into borehole or test pit noted during drilling or excavation.		
Samples	ES	Sample taken over depth indicated, for environmental analysis.		
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.		
	DB	Bulk disturbed sample taken over depth indicated.		
	DS	Small disturbed bag sample taken over depth indicated.		
	ASB	Soil sample taken over depth indicated, for asbestos analysis.		
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.		
	SAL	Soil sample taken over depth indicated, for salinity analysis.		
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.		
	VNS = 25	Vane shear reading in kPa of undrained shear strength.		
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).		
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.		
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.		
	w < PL	Moisture content estimated to be less than plastic limit.		
	w ≈ LL	Moisture content estimated to be near liquid limit.		
	w > LL	Moisture content estimated to be wet of liquid limit.		
(Coarse Grained Soils)	D	DRY – runs freely through fingers.		
	M	MOIST – does not run freely but no free water visible on soil surface.		
	W	WET – free water visible on soil surface.		
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.		
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.		
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.		
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.		
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.		
	Hd	HARD – unconfined compressive strength > 400kPa.		
	Fr	FRIABLE – strength not attainable, soil crumbles.		
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.		
Density Index/ Relative Density (Cohesionless Soils)			**Density Index (I_D) Range (%)**	**SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.		

Log Column	Symbol	Definition
Remarks	‘V’ bit	Hardened steel ‘V’ shaped bit.
	‘TC’ bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description ‘slopewash’ is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'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'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	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 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

Date: 17 November 2025
Ref: 37122L Let1

Water NSW
169 Macquarie Street
PARRAMATTA NSW 2124

Attention: Simone Tonkin
Email: simone.tonkin@waternsw.com.au

GEOTECHNICAL COMMENTS
PROPOSED DEVELOPMENT
LOT 1 (DP 746012) 34-42 PACIFI CHIGHWAY ST LEONARDS, NSW

(WaterNSW Reference IDAS1163820)

We have been engaged as the geotechnical consulting engineers for the project at Lot 1 DP746012 – 34 to 42 Pacific Highway St Leonards. The client has sent us your letter reference IDAS1163820, dated 30 September 2025, requesting further information.

Our understanding is that it is proposed to construct a fully tanked basement for the development. For that scenario you have requested that the client provides further information on the volume of water to be extracted, the duration of water take for dewatering and the method of measuring the water take.

JK Geotechnics carried out a geotechnical investigation in October last year. Due to site access constraints (existing buildings cover most of the site), the geotechnical investigation included the drilling of only one deep borehole in the only accessible location within the south-eastern corner of the site. The borehole also included installation of a groundwater monitoring well. That borehole extended to a depth of about 18m which is in the order of 11.5m below the proposed lowest basement level. Currently groundwater levels have only been recorded once, about 1 week after the drilling (i.e. on 5 November 2024). In order to be able to provide the information requested in your RFI (in particular the volume of water to be extracted), we acknowledge that some additional work will need to be carried out to check the groundwater levels, carry out some pump out testing for permeability in the monitoring well, and carry out seepage analyses/modelling.

We are aware of The Department of Planning and Environment's Document titled '*Minimum Requirements for Building Site Groundwater Investigation and Reporting*' dated October 2022, which requires a minimum of three groundwater monitoring wells, and groundwater to be monitored for a period of three months. However, as discussed above, access onto the site for suitable drilling equipment, to be able to drill to the required depths, is extremely challenging at present with the current buildings, and in fact it may not be feasible without some demolition works etc. On that basis, we were hoping to obtain your feedback and approval on whether it would be possible to approach the assessment of volume of water to be extracted on the basis of the following;

- Install a data logger into the existing groundwater monitoring well and monitor the groundwater level for a period of at least one month.
- Carry out pump out testing within the groundwater monitoring well to assess the permeability of the sandstone bedrock. We note that the limited groundwater monitoring to date indicated that the groundwater level was within the sandstone bedrock and about 2m above the proposed lowest basement level.
- Complete seepage analysis based on the information obtained and provide a preliminary assessment of the volume of water take.

From a geotechnical and hydrogeological perspective, we are still of the opinion that at least two more boreholes, and a longer period of monitoring will be required at some stage, so that a more considered assessment of the geological and hydrogeological conditions across the site can be made. However, until full demolition, or at least substantial partial demolition is carried out this is going to be extremely difficult and costly. Therefore considering that the proposal is for a tanked basement, is it feasible to have a conditional general terms of approval, with final approval provided following the additional investigations, groundwater monitoring and analysis, when access is possible after demolition.

We would appreciate your consideration of the above and look forward to your reply. I would be pleased to discuss this further with you if anything above is unclear or if you require further information from a geotechnical perspective. The project architect and planners can also be contacted if you need to further assistance to evaluate this request.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

Linton Speechley
Principal Geotechnical Engineer