REPORT TO

DAVID AND CHRISTINE LAROSE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

24 OGILVY ROAD, CLONTARF, NSW

Date: 8 April 2025

Ref: 36816SrptRev2

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Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 24 Ogilvy Road, Clontarf, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr David LaRose by signed 'Acceptance of Proposal' form dated 23 June 2024 and was carried out in accordance with our fee proposal, Ref: P60554B, dated 7 May 2024.

Based on the provided architectural drawings prepared by Archisoul Architects (Project No. 2266, Drawing Nos.DA01 to DA34, Revision 1, dated 2 April 2025), we understand that following demolition of all site structures that two-storey structure will be constructed. A garage will be situated below the southern end of the house, and will require excavation into the hillside by up to about 2.7m depth on the northern side, reducing to nil on the southern side. Locally deeper excavation will be required for the lift pit, service trenches, footings, etc. The garage excavation will be set-back by about 0.9m and 2.3m from the western and eastern boundaries, respectively, and by several metres from the northern boundary. The ground and first floors will be constructed above and beyond the garage footprint and will be constructed at or above existing ground levels. A driveway will be lead from Ogilvy Road to the garage. An in-ground swimming pool will be situated within the north-eastern corner of the ground floor, and will require excavation to depths of about 1.8m. The pool will be set-back from the eastern boundary by about 1.8m.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on excavation, shoring, retention, hydrogeology, footings, slab on grade and engineered fill.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 1 July 2024 and included the drilling of two boreholes using portable hand operated drilling equipment. The boreholes were initially drilled using a hand auger and then continued using portable Melvelle hydraulically powered core drilling equipment. The boreholes extended to total depths of 6.96m and 7.34m at BH1 and BH2, respectively, below existing surface levels.

A Dynamic Cone Penetration (DCP) test was also carried out adjacent to each borehole and at five additional locations. The DCP tests were used to assess the degree of compaction of the fill and attempt to probe to the top of the underlying sandstone bedrock. The DCP tests extended to refusal depths ranging from 0.28m to 2.9m below existing surface levels. It is noted however that DCP refusal may also occur on inclusions within the fill, harder iron indurated bands in residual soils etc. We note that the refusal of the DCP tests (where cored boreholes were not carried out) may not indicate the top of the bedrock, rather a sandstone boulder or similar. Additional cored boreholes would be required to confirm the sandstone bedrock at these locations.

The test locations are shown on the attached Figure 2 and were set out by taped measurements from existing surface features and structures shown on the survey plan by CMS Surveyors Pty Ltd (Ref: 23325detail, Issue 1 and dated 28 March 2024). The surface reduced levels indicated on the attached borehole logs and DCP test results sheets were interpolated from spot levels on the above referenced survey plan and are therefore only approximate. The survey datum is Australian Height datum (AHD).

Where the Melvelle coring commenced, the recovered core was placed in steel boxes and returned to our laboratories where it was photographed and Point Load Strength Index (I_{s50}) testing carried out. Using established correlations, the unconfined compressive strength (UCS) of the rock was estimated from the I_{s50} results. The point load strength index tests are attached as Table A and are also plotted on the borehole logs. Copies of the core photographs are provided with the borehole logs.

Selected soil samples were returned to Envirolab Services Pty Ltd, a NATA accredited laboratory, for soil aggression testing and the results are provided in the attached Certificate of Analysis No.355720.

Groundwater observations were made in the boreholes during and on completion of hand auger drilling. During core drilling, water is used to flush rock cuttings from the hole and therefore groundwater measurements on completion of coring can be artificially high. No longer term groundwater monitoring was carried out.

Our geotechnical engineer was present on a full-time basis during the fieldwork to set out the borehole locations, nominate the testing and sampling and to prepare the attached borehole logs and DCP test results sheets. Further details of the methods and procedures used during the investigation are presented in the attached Report Explanation Notes.

A contamination screen of the site soils was not carried out as this was beyond the agreed scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located mid-slope on a south facing hillside, which slopes and steps down towards North Harbour. Localised gullies have been 'cut' into the regional hillside and have moderately sloping flanks, with the site situated near the base of such a gully. The site is bound to the south by Ogilvy Road, and by residential properties on its remaining sides. An elevation relief of about 7.5m exists between the rear (northern) and front (southern) boundaries, respectively.

The site contains a two-storey split level brick and clad house situated within the northern portion of the site that appears to be in good external condition, based on a cursory inspection of the exterior. A small single car brick garage and carport are located adjacent to the Ogilvy Road frontage within the south-eastern corner of the site. A 0.9m high timber retaining wall is located immediately to the north of the garage, supports the front garden, and appears to be in good condition.

A brick paved pathway leads up from the carport to the house through the centre of the front garden. The front garden is densely vegetated either side of the path and slopes down to the south at about 10°. A large timber balcony stretches from the second floor of the house to the south, and forms an undercover area at ground level. A raised brick paved patio extends from the ground floor of the house and is supported by a 0.7m high timber retaining wall. The brick paved pathway extends along the eastern side of the house towards the rear yard, which comprises a brick paved courtyard. A large sandstone boulder is located in the undercover area, near the north-western corner of the site. The boulder was assessed to be of medium strength, based on a tactile assessment using a geopick. A perimeter garden bed extends along the northern boundary and is supported by a 0.4m high timber retaining wall. A small tin shed is located in the north eastern corner of the rear yard. A sandstone outcrop (possible boulder), was observed adjacent to the western side of the house, and was assessed to be of low-medium strength. The outcrop measured 0.3m high.

The neighbouring property to the west, No.26 Ogilvy Road, contains a one to three-storey split level brick and clad house, which is centrally located and appears to be in good external condition based on a cursory inspection from within the subject site. The house is set-back from the common boundary by about 3m. Ground levels along the common boundary are generally similar to that of the subject site, besides the central portion which is 0.5m to 1m lower than the subject site and retained by a timber retaining wall. At the front of the property, adjacent to the concrete driveway is a sandstone outcrop approximately 0.7m high that appears to comprise low to medium strength sandstone. Seepage was noted over the face of the sandstone.

The neighbouring property to the east, No.22 Ogilvy Road, contains a three-storey rendered brick and timber house, located near the Ogilvy Road frontage. The corner of the house abuts the common boundary at its north-western corner, increasing in set-back distance to the south. A stone garage abuts the common boundary adjacent to the Ogilvy Road frontage. Ground levels were generally lower than the subject site and retained by a brick retaining wall along the northern half, and a timber retaining wall along the central portion. Ground levels were between 0.6m to 1.2m lower than the subject site.

The neighbouring property to the north, No.41 Cutler Road, contains a two-storey house set-back from the common boundary by several metres. Adjacent to the common boundary was a backyard which contains an in-ground swimming pool which is set-back from the common boundary by approximately 4m. Ground levels along the common boundary generally appear to be similar to those of the subject site.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates that the site is underlain by the Hawksbury Sandstone of the 'Wianamatta Group', which comprises *'medium to coarse-grained quartz sandstone, very minor shale and laminite lenses'.*

The site is located on a sloping hillside with some medium to large detached sandstone floaters observed at the surface, along with some sandstone outcrops near the lower elevations of the site area. Based on the results of the investigation, sandstone bedrock was encountered at shallow to moderate depths. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the borehole logs for specific details at each location.

Fill

Silty sand and silty sandy clay fill was encountered in both boreholes and extended to depths of 1.72m (BH1) and 2.2m (BH2), though the fill may extend to a depths of 2.06m in BH1 and 2.2m in BH2 due to the 'no-core' zones within the upper portion of the cored portions of the boreholes. Based on the DCP test results, the fill was assessed to be poorly compacted. Premature refusal of the hand auger occurred within the fill profile at BH2 at a depth of 1.2m and required the use of casing advancer techniques and as such, observations of the soil profile below this depth were limited and inferred to comprise fill.

Sandstone Bedrock

Underlying the fill at both boreholes was sandstone bedrock that was encountered at depths of 2.06m (≈RL29.0m) and 2.2m (≈RL30.2m) at BH1 and BH2, respectively. Within the cored portions of the boreholes, the sandstone bedrock was typically of medium strength and slightly weathered. A thin layer of low strength sandstone was encountered above the better-quality sandstone in BH2. Defects within the recovered core comprised sub-horizontal weathered seams up to 10mm thick, sub-horizontal bedding partings, joints inclined between 60° and 80°, and healed joints inclined up to 80°. The upper 'no core' zones likely represent the overlying soil profile above the sandstone bedrock that has been washed away during the drilling process. Conversely, the thinner 'no core' zone encountered in BH1 within the sandstone bedrock profile is likely to represent a weathered seam or weaker rock which has been washed away during the drilling process.

Groundwater

Some seepage was noted during drilling in BH1 at a depth of 0.9m, whilst BH2 was dry during and on completion of hand auger drilling.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s(50)}$) tests generally ranged from 0.5MPa to 0.6MPa, although locally higher and lower results were measured. These are also plotted on the attached borehole logs. Estimated unconfined compressive strength (UCS), based on the relationship of UCS = 20 x $I_{s(50)}$), ranged generally from 10MPa to 14MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below. The Envirolab Certificate of Analysis No.355720 is attached and provides further specific details for these tests.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
1	0.9 – 1.1	FILL: Silty Sandy Clay	6.8	<10	20	28,000

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
2	0.9 – 1.1	FILL: Silty Sand	6.2	<10	29	20,000
2	2.8 – 3.0	Sandstone Bedrock	5.0	32	<10	39,000

4 COMMENTS AND RECOMMENDATIONS

4.1 Primary Geotechnical Issues

We consider that the principal geotechnical issues for the proposed development to be how to appropriately support the excavation during construction without adversely impacting neighbouring structures, and obtaining approvals from Sydney Water.

Considering the set-back distances from the proposed garage excavation to the site boundaries, and the depth to bedrock, a shoring system will need to be installed prior to excavation commencing. The sandstone generally appears to be of good-quality on first contact and thus suitable to be cut vertically. However, given the depth to rock (i.e about 0.5m above the BEL), consideration could be given to the adoption of a full-height retention system in order to minimise the need for temporary propping and/or anchoring and to de-risk the project. Site access for suitable machinery to install shoring piles may be difficult, though we expect that a piling rig will be required.

4.2 Adjoining Properties and Dilapidation Surveys

Dilapidation surveys of adjoining buildings and structures that fall in the area of influence of the excavation are a necessary part of the process of claim protection, i.e. avoiding spurious claim of damage where, in fact, the damage existed prior to excavation or demolition commencing.

Consequently, prior to demolition and excavation commencing, we recommend that detailed dilapidation reports be compiled on buildings and structures that fall within the zone of influence of the excavation. The zone of influence regarding excavation for dilapidation surveys may be defined by a distance back from the excavation perimeter of twice the depth of the excavation. At a minimum this should include Nos. 22 and 26 Ogilvy Road, the Sydney Water asset which traverses the site and the council stormwater pipe.

The dilapidation surveys should comprise detailed inspections of the adjoining properties and structures (including retaining walls), both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. Pipes should be surveyed by camera. The respective owners should be asked to confirm in writing that these reports represent a fair record of actual conditions. These reports should be carefully reviewed prior to excavation commencing to ensure that appropriate equipment is used. In particular, the size/energy of the rock impact breakers should be considered.

4.3 Excavation Conditions

Prior to any excavation commencing we recommend that reference be made to the latest version of the WorkCover Authority of NSW's Code of Practice – Excavation Work.

We understand excavation to a maximum depth of about 2.7m will be required for the garage excavavation, with localised deeper excavations for lift pits, footings, service trenches, etc. Prior to any excavation, a retention system will be required, as discussed in Section 4.6 below. The proposed pool will require excavation to about 1.8m depth.

Based on the boreholes, excavation for the proposed garage will likely extend through fill to depths of about 2m, and then through slightly weathered, medium strength sandstone bedrock. The depth to sandstone bedrock is expected to step/slope down to the south. Sandstone boulders may be encountered within the overlying soils.

Excavation through the fill will be achievable using buckets of conventional earthmoving equipment (such as tracked excavators). Where sandstone floaters/boulders are encountered, these will need to be carefully removed with the buckets of excavators or alternatively broken up using rock excavation techniques (such as rock saws or hydraulic rock hammers). A waste classification will need to be assigned to any excavated material that is to be disposed of offsite. This needs to be completed prior to offsite disposal. We can provide the appropriate testing if required.

Where sandstone bedrock of low strength or greater is encountered, "hard rock" excavation conditions will be encountered and will require the adoption of either percussive or non-percussive rock excavation techniques. Percussive techniques comprise the use of rock hammers while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting, etc.

Some seepage was encountered in BH1 during hand auger drilling. Notwithstanding this, we expect that some groundwater seepage may occur at the fill/natural soil (if any) and soil/bedrock interface and through defects present within the rock mass during or immediately following periods of wet weather. We consider that any seepage will be able to be controlled by gravity discharge and/or sump and pump techniques. We recommend that a hydraulic engineer inspect the site during construction and/or once the bulk excavation has been carried out to provide comments regarding drainage requirements.

4.4 Excavation Vibration

Considerable caution must be taken during rock excavation (including breaking up of sandstone floaters/boulders) on this site as there will likely be direct transmission of ground vibrations to the neighbouring buildings and structures and nearby buried services. The neighbouring structures and Sydney Water Asset are likely to be founded on the sandstone bedrock, though this is unconfirmed at present.

Excavation procedures and the dilapidation reports should be carefully reviewed by the geotechnical and structural engineers prior to the commencement of demolition and excavation so that appropriate equipment is used.

Excavation through rock with hydraulic rock hammers should commence away from likely critical areas (i.e. commence within the central portion of the site) employing a moderately sized excavator fitted with a relatively low energy rock hammer. To reduce the transmission of vibrations, consideration may also need to be given to vertical saw cutting around the perimeter of the excavation, with the base of the saw cut slot maintained at a lower level than the adjoining rock excavation at all times. While this will help with the attenuation of transmitted vibrations, the magnitude of such attenuation is typically minor.

Continuous quantitative vibration monitoring should be carried out during all rock excavation. Vibration monitors should be set up on the adjoining dwellings at No.22 and No.26 Ogilvy Road, and may be required for the Sydney Water asset also. Vibration monitors should be fitted with flashing warning lights, sirens, text messages etc to provide real-time feedback to warn when vibrations exceed pre-set limits. Subject to review of the dilapidation reports, vibrations, measured as Peak Vibration Velocity (v_{imax}), should be limited to no higher than 5mm/sec on the neighbouring buildings. Vibration Emission Design Goals are attached to the rear of this report.

If during excavation with rock hammers it is found that transmitted vibrations are excessive, then it would be necessary to use a smaller rock hammer or alternative non-percussive excavation techniques. The use of a rotary grinder or grid sawing in conjunction with ripping and hammering present alternative lower vibration excavation techniques.

When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations where rock hammers are used:

- Maintain the rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate hammer in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil

Alternatively, non-percussive excavation methods may be adopted. These methods may consist of the use of rock saws, rotary grinders, rock splitting or ripping tynes. Where ripping tynes are used we consider that they will need to be attached to medium sized or bigger excavators to effectively rip sandstone bedrock of better than low strength, but even so it is not likely that ripping will be effective without saw-cutting where higher strength sandstone bedrock is encountered. Where ripping tynes attached to excavators are used, care must be taken to ensure that the tyne is not hammered into the rock in an attempt to break or dislodge the bedrock as this action will result in the creation of possibly damaging transmitted vibrations. Where non-percussive excavation techniques are adopted we consider that vibration monitoring will not be necessary.

Only excavation contractors with the appropriate experience and insurances with a competent and experienced supervisor who is aware of vibration damage risks, possible rock face instability issues etc should be considered for this project. The contractor should be provided with a copy of this report.

4.5 Hydrogeology

Groundwater seepage was encountered in BH1 slightly above the bedrock surface. Consequently, while it is expected that some groundwater seepage will be encountered both during and on completion of construction, due to the topographical setting of the site in proximity to North Harbour, we do not anticipate that the measured water levels represent a groundwater table, rather that they reflect transient flows that may occur during and following periods of rainfall. While some inflow of water into the excavation during construction should be anticipated, it is expected that all water inflows may be controlled by collecting and then discharging by gravity to the stormwater system.

Inspection and monitoring of groundwater seepage during excavation is recommended, so that any unexpected conditions, which may be revealed can be incorporated into the drainage design. Since dewatering will not be required, there will be no groundwater 'drawdown' outside of the excavation and assuming free drainage of the rock cuts (if required) and provision of adequate drainage behind any retaining walls there will be no damming effects from the below ground level floors.

In the long term, drainage will need to be provided behind all retaining walls for the garage to intercept ephemeral seepage and dispose of it directly to the stormwater. The completed excavation should be inspected by the hydraulic and geotechnical engineers to assess if the designed drainage system is adequate for actual seepage flows.

4.6 Retention Methodology

4.6.1 Garage Excavation

Excavation for the proposed garage is anticipated to extend to maximum depths of about 2.7m at the northern end, reducing to nil at the southern end. The excavation will be set back from the western and eastern boundaries by about 0.9m and 2.3m, respectively, and by several metres from the northern boundary. Sandstone bedrock was encountered at depths ranging between about 2.1m (BH1) and 2.2m (BH2), with the overlying soils comprising sandy fill and sandy clay fill.

Based on the depth to bedrock, there generally appears to be sufficient space to form temporary batters for the garage excavation on the northern side, and marginally feasible on the eastern side to facilitate the proposed excavation. However, considering the limited space available on the western side, a retention system will need to be installed prior to the garage excavation. To minimise the impact of the excavation on the remainder of the site, consideration could be given to installing the retention system around the entire garage footprint, so that construction activities outside of the excavation can continue. This will also minimise the amount of excavation and filling required.

Temporary batters through the fill and any residual soils encountered, should be formed no steeper than 1 Vertical to 1.5 Horizontal (1V:1.5H). Where sandstone bedrock of less than low strength is present, temporary batters may be formed no steeper than 1V:1H. Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crests of batters. Batter slopes should be protected from erosion. As a minimum, surface drainage should not be allowed to flow over the crest of temporary batters, and should be directed and discharged in a manner which avoids concentrated flows and erosion.

The toe of the batter should be set back from the crest of any vertical rock cut by at least 0.5m and sand bags installed to minimise loose or slumping material impacting the excavation below, noting the seepage that was observed immediately above the rock surface. In the long-term, cantilever, propped or dincel type block retaining walls can be constructed in front of the temporary cut batter slopes, and then backfilled on completion of excavation.

Where the above temporary batter slopes cannot be accommodated within the site geometry, or where they are not desired, an engineer designed retention system would be required and should be installed prior to bulk excavation commencing. If the retention system is only used along the western side of the excavation, then the shoring wall must extend a sufficient length to the north past the footprint of the proposed excavation to ensure that the temporary batters do not encroach past the site boundaries.

Given the sandy nature of the fill, and nearby structures, the use of soldier pile retaining walls with shotcrete infill panels may be difficult to construct as the soils may not stand sufficiently to allow placement of shotcrete. As such, a contiguous piled wall is recommended and will help limit deflections and the potential impact on nearby structures. As excavation progresses, the gaps between the contiguous piles must be dry packed to prevent sand runs and the loss of material from behind the wall. We consider that bored piles will be feasible through the upper soils, however due to the granular nature of the upper soils, temporary casing will need to be provided to maintain support to the upper soils during drilling. We recommend that the temporary casing be installed once the pile depth reaches the top of the bedrock and then the remainder of the pile drilled from within the casing and through into the bedrock. This will assist in reducing the risk of pile decompression (which can occur during drilling of deeper rock sockets and removal of excess spoil from the hole).

At least the initial stages of shoring pile drilling should be inspected by a geotechnical engineer to provide greater confidence that the piles are suitably socketed into the underlying sandstone bedrock and to check initial design assumptions. Inspection of piles will require the geotechnical engineer to be on site during the drilling process so that they can inspect the material being drilled and compare this information to the nearby borehole logs.

Considering the depth to bedrock, there appears to be two possible options for the design of the shoring wall which are:

1. Extend piles to the surface of the sandstone bedrock (above BEL), temporarily prop or anchor the shoring wall and then vertically excavate the sandstone below the pile toes, or

2. Construct a full-depth shoring wall that is socketed into the sandstone below the BEL, and would be our recommended approach.

Option 1

Based on the depth to good quality sandstone, the pile walls may be terminated above BEL, and the sandstone then cut vertically, provided lateral support is provided by the installation of at least two rows of temporary anchors or internal props. The suitability of anchors will depend on the level of the adjoining properties and where anchors cannot be accommodated, or are not preferred, internal props would be required. Due to the footprint of the proposed excavation and its proximity to the site boundaries, where adopted, anchors will extend across the site boundaries. Consequently, permission will need to be obtained from the owners of the adjoining properties prior to installation of anchors. Anchors will need to have their bond formed within sandstone bedrock below a line drawn up at 1 Vertical in 1 Horizontal from the base of the excavation. A variation on this scheme could be to install vertical dowel bars through the piles that extend into the bedrock below bulk excavation level. The vertical bars would thus secure the base of the piles but the upper row of props or anchors would still be required.

Competent sandstone bedrock (low or higher strength) may be cut vertically in the short term, subject to geotechnical inspection. If the rock is proven to be of sufficient quality to be self-supporting in the long term, it may be possible to provide an accessible void to maintain drainage around the ground floor level to keep a ‘dry’ wall around that level. However, we recommend that a retaining wall is constructed against all cut faces, or protected using shotcrete and dowels.

Initially we recommend that excavation extend no deeper than 1.0m below the toe of the shoring piles, **and** not closer than 1.5m from the toe of the shoring piles, until a geotechnical engineer can inspect and assess the quality of the bedrock and the likely founding material of the shoring piles. Further excavation can then be approved by the geotechnical engineers, who may require a specific excavation methodology to be undertaken. As a minimum, geotechnical inspections would be required at not greater than 1.5m depth intervals, but, subject to the quality of the rock below the toe of the shoring piles, closer inspections and/or staged excavation, underpinning and inspections may be required. Where adverse joints or defects are encountered some additional stabilisation of rock cuts will be required as excavation proceeds. This may include rock bolts and shotcrete protection etc. Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

Option 2

Alternatively, considering that sandstone bedrock will likely be encountered slightly above the proposed BEL, consideration could be given to the installation of a full depth shoring wall that is socketed into the sandstone below BEL. This is our preferred methodology as it will reduce the risk posed by the presence of adverse defects within the better-quality rock mass requiring stabilisation and the need to temporarily prop and/or anchor the piled wall. Cantilevered retaining walls supporting heights up to 3m to 3.5m are commonly adopted, though this assumption must be checked by the structural engineer. However, we note that the cantilevered walls will need to extend into the underlying sandstone bedrock of medium to high strength. Consequently, we recommend that piling contractors be contacted to assess the suitability of their equipment to form the required pile sockets.

In this regard the design approach would be to design the wall to provide full support to the materials above the better-quality bedrock and allow for a nominal behind wall pressure where the wall extends through the better-quality bedrock. This nominal behind wall pressure would allow for the support of small wedges of rock that may be present in the face but not for larger more persistent defects.

4.6.2 Pool Excavation

Excavation to a maximum depth of about 1.8m will be required for the pool excavation and will be set-back from the eastern boundary by about 1.8m. Based on the DCP test results near the proposed pool excavation, sandstone bedrock is inferred at depths of about 0.28m (DCP6), 0.9m (DCP6) and 1.55m (DCP1). As such, where the depth to low strength sandstone is shallow (i.e DCP5 and DCP6), temporary batters formed through the soils and then vertical rock cuts through the low strength (or better) sandstone bedrock may be feasible. Temporary batters and vertical rock cuts must be carried out based on the recommendations provided above.

To confirm the depth to sandstone, a series of test pits should be excavated using an excavator at the commencement of construction along the length of the proposed pool and inspected by a geotechnical engineer. Further advice can then be provided following this inspection on an appropriate retention methodology.

4.7 Retaining Wall Design Parameters

The major consideration in the selection of earth pressures and parameters for the design of the retention system is the need to limit deformations occurring outside the excavations. The characteristic earth pressure coefficients and subsoil parameters provided below may be adopted for the design of the retention systems:

- Where cantilevered or gravity walls are adopted, they should be designed to resist a triangular earth pressure distribution. Where movement sensitive structures are not present within the zone of influence of the excavation (which is defined as everything above a line drawn upwards from bulk excavation level at 1 Vertical(V):2 Horizontal(H)) a coefficient of active lateral earth pressure, K_a, of 0.35 for the retained soils may be adopted, assuming a horizontal surface behind the wall. Where movement sensitive structures are located within the zone of influence of excavation, a coefficient of lateral earth pressure, K, of at least 0.55, for the retained soils should be adopted, assuming a horizontal surface behind the wall. Any sloping ground/backfill must be added to the above pressures as a surcharge load.
- Propped or anchored walls supporting soil and weathered bedrock of less than low strength should be designed to resist a rectangular earth pressure distribution. Where movement sensitive structures are not present within the zone of influence of the excavation, a rectangular lateral pressure of 6H kPa may be adopted (where the depth is defined as H). Where movement sensitive structures are present within the zone of influence of the excavation, a rectangular lateral earth pressure distribution of 8H kPa may be adopted. Any sloping ground/backfill must be added to the above pressures as a surcharge load.

- Where the pile wall extends to below the BEL, we recommend that where better-quality bedrock is present, that a nominal back of wall pressure of 5kPa be adopted to account for small unstable wedges that may be present within this better-quality bedrock.
- All surcharge loads, such as from the sloping ground above the walls, construction equipment, stockpiles, structures, etc. and appropriate hydrostatic pressures should be added to the above pressures.
- Bulk unit weights of $20kN/m^3$ and $22kN/m^3$should be adopted for the soil and weathered bedrock profiles, respectively.
- Where walls are designed as drained, they should be provided with complete and permanent drainage of the ground behind the walls with drainage exiting at the base of the shotcrete. The subsoil drains should incorporate a non-woven geotextile fabric (eg. Bidim A34), to act as a filter against subsoil erosion. All behind wall drainage should be connected to the stormwater system and disposed of, in a controlled manner to council stormwater systems.
- Toe resistance of the piled walls may be achieved by socketing the pile or dowels into the sandstone bedrock of at least medium strength below bulk excavation level and may be designed based on an allowable lateral pressure of 300kPa. This assumes that full passive restraint can be mobilised in the rock and that features such as excavations in front of the toe of the wall do not reduce the available capacity. In this regard we recommend that when calculating the required depth of embedment needed for lateral restraint, the first 0.3m of the socket below bulk excavation and all localised excavations be ignored. This allows for accidental over-excavation or similar. A minimum embedment depth (ignoring the 0.3m allowance above) of 1m should apply. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, lift pits, buried services, etc. must be taken into account in the wall design. All retaining wall designs should be reviewed by a geotechnical engineer prior to construction to confirm that appropriate design values and principles have been adopted.
- Anchors or bolts may be designed based on a preliminary allowable bond strength of 150kPa in weathered bedrock of at least low strength and 300kPa in medium strength or better bedrock. Anchors should have free and bond lengths of at least 3m. Temporary anchors used for lateral support should be bonded below a line drawn up at 45° from bulk excavation level. Anchors should be proof stressed to at least 1.3 times their working load and then locked off at about 80% of the working load. Proof loading should be carried out in the presence of an engineer independent of the anchor contractor. Where temporary anchors extend below adjoining properties permission from the respective property owners must be obtained before installation.
- Long term support is understood to be provided by the built structure. Once the structure is built, temporary anchors or props must be destressed.
- The pool walls could be designed as drained, and measures taken to provide permanent and effective drainage of the ground behind the walls, though this would require a drainage trench to be excavated at and below the level of the base of the pool. Subsurface drains should incorporate a non-woven geotextile fabric such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the storm water system. More likely it would be appropriate to design the pool walls for hydrostatic pressures rising to, say, 0.5m above the rock level.

4.8 Footings

Due to the presence of uncontrolled clayey fill extending to a depth of greater than 0.4m and sandy fill to more than 0.8m depth, the site classifies as a 'Class P' in accordance with AS2870-2011. The uncontrolled fill is considered unsuitable as a bearing stratum or supporting subgrade for footings, slabs and pavements. Reference should also be made to AS2870-2011 for design, construction, performance criteria and maintenance precautions on 'Class P' sites.

Following excavation to the proposed BEL for the proposed garage, we expect that sandstone bedrock will be exposed over much of the northern portion of the excavation. Due to existing site levels, the excavation depth reduces to the south, and fill and residual soils may be exposed at the BEL. The proposed ground floor will extend beyond the footprint of the garage. Consequently, we recommend that all footings be uniformly founded on the sandstone bedrock to provide uniform support and reduce the risk of differential settlements. Pad and strip footings will be suitable where sandstone is exposed in the base of the excavations or at a relatively shallow depth, whilst where the depth to bedrock is greater bored piles are likely to be suitable.

Based on the results of the cored boreholes, the sandstone is typically of medium strength with few defects, though we note that a 0.16m thick seam was encountered within BH1 at a depth of 5.5m. Footings founded on the underlying medium strength sandstone bedrock may be designed for a maximum allowable bearing pressure of 2,000kPa, subject to inspection by a geotechnical engineer prior to pouring. Designing for a lower bearing pressure of 1,000kPa will simplify construction as it will reduce the likelihood of needing to deepen or widen footing excavations in the event lower quality rock is present.

Where footings are located near excavations, such as near the garage or pool excavations, or localised excavations (lift pits, buried services, etc.) they must be wholly founded below a line drawn upwards from the base of the excavation at 1V:1H. This includes the load carrying portion of the shaft of the pile. In addition to this, close inspection of the cut faces by a geotechnical engineer will be required to assess if any defect are present. If defects are present within the cut close to nearby footings, then these may need to be stabilised or the footings deepened to found below the defects.

Where piles are adopted, we recommend that the piles are drilled to a achieve a minimum embedment depth of about 0.5m into the appropriate quality of rock. For that part of the pile that extends below this nominal socket, a shaft adhesion of 10% in compression and 5% in tension of the above allowable bearing pressures may be adopted.

The allowable bearing pressure given above is based on a serviceability criteria of deflection at the pile toe or footing base of less than or equal to 1% of the pile diameter or footing width. Footings on rock can also be designed using 'Limit State Design' principles. For limit state design, higher ultimate bearing capacities would be adopted, used in conjunction with an appropriate geotechnical strength reduction factor (Φ_g) as determined from AS2159-2009 by the pile designer. Specific settlement analysis would be required where ultimate bearing pressures are adopted.

Prior to pouring concrete, all footings must be inspected by a geotechnical engineer to confirm that the design allowable bearing pressures are achieved. All footings should be free from all loose or softened materials prior to pouring. Where water ponds in the base of the footing excavation it should first be pumped dry and then all loosened or softened materials removed prior to pouring. Where piles are adopted it is recommended that they be poured on the same day as drilling.

4.9 Floor Slabs

We expect that at BEL for the proposed garage, a mixture of sandstone bedrock, residual soils and fill will be exposed. Similarly, we expect that following stripping for the proposed ground floor, the subgrade will expose predominately fill soils. Due to the variability in subgrade conditions, and difficulty in completing earthworks on a limited access site, we recommend that the ground floor slabs be designed to be fully suspended on the underlying sandstone bedrock. This will require piles to be drilled and socketed into the sandstone bedrock.

For the portion of the floor slab directly overlying sandstone bedrock, which will be limited to the northern portion of the garage, drainage will need to be provided below this portion of the slab either as a closely spaced grid of subsoil drains or a (single sized) gravel blanket. Where gravity drainage is not feasible it should be connected to a permanent fail-safe pump out system, which is fitted with automatic level control pumps to avoid flooding. If a drainage blanket is not adopted the basement slab should be designed with a grid of closely spaced drains and a subbase layer of at least 100mm thickness of crushed rock to TfNSW QA specification 3051 unbound base material (or other approved good quality and durable fine crushed rock), which is compacted to at least 100% of Standard Maximum Dry Density (SMDD). This subbase layer will provide a separation between the sandstone subgrade and the slab and provide a uniform base for the slab.

4.10 External Pavements

A concrete driveway is proposed within the southern portion of the site and will extend up from Ogilvy Road to the front of the house. The subgrade conditions for the driveway may comprise uncontrolled fill to a depth of 2.9m, based on the results of DCP4. It is likely that variable conditions will be encountered, particularly given the sloping and terraced nature of the site and the presence of numerous retaining walls within the area. Considering the difficulty associated with adequately undertaking earthworks on a small residential site, the likely presence of variable uncontrolled fill, and site access restrictions, we recommend that the driveway is designed as a fully suspended slab, supported on piles founded on the underlying sandstone bedrock. Notwithstanding this, fill may be placed as a formwork for the driveway. Should an on-grade pavement be preferred, then further advice from this office should be sought though we note that the risk of poor performance of the driveway must be accepted by the client.

4.11 Soil Aggression

The results indicate that the fill would have an exposure classification of “Non-Aggressive”, whilst the sandstone would have an exposure classification of “Mild“ when assessed in accordance with the criteria of concrete piling exposure classification given in Table 6.4.2 (C) of AS2159-2009 “Piling Design Installation”.

The above results also indicate that all samples would have an exposure classification of "Non Aggressive" when assessed in accordance with the criteria for steel piling exposure classification given in Table 6.5.2 (C) of AS2159-2009 "Piling Design Installation".

4.12 Sydney Water Assets

A Sydney Water Asset and easement extends within the site in a north-east to south-west orientation. The proposed garage excavation extends adjacent to the easement. Reference should be made to the Sydney Water Technical Guideline, Building Over and Adjacent (BOA) to Pipe Assets, for further advice in this regard.

Sydney Water will require a Specialist Engineering Assessment (SEA) to predict the potential impact the excavation and construction of the proposed development will have on their assets. The SEA will require input from both the geotechnical and structural engineer and will include finite element analysis (FEA). We can assist with the FEA. The SEA can take significant time for its preparation and for subsequent approval by Sydney Water, and so the SEA, should be completed at an early stage. A water services co-ordinator (WSC) should be engaged to help navigate the process.

4.13 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:

- Review of structural design to confirm the intention of this report are correctly interpreted.
- Completion of SEA for Sydney Water assets (if required).
- Dilapidation reports on adjoining properties. Sydney Water assets and Council assets.
- Inspection of test pits near the pool excavation to confirm the depth to rock.
- Vibration Monitoring during rock excavation where percussive excavation techniques are adopted.
- Inspection of initial shoring piles drilled.
- Witnessing installation and proof testing of anchors, if required.
- Progressive inspection by a geotechnical engineer of exposed rock conditions as the excavations proceed.
- Monitoring of groundwater seepage into bulk excavation.
- Inspection of all footing excavations, or bored pile drilling prior to pouring by the geotechnical engineer.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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 between the completed boreholes 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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CERTIFICATE OF ANALYSIS 355720

Client Details

Client	JK Geotechnics
Attention	Michel Hraibi
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**36816S, Proposed Residential Develop., CLONTARF**
Number of Samples	3 Soil
Date samples received	04/07/2024
Date completed instructions received	04/07/2024

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	11/07/2024
Date of Issue	09/07/2024

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Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		355720-1	355720-2	355720-3
Your Reference	UNITS	BH1	BH2	BH2
Depth		0.9-1.1	0.9-1.1	2.8-3.0
Date Sampled		01/07/2024	01/07/2024	01/07/2024
Type of sample		Soil	Soil	Soil
Date prepared	-	8/07/2024	8/07/2024	8/07/2024
Date analysed	-	8/07/2024	8/07/2024	8/07/2024
pH 1:5 soil:water	pH Units	6.8	6.2	5.0
Chloride, Cl 1:5 soil:water	mg/kg	20	29	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	32
Resistivity in soil*	ohm m	280	200	390

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	-			08/07/2024	[NT]	[NT]	[NT]	[NT]	08/07/2024	[NT]
Date analysed	-			08/07/2024	[NT]	[NT]	[NT]	[NT]	08/07/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	103	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	112	[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.

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: David and Christine LaRose c/o Archisoul Architects — **Ref No:** 36816S

Project: Proposed Residential Development — **Report:** A

Location: 24 Ogilvy Road, CLONTARF, NSW — **Report Date:** 3/07/24

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	2.25 - 2.28	0.6	12	A
	2.80 - 2.83	0.5	10	A
	3.07 - 3.10	0.4	8	A
	3.91 - 3.93	0.5	10	A
	4.20 - 4.23	0.5	10	A
	4.86 - 4.89	0.6	12	A
	5.10 - 5.13	0.6	12	A
	6.00 - 6.03	0.8	16	A
	6.45 - 6.48	1	20	A
	6.87 - 6.89	1.3	26	A
2	2.29 - 2.32	0.4	8	A
	2.73 - 2.76	0.3	6	A
	3.06 - 3.09	0.7	14	A
	3.85 - 3.87	0.6	12	A
	4.14 - 4.17	0.6	12	A
	4.93 - 4.95	0.5	10	A
	5.07 - 5.10	0.5	10	A
	5.83 - 5.86	0.6	12	A
	6.06 - 6.08	0.7	14	A
	6.42 - 6.45	0.6	12	A
	6.91 - 6.93	0.7	14	A
	7.16 - 7.18	0.2	4	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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BOREHOLE LOG

Borehole No.
1

Client: DAVID AND CHRISTINE LAROSE
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 24 OGILVY ROAD, CLONTARF, NSW

Job No.: 36816S **Method:** HAND AUGER **R.L. Surface:** ~31.1 m
Date: 1/7/24 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.M./B.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
					REFER TO DCP TEST RESULTS SHEET	31	0			FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, and root fibres.	M			APPEARS POORLY COMPACTED
▶						30	1			FILL: Silty sandy clay, low plasticity, dark brown, fine to medium grained sand, trace of fine to medium grained sandstone gravel, slag and root fibres.	w>PL			
						29	2			REFER TO CORED BOREHOLE LOG				
						28	3							
						27	4							
						26	5							
						25	6							

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CORED BOREHOLE LOG

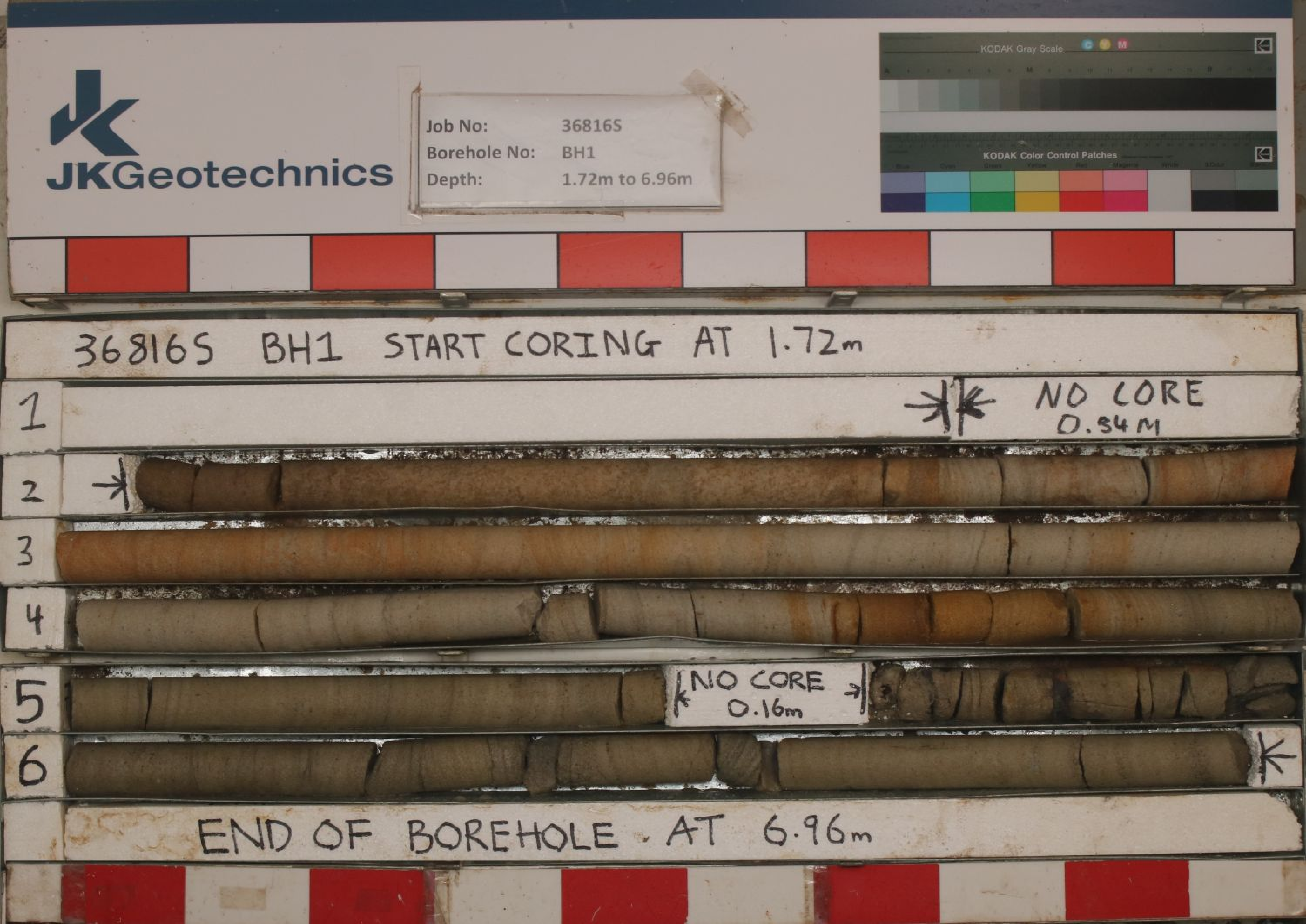
Borehole No. 1

Client: DAVID AND CHRISTINE LAROSE
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 24 OGILVY ROAD, CLONTARF, NSW

Job No.: 36816S **Core Size:** TT56 **R.L. Surface:** ~31.1 m
Date: 1/7/24 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** A.M./B.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
		30			START CORING AT 1.72m						
			2		NO CORE 0.34m						
75% RETURN		29 28 27	3 4		SANDSTONE: fine to medium grained, light grey and orange brown, distinctly bedded at 0-20°.	SW	M	0.60 0.50 0.40 0.50 0.50		(2.66m) Be, 0°, P, R, Cn (2.76m) Be, 5°, P, R, Clay Vn (4.64m) Be, 0°, P, R, Fe Sn (4.69m) Be, 10°, P, R, Fe Sn (4.81m) Be, 5°, P, R, Clay Vn	Hawkesbury Sandstone
		26	5		as above, but light grey.	FR		0.60 0.60			
					NO CORE 0.16m						
		25	6		SANDSTONE: fine to medium grained, light grey, distinctly bedded at 0-20°.	FR	M M - H	0.80 1.0 1.3		(5.68m) Be, 5°, P, R, Clay Ct (5.70m) CS, 0°, 5 mm.t (5.79m) CS, 0°, 10 mm.t (5.84m) Jh, 80°, P (5.95m) J, 80°, P, R, Cn (6.25m) Be, 20°, P, R, Cn (6.35m) CS, 0°, 10 mm.t (6.39m) J, 80°, P, R, Cn (6.58m) Be, 5°, P, R, Clay Vn	Hawkesbury Sandstone
		24	7		END OF BOREHOLE AT 6.96 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 36816S
Borehole No: BH1
Depth: 1.72m to 6.96m
KODAK Gray Scale
KODAK Color Control Patches
36816S BH1 START CORING AT 1.72m
1
NO CORE 0.54M
2
3
4
5
NO CORE 0.16m
6
END OF BOREHOLE AT 6.96m

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BOREHOLE LOG

Borehole No.
2
1 / 2

Client: DAVID AND CHRISTINE LAROSE
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 24 OGILVY ROAD, CLONTARF, NSW

Job No.: 36816S **Method:** HAND AUGER **R.L. Surface:** ~32.0 m
Date: 1/7/24 **Datum:** AHD
Plant Type: **Logged/Checked By:** A.M./B.S.

Groundwater Record	SAMPLES ES	U50	DB	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					REFER TO DCP TEST RESULTS SHEET					FILL: Silty sand, fine to medium grained, dark brown, trace of fine to coarse grained sandstone gravel, roots and root fibres.	D			APPEARS POORLY COMPACTED
ON COMPLETION OF CORING						31	1			FILL: Silty clayey sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, and root fibres.	M			HAND AUGER REFUSAL ON OBSTRUCTION IN FILL - COMMENCE CASING ADVANCE
						30	2			REFER TO CORED BOREHOLE LOG				
						29	3							
						28	4							
						27	5							
						26	6							

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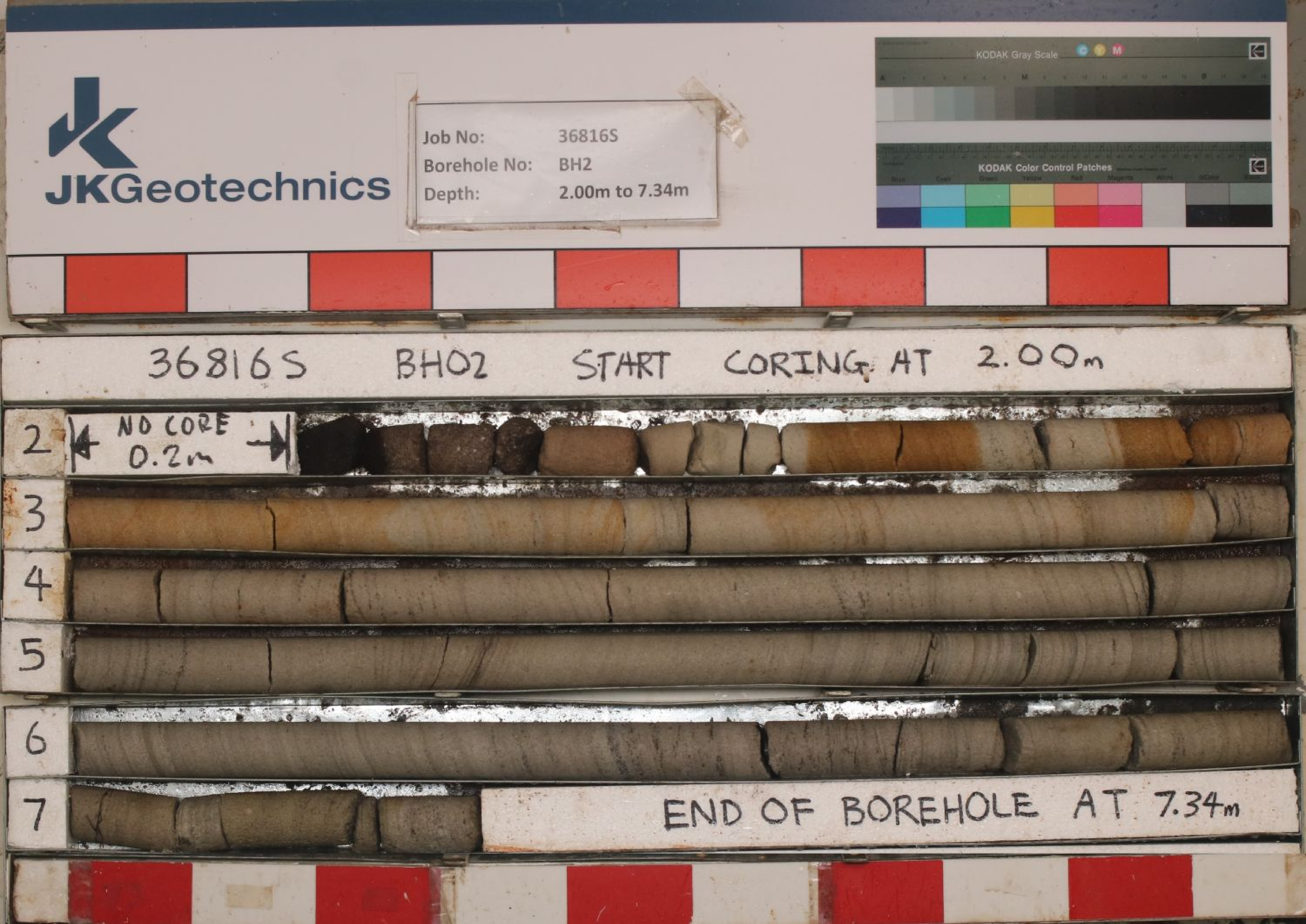
CORED BOREHOLE LOG

Client: DAVID AND CHRISTINE LAROSE
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 24 OGILVY ROAD, CLONTARF, NSW

Job No.: 36816S **Core Size:** TT56 **R.L. Surface:** ~32.0 m
Date: 1/7/24 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** A.M./B.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
		30	2		START CORING AT 2.00m						
					NO CORE 0.20m						
					SANDSTONE: fine to coarse grained, dark grey, indistinctly bedded at 0-5°.	HW	L	0.40		(2.21m) J, 70°, P, R, Cn (2.24m) Be, 20°, P, R, Clay FILLED	Hawkesbury Sandstone
		29	3		SANDSTONE: fine to medium grained, light grey and orange brown, distinctly bedded at 0-20°.	SW	M	0.30 0.70 0.60		(2.46m) Be, 5°, P, R, Cb Sn (2.52m) J, 60°, P, R, Cn (2.78m) Be, 10°, P, R, Clay Ct	
		28	4		as above, but light grey and occasional dark grey laminae.	FR		0.60			
		27	5					0.50 0.50		(4.88m) Be, 0°, P, R, Clay Vn	
		26	6					0.60 0.70 0.60		(5.69m) Be, 10°, P, R, Clay Vn	
		25	7					0.70 0.20		(6.75m) Be, 20°, P, R, Clay Vn (7.09m) Be, 0°, P, R, Clay Vn (7.25m) Be, 10°, P, R, Cn	
					END OF BOREHOLE AT 7.34 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 36816S
Borehole No: BH2
Depth: 2.00m to 7.34m
KODAK Gray Scale
KODAK Color Control Patches
36816S BH02 START CORING AT 2.00m
2
NO CORE 0.2m
3
4
5
6
7
END OF BOREHOLE AT 7.34m

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	DAVID AND CHRISTINE LAROSE		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	24 OGILVY ROAD, CLONTARF, NSW		
Job No.	36816S	Hammer Weight & Drop: 9kg/510mm	
Date:	1-7-24	Rod Diameter: 16mm	
Tested By:	A.M./M.H.	Point Diameter: 20mm	

Test Location	1	1A	2	3	4	5	6
Surface RL	≈31.1m	≈31.0m	≈32.0m	≈29.5m	≈28.9m	≈32.0m	≈33.7m
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK	SUNK	1	SUNK	1	EXCAVATED	SUNK
100 - 200	2	1	1	3	↓	8	3
200 - 300	3	2	↓	3	2	19/80mm	17
300 - 400	2	3	3	4	5	REFUSAL	9
400 - 500	2	6	3	2	10		11
500 - 600	5	10	3	1	8		2
600 - 700	3	8	3	2	9		1
700 - 800	3	10	1	2	7		2
800 - 900	6	14		6/30mm	5		5
900 - 1000	7	REFUSAL	↓	REFUSAL	3		REFUSAL
1000 - 1100	4		2		7		
1100 - 1200	5		2		3		
1200 - 1300	9		2		4		
1300 - 1400	9		1		4		
1400 - 1500	9		5		4		
1500 - 1600	11/50mm		7		5		
1600 - 1700	REFUSAL		3		2		
1700 - 1800			8/50mm		4		
1800 - 1900			REFUSAL		4		
1900 - 2000					1		
2000 - 2100					1		
2100 - 2200					2		
2200 - 2300					2		
2300 - 2400					2		
2400 - 2500					1		
2500 - 2600					4		
2600 - 2700					3		
2700 - 2800					6		
2800 - 2900					20		
2900 - 3000					REFUSAL		

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	DAVID AND CHRISTINE LAROSE		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	24 OGILVY ROAD, CLONTARF, NSW		
Job No.	36816S	Hammer Weight & Drop: 9kg/510mm	
Date:	1-7-24	Rod Diameter: 16mm	
Tested By:	A.M./M.H.	Point Diameter: 20mm	

Test Location	**7**						
Surface RL	≈31.2m						
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK						
100 - 200	↓						
200 - 300	1						
300 - 400	2						
400 - 500	5						
500 - 600	2						
600 - 700	2						
700 - 800	1						
800 - 900	1						
900 - 1000	1						
1000 - 1100	2						
1100 - 1200	1						
1200 - 1300	2						
1300 - 1400	4						
1400 - 1500	10						
1500 - 1600	REFUSAL						
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

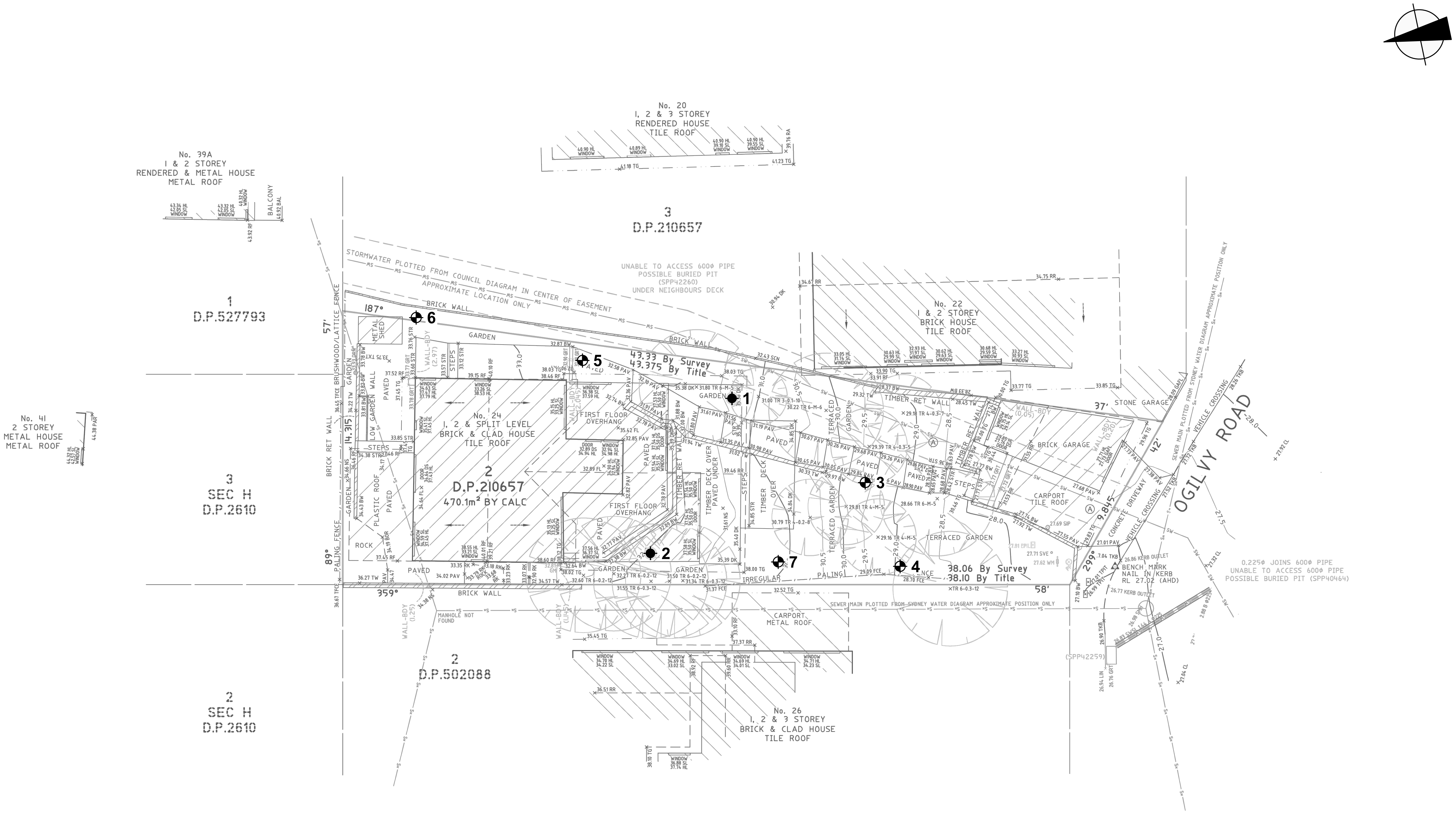
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	24 OGILVY ROAD, CLONTARF, NSW
Report No: 36816S	Figure No: 1

JKGeotechnics

PLOT DATE: 5/07/2024 3:46:15 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36816S CLONTARF\CAD\36816S.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST

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

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

Title:	**INVESTIGATION LOCATION PLAN**
Location:	24 OGILVY ROAD, CLONTARF, NSW
Report No: 36816S	Figure No: 2

JKGeotechnics

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

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.

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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:

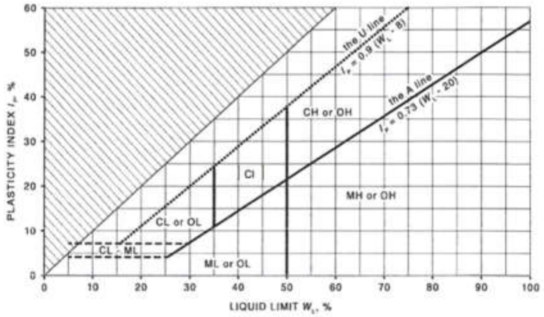
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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 $Is_{(50)}$ (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