REPORT TO

AMANDA AND CHRISTOPHER SMITH

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

47 EAST CRESCENT STREET, LAVENDER BAY, NSW

Date: 7 May 2025
Ref: 37572Srpt

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed alterations and additions at 47 East Crescent Street, Lavender Bay, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Amanda Smith by signed 'Acceptance of Proposal' form and was carried out on the basis of our fee proposal, Ref. P71443S, dated 11 April 2025.

Based on the provided architectural drawings prepared by Michael Bell Architects Pty Ltd (Job No. 528, Drawing Nos. DA-01 to DA-23, dated 17 April 2025), we understand the proposed alterations and additions include the following:

- Demolition of the northern portion of the house and garage.
- Lowering of the existing courtyard on the western side of the house to RL31.74m, which will result in excavation to about 3.5m depth below existing ground levels. The proposed courtyard excavation will extend up to the western boundary wall (which will remain).
- Lowering of the main portion of the existing basement to a floor level at RL31.84m, resulting in excavation to about 1m depth, to form a full height basement level. In addition, the basement will be extended to the north by about 0.6m to 1m. The basement extension and deepening will extend up to the eastern common wall with No.45 East Crescent Street, and within about 7.5m of the northern boundary.
- Construction of a two-storey extension to the north of the existing house.

The purpose of the preliminary 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 and retention, footings, hydrogeology and floor slabs.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 17 April 2025 and comprised four Dynamic Cone Penetration (DCP) tests, DCP1 to DCP4, to refusal depths ranging between 0.15m and 0.85m below existing ground levels. The DCP tests were used to probe down to the surface of the underlying inferred bedrock.

The investigation was designed to minimise the amount of disruption to the site, and therefore the composition of the soils overlying the inferred bedrock are unknown. Similarly, it is noted that DCP refusal may also occur on inclusions within the fill, harder iron indurated bands in residual soils, or other hard layers. The tests do not provide any indication of rock strength.

The DCP test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the DCP test result sheet, were estimated by interpolation between spot levels shown on the survey plan by Daw & Walton Consulting Surveyors (Job No. 6331-24, Sheet 1, Revision 3, dated 11 March 2025), and existing floor plans provided within the architectural drawings. The datum of the levels is the Australian Height Datum (AHD).

Our senior geotechnical engineer, Ben Sheppard, was on site full time during the fieldwork to make observations of existing footings, et out the investigation locations and record the DCP test results. For details of the investigation techniques adopted, their limitations and a glossary of logging terms and symbols used, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on a hillside which steps and slopes down to the east towards Lavender Bay with an average overall slope of about 15°. The site is bound to the south and west by East Crescent Street and Waiwera Street, respectively, and by residential properties on its remaining sides. The site is a duplex, having a common wall along the eastern side with No.45 East Crescent Street.

At the time of the fieldwork, the site contained a two-storey duplex, which broadly covered the southern portion of the site. A single storey garage was situated within the north-western corner, on the Waiwera Street frontage. Both structures generally appeared to be in good external condition based on a cursory inspection. Paved courtyards covered the remainder of the site footprint. A basement was situated below the majority of the ground floor. The basement was tiled along the eastern and northern sides which formed a cellar and storeroom, whilst the southern and western sides comprised a low-height sub-floor space. Sandstone bedrock was observed outcropping at three locations within the basement level, and based on a tactile examination using a geopick, the sandstone assessed was to be of very low and very low to low strength with extremely weathered bands, and is shown in the below Plates 1, 2 and 3 and shown on Figure 2.

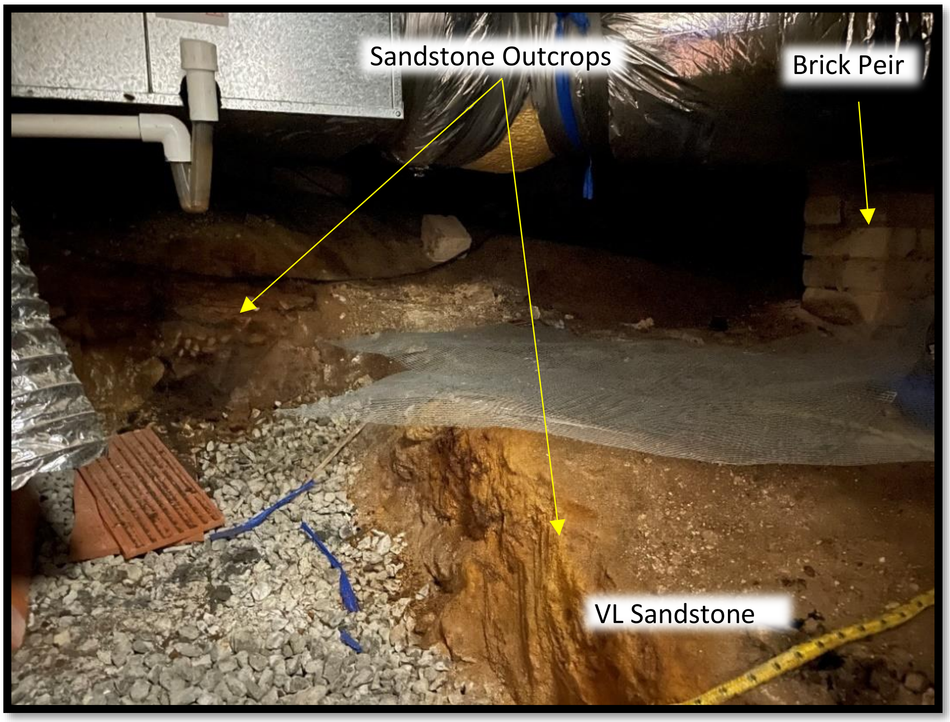

Plate 1 – Centrally within basement

Plate 2 – Eastern part wall at southern end

Plate 3 – Southern portion of basement

The neighbouring property to the east, No.45 East Crescent Street, formed the adjoining structure of the duplex, and generally appeared to be in good external condition. Ground levels along the common boundary are generally unknown.

The neighbouring property to the north, No.1A Waiwera Street, contained a two-storey brick house, with a concrete driveway extending between the house and the common boundary. Ground levels generally appeared to be similar to those of the subject site at the Waiwera Street frontage, reducing to below the subject site towards the east. Sandstone bedrock was seen outcropping below the boundary wall and had an exposed height of up to about 2.5m.

3.2 Subsurface Conditions

The Sydney 1:100,000 Geological Series Sheet 9130 indicates that the site is mapped to be underlain by Hawkesbury Sandstone of the Wianamatta Group.

Sandstone bedrock has been inferred at the DCP refusal depths, as tabulated below:

DCP Test	Approximate Surface RL (mAHD)	Approx. Depth to Inferred Sandstone Bedrock (m)	Approximate Surface RL of Inferred Sandstone Bedrock[1] (mAHD)
1	RL35.0	0.55	RL34.4
2	RL34.9	0.85	RL34.0
3	RL32.8	0.8	RL32.0
4	RL32.8	0.15	RL32.6

Note 1 – Rounded down due to the approximate nature of the interpolated Reduced Levels

Sandstone was outcropping within the existing basement level, as shown on the attached Figure 2 and Plates 1 to 3. The sandstone was generally assessed to be of very low and very low to low strength, and also contained extremely weathered bands. Based on tape measurements from the underside of the ground floor, the top of the exposed sandstone bedrock within the basement has been assessed to be at about RL33.8m (Plate 1), RL33.4m (Plate 2) and RL33.7m (Plate 3).

The eastern wall footing was observed to be founded on sandstone bedrock of very low to low strength at the southern end. The footing had an outstand of about 50mm.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principle Geotechnical Issues

The main geotechnical issues for the site and the proposed development are summarised as follows. These issues are discussed in more detail in the following sections of this report:

- The excavation adjacent to and below the existing house for the proposed basement deepening and extension will be complex and there will be a risk of movement and damage to portions of the existing house that will remain, even assuming the works will be carried out by a suitably experienced and insured contractor under the guidance of JKG and a structural engineer.
- How the proposed courtyard excavation and northern basement extension excavations will be retained during construction and in the long-term will be subject to the depth and quality of the underlying sandstone bedrock. The sandstone exposed within the basement is poor quality and not deemed to be self-supporting. Therefore, it is possible that full depth underpinning and/or shoring walls will be required to retain the proposed excavations. In this regard, we strongly recommend that two cored boreholes are drilled to confirm the depth to and quality of the sandstone bedrock, as this will have implications regarding the assessment of whether the sandstone bedrock will likely be stable when cut vertically. The recommendations provided in this report will then need to be reviewed and possibly updated following the additional investigation.
- Deepening of the existing basement will require excavation below existing footings. These will need to be underpinned to below the proposed bulk excavation level prior to bulk excavation. An underpinning methodology will need to be developed by the structural engineer. For existing footings which will remain external to the basement footprint or where they are founded much higher than the proposed excavation level which makes underpinning difficult, temporary piles founded below the excavation level and needle underpins may be required to support these footings.
- A Sydney Water asset runs near the western boundary and may be deemed to be within the zone of influence of the proposed courtyard excavation. How these assets will be managed and the potential impact of the proposed development on these assets will need to be addressed in the early stages of design. Approval from Sydney Water may be required.

4.2 Detailed Inspection and Methodology Plans

Care will need to be taken during all works on the site, particularly during excavation adjacent to and below any parts of the existing structure, that the risk of damage to the existing structure and adjoining structures are reduced as much as possible. Prior to any works commencing we recommend the following be carried out;

1. Cored boreholes are drilled to confirm the feasibility of adopting vertical rock cuts in the temporary and permanent case.
2. A detailed assessment of the adjoining properties, including dilapidation surveys.

3. Preparation of a detailed methodology plan which is provided to the structural and geotechnical engineers for review and approval. The methodology plan should include details of all demolition, shoring and construction stages, as well as measures taken to ensure support is maintained to the existing and adjoining structures at all times.
4. Preparation of a detailed inspection and test plan, prepared by the geotechnical and structural engineers to ensure that the works are carried out with appropriate supervision and hold points.

4.3 Dilapidation Reports

Prior to the commencement of demolition, we recommend that detailed dilapidation reports be prepared for the neighbouring properties to the east (No.45 East Crescent Street), north (No.1A Waiwera Road) and possibly the Council's verge and footpath. The reports can then be used as a benchmark for the assessment of damage that may occur to the adjoining properties during the work and in this way would also help to guard against opportunistic claims for damage that was present prior to the start of the work.

The dilapidation surveys should comprise detailed inspections of the adjoining properties, both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the relevant dilapidation report and asked to confirm in writing that it presents a fair representation of existing conditions.

4.4 Excavation

4.4.1 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.

The proposed excavation areas are as follows:

1. Excavation to depths of about 3.5m will be required for the proposed courtyard and will extend adjacent to the western boundary wall.
2. Excavation to depths of about 3.5m will be required for the extension of the existing basement to the north behind the existing basement wall.
3. Lowering of the existing basement floor level will require excavation to about 1m depth

Based on the investigation results and inferred subsurface profile, the proposed excavations are anticipated to encounter a limited depth of soils (fill and/or residual soils) overlying sandstone bedrock. At this stage, the quality of the sandstone bedrock is generally unknown, although from exposures within the basement, the sandstone may be very low and very low to low strength. We anticipate the site will be accessible for tracked excavators for the courtyard and northern extension excavations, and possibly for very small tracked excavators or hand operated equipment within the existing basement.

Excavation of the soil profile and any extremely weathered or very low strength bedrock may be completed using a ‘digging’ bucket fitted to a small to moderate size hydraulic excavator. Bedrock of low or higher strength will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders and rock saws. Grid sawing techniques in conjunction with ripping or hammering will help to facilitate excavation. Hand operated jackhammers may be quite productive if the rock is confirmed to be of low strength.

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.

4.4.2 Excavation Vibrations

Considerable caution must be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to the existing house and adjoining structures (which are likely founded on bedrock). The use of a hydraulic rock hammers is not preferred due to the risk of vibrations potentially damaging the existing structure. If excavation using a hydraulic rock hammer is attempted, the hammer should be as small as possible and the vibrations monitored as discussed below.

The vibrations transmitted to the existing and neighbouring structures to the north and east must be quantitatively monitored at all times during rock hammer use. Vibration monitors should be solidly fixed to the existing footings, with the monitors attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached at which point such excavation techniques should cease. It is likely that the vibration monitors will need to be moved to different parts of the existing structure as the excavation progresses.

Vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec. However, if the structure is considered to be sensitive, then a lower target limit may be appropriate. The appropriate limit should be assessed by the structural engineer following review of the dilapidation reports.

If higher vibrations are recorded than the target limits, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. However, any on site warning devices can only be set against the PPV and not the associated vibration frequency so will need to be set for the lower PPV values. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative lower percussion techniques as discussed below.

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.

We recommend use of excavation contractors with experience in such work and with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

Where hand-held tools are used, such as hand-held jackhammers, then such equipment is unlikely to produce damaging vibrations during bedrock excavation and therefore vibration monitoring is not required. However, if concerns are raised regarding potentially damaging vibrations caused by rock excavation using this equipment, then some initial vibration monitoring may be carried out to demonstrate the vibration levels are within acceptable limits.

4.5 Existing Footings

As mentioned above, the proposed excavations will extend adjacent to and below existing footings and will therefore be complex and must be carried out with care to ensure that the existing footings are not undermined or rendered unstable.

The details of the existing building footings will need to be confirmed to assess how the underpinning can be carried out. Investigation of the footings could be carried out prior to construction by the excavation of test pits to expose the existing footing, but excavation of such test pits would be difficult without some initial clearing of the proposed excavation areas. Therefore, we consider it may be more practical to determine the footing details during construction when the areas have been cleared.

Test pits should be excavated at locations advised by the geotechnical and structural engineers to expose the base of the footings and the foundation material. Once exposed the footings should be inspected by the geotechnical engineer and structural engineer to assess how the underpinning of the footings can be carried out. If the existing footings are founded below the base of the proposed excavations, then underpinning of the footings would not be required. However, if the footings are founded on soils or poor quality sandstone above the base of the proposed excavation then they will need to be underpinned prior to bulk excavation. Based on our observations of the quality of the rock within the basement, it is likely that all footings within the basement will need to be underpinned to below the proposed bulk excavation level.

A detailed underpinning methodology must be developed by the structural engineer and reviewed by the geotechnical engineer prior to commencing such works. Any underpinning should be carried out by the excavation of discrete sections with each section fully underpinned prior to excavation of the adjacent sections. Regular geotechnical and structural inspections would be required during the underpinning works.

Some footings, such as for the bay windows on the western side of the existing house will remain and are likely to be founded above the existing basement level. Similarly, the western wall of the structure may be founded much higher than the existing basement level as a result of the sandstone stepping up to the west. Where underpinning footings to the bulk excavation level is not feasible (due to the depth of underpinning required), then these footings could be underpinned using temporary piles founded below the proposed excavation level and needle beams or corbels installed to support the footing. Once excavation is completed, a permanent wall/column can be constructed from the excavation level to support the footing in the permanent case.

4.6 Batters and Retaining Walls

We strongly recommend that two cored boreholes be completed to confirm the depth to and quality of the sandstone bedrock, as this will have implications regarding the assessment of whether the sandstone bedrock has sufficient strength to be cut vertically and left unsupported both in the short and long term. The recommendations provided in this report will then need to be reviewed and possibly updated following the additional investigation. We can provide a fee proposal for this additional work, if requested to do so.

4.6.1 Courtyard Excavation

For the courtyard excavation, soils are anticipated to be encountered to depths of about 0.5m to 1m, and then sandstone bedrock. Given the excavation extends adjacent to the western boundary wall, there is insufficient space to form temporary batters and the upper soils and poor quality sandstone bedrock will need to be supported prior to excavation by an engineer designed shoring wall. Based on the exposures evident in the basement, we anticipate that the bedrock may be of poor quality and not considered suitable to be self-supporting. As such, the proposed excavation will need to be retained by a full-depth shoring wall socketed below the proposed BEL. This will result in a reduced basement footprint, as the piles will need to be drilled adjacent to the existing western boundary wall. The shoring wall may need to be laterally restrained using internal props or temporary anchors (provided permission from council is obtained).

Assuming the soils comprise sandy fill, a contiguous pile wall will be required as the soils are unlikely to stand vertical prior to the placement of shotcrete panels. 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, although allowance should be made for the use of temporary or sacrificial casing, where collapse or seepage is an issue.

We note that the shoring wall will need to extend into the underlying sandstone bedrock, which may contain high strength iron indurated bands, or better-quality bands. We recommend that piling contractors be contacted to assess the suitability of their equipment to form the required pile sockets.

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.

Where the sandstone bedrock is shown to be good quality and deemed to be self-supporting from the recommended cored boreholes, then the upper soils and any poor quality bedrock may be retained by either underpinning the western boundary wall or construction of a concrete block or gravity wall founded on the good quality sandstone. Each method will require careful consideration of how to maintain stability of the oversteep batters and in this regard, they should be progressively installed. Further advice on these methods may be provided following the additional investigation.

Good quality sandstone may be cut vertically and is discussion in more detail in Section 4.6.4.

4.6.2 Northern Basement Extension

It appears that there is sufficient room to form temporary batters within the fill and poor quality bedrock along the northern side, however some form of shoring will be required for the return along the eastern boundary and similar methods to those discussed above may be considered.

Temporary batters formed through fill and residual sandy soils should be formed at no steeper than 1 Vertical to 1.5 Horizontal (1V:1.5H) and through residual clays and sandstone bedrock of less than low strength at 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. 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. In the long-term, cantilever or propped block retaining walls can be constructed in front of the temporary cut batter slopes, and then backfilled on completion of excavation.

4.6.3 Sandstone Cut Faces

Competent sandstone bedrock (low or higher strength) may be cut vertically in the short term, subject to geotechnical inspection. Such inspections should be carried out at depth intervals of no more than 1m to 1.5m to assess if any additional support of the sandstone is required. Any additional support recommended by the geotechnical engineer must be installed prior to further excavation. Stabilisation may comprise shotcreting and bolting (subject to permission from the neighbours and council). Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

Good quality sandstone bedrock may be left unsupported in the long term, subject to geotechnical inspection, but the exposed sandstone will deteriorate and fret with time. This may lead to debris collecting at the base of the cut face, which will need to be cleared from any drains to prevent the drains becoming blocked and causing water issues for any walls in front. If such maintenance to clear the drains cannot be achieved then retaining walls should be constructed in front of the cut faces and the gap filled with gravel or the cut faces covered with shotcrete.

4.6.4 Retention 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:

- For the design of retaining walls propped by other structural elements, underpins or where movements are to be reduced, a triangular earth pressure distribution may be used with an 'at rest' lateral earth pressure coefficient, K_o, of 0.6, for the soil and weathered rock profile, assuming a horizontal backfill surface.
- For the design of cantilevered retaining walls where some movements are tolerable, a triangular earth pressure distribution may be used with an 'active' earth pressure coefficient, K_a, of 0.35 may be adopted for the soil and weathered rock profile, assuming horizontal backfill.
- Bulk unit weights of $20kN/m^3$ and $22kN/m^3$should be adopted for the soil and poor quality weathered bedrock profiles, respectively.
- For walls which support the good quality bedrock profile, a nominal lateral pressure of 5kPa should be adopted to account for small wedges of rock applying lateral loads on the back of retaining walls.
- For conventional retaining wall footings, shoring piles or underpins keyed into the bedrock below BEL, an allowable lateral bearing pressure of 200kPa may be adopted for bedrock of at least very low strength. The key/socket depth should commence below the base of any nearby excavations such as for service trenches or footings, and also below a nominal allowance for over-excavation or fracturing during excavation of say, 0.3m.
- Any retaining or shoring walls supporting a soil and bedrock profile must be designed as permanently drained and PVC pipes should be installed at nominal 1.2m horizontal spacing just above the bedrock surface and just above BEL. Holes will need to be drilled to allow installation of the pipes and/or use gaps between contiguous piles. The end of the pipe penetrating the retained soils behind the retention system must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The pipes should discharge into the perimeter drainage system.
- Lateral restraint of the shoring walls may be provided by internal props or temporary anchors. Temporary anchors or bolts may be designed based on a preliminary allowable bond strength of 200kPa in weathered bedrock of at least low strength. Stressed 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. All anchors should be proof loaded to at least 1.3 times their working load and then locked off at approximately 85% of their working load. Proof loading should be carried out in the presence of an engineer independent of the anchor contractor. Anchors must be bonded behind a line drawn up at 45° from the base of the excavation, with all anchors having a free length and bond length of at least 3m each. Lift off tests should be carried out on at least 10% of all anchors 24 to 48 hours following locking off to confirm that the anchors are maintaining their load.
- Long term support is understood to be provided by the built structure. Once the structure is built, temporary anchors or props must be destressed.
- Where temporary batters are adopted, consideration will need to be given to the type of backfill used. Backfill behind retaining walls should comprise engineered fill. Compaction of engineered fill behind

retaining walls is very difficult. The use of a single sized durable aggregate, such as ‘blue metal’ or recycled concrete, which do not require significant compactive effort, is often preferred if good performance is a priority. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. Where there is only a narrow gap between the wall and rock face, a poker vibrator can be used to ensure the gravel fully occupies the void space. A non-woven geotextile filter fabric (such as Bidim A34) should be placed as a separation layer over the cut face/batter to control subsoil erosion into the voids of the aggregate. The geotextile should be wrapped over the surface of the gravel backfill and capped with at least a 0.3m thick layer of well compacted clayey fill or a pavement to reduce infiltration of surface water into the backfill. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required.

4.7 Drainage

Only DCP testing has been completed at this stage, and therefore, groundwater observations are not possible with such testing equipment.

Notwithstanding, we expect that seepage into the excavation may occur as local seepage flows within fill, at the fill/residual soil interface, at the residual soil/bedrock interface and through joints and bedding partings within the bedrock profile, particularly after heavy or prolonged rain. If seepage does occur, it is likely to be the result of local infiltration, be intermittent and of a small flowrate, and should be readily controlled during construction by sump and pump methods to the Council’s stormwater system for disposal.

In the long term, drainage will need to be provided behind any retaining walls and below any floor slabs that overlie bedrock to intercept ephemeral seepage and dispose of this directly to Council’s stormwater system, presumably via a pump-out pit with fail-safe pump system. The underfloor drainage should comprise a strong, durable, single-sized aggregate such as ‘blue metal’ gravel. The completed excavation should be inspected by the hydraulic consultant to confirm that the drainage system is adequate for the actual seepage flows.

4.8 Footings

Sandstone bedrock is expected to be exposed at bulk excavation level within the proposed basement. For the proposed two storey extension to the north of the basement, sandstone bedrock is also expected to be present at relatively shallow depths. Therefore, we recommend that all new footings are uniformly founded within the bedrock to prevent issues of differential settlement associated with different foundation materials.

Pad and strip footings, and piles, founded within at least very low strength bedrock may be provisionally designed for an allowable bearing pressure of 800kPa. All footings must be founded below the depth of any locally deeper excavations (basements, lift pits, service trenches, etc.).

The proposed extension extends beyond the basement footprint and therefore, footings will be situated near the crest of vertical rock cuts or shoring walls. All footings must be founded behind a line inclined up from the toe of all cut faces at 45° and in this regard, this will require piles to be socketed to below this line. Alternatively, where it is found that the rock is of high quality and can be inspected by a geotechnical engineer, then shallow footings may be adopted above this line where the geotechnical engineer deems the rock can adequality support the structural loads.

As a minimum requirement, the initial stages of footing excavation should be inspected by a geotechnical engineer to confirm that the recommended foundation has been reached and to check initial assumptions about foundation conditions. The need for further inspections can be assessed following the initial visit.

All footings should be excavated, cleaned, inspected and poured with minimal delay. If delays in pouring high level footings on weak, weathered rock are anticipated we recommend that the footing base be covered with a protective blinding layer of concrete.

4.9 Basement Floor Slab

Based on the investigation results, we expect sandstone bedrock will be exposed at bulk excavation level within the proposed basement. We therefore recommend that an underfloor drainage blanket be provided. The drainage material should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead any transient groundwater seepage to a sump for disposal to the stormwater system.

4.10 First Floor Level Construction

The proposed extension to the north will extend beyond the footprint of the basement. We anticipate the sub-surface to comprise uncontrolled fill overlying sandstone bedrock and therefore, following demolition and stripping to the design floor level, the subgrade is likely to comprise uncontrolled fill. We are unaware of any records of placement or compaction control of the fill and as such it must be considered 'uncontrolled'. Due to this and the poorly compacted nature of the fill, it is not considered desirable to support floor slabs on this fill due to the risk of differential settlements. As such we recommend that the floor slab be designed as suspended on the underlying bedrock to reduce the potential for differential movements occurring.

Alternatively, the uncontrolled fill may be completely excavated to expose the underlying sandstone bedrock, and levels then raised using engineered fill. If this option is preferred then additional advice may be sought from this office.

4.11 Sydney Water

A 100mm diameter cast iron concrete lined Sydney Water asset extends within Waiwera Road near the western boundary of the site.

Liaison with, and approval by, Sydney Water may be required, depending on the depth of the asset and set-back from the proposed works. Should approval from Sydney Water be required, then they will require a Specialist Engineering Assessment (SEA) to predict the potential impact the excavation and construction of the proposed development will have on their asset. 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.

4.12 Further Geotechnical Input

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

- Additional investigation comprising at least two cored boreholes drilled to depths of about 3m below the proposed BEL and preferably some test pits to expose existing footings.
- Review of structural drawings to check good geotechnical principles have been embodies in the design.
- Review and approval of the proposed demolition and excavation methodology prepared by the contractor.
- Geotechnical analysis of the shoring walls and Sydney Water assets using FE software.
- Inspection of test pits to determine the foundation material below the existing footings if not completed previously.
- Regular inspections of the conditions encountered during excavations below the house.
- Quantitative Monitoring of transmitted vibrations if a rock hammer is used.
- Proof testing of temporary anchors (if required).
- Inspection of any rock cut faces.
- Inspection of underpinning excavations and footings.
- Inspection by a hydraulic or geotechnical engineer, during construction and/or once the bulk excavation has been carried out to provide comments regarding subfloor drainage.

5 GENERAL COMMENTS

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

Occasionally, the subsurface conditions 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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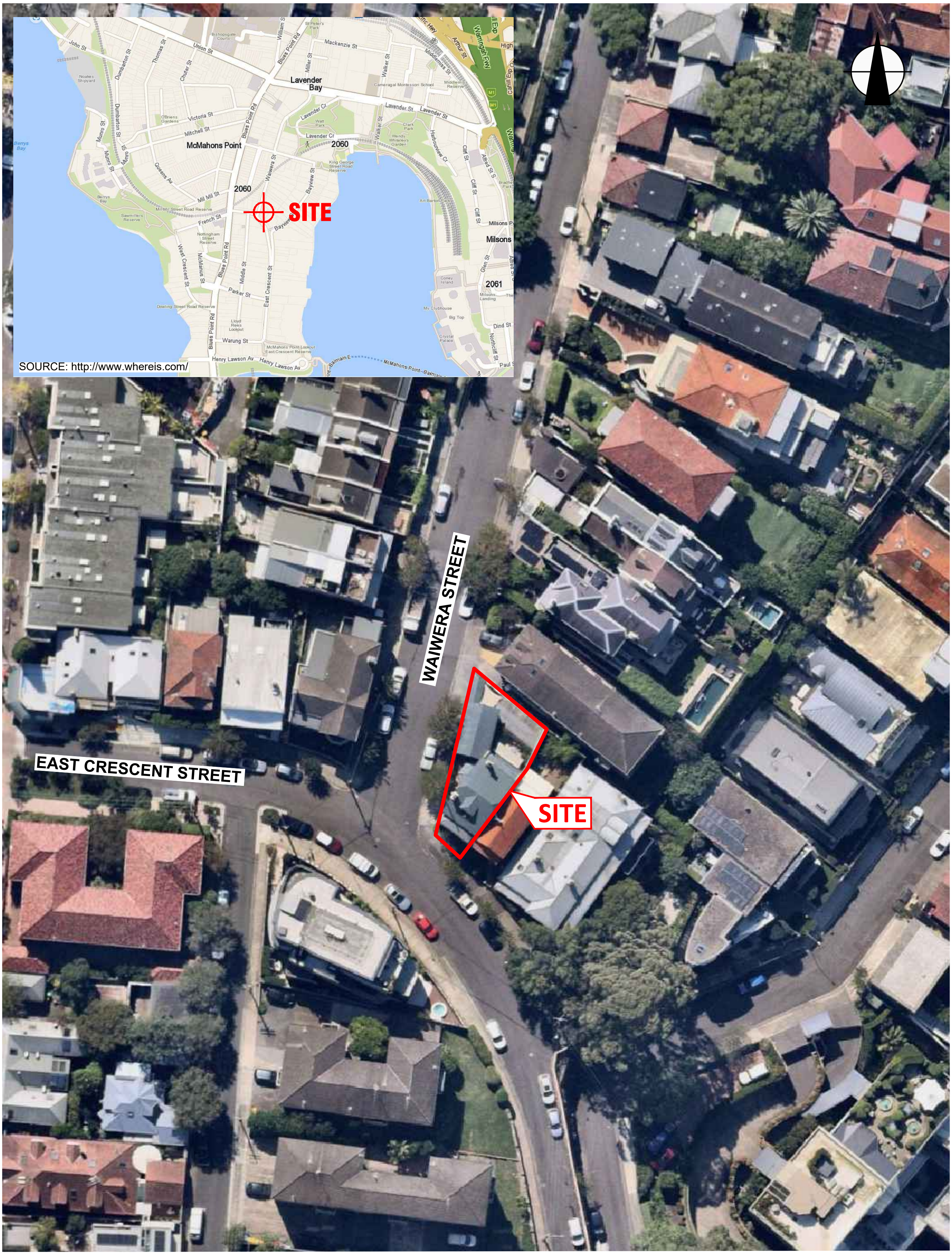
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	AMANDA AND CHRISTOPHER SMITH		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	47 EAST CRESCENT STREET, LAVENDER BAY, NSW		
Job No.	37572S	Hammer Weight & Drop: 9kg/510mm	
Date:	17-4-25	Rod Diameter: 16mm	
Tested By:	B.S.	Point Diameter: 20mm	

Test Location	1	2	3	4			
Surface RL	≈35.0m	≈34.9m	≈32.8m	≈32.8m			
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	DRILLED	DRILLED	DRILLED	DRILLED			
100 - 200	↓	↓		5/50mm			
200 - 300	1	1	↓	REFUSAL			
300 - 400	1	↓	3				
400 - 500	1	1	5				
500 - 600	11/50mm		7				
600 - 700	REFUSAL		9				
700 - 800		↓ *	9				
800 - 900		9/50mm	4/0mm				
900 - 1000		REFUSAL	REFUSAL				
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
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
4. * Denotes rod wet on extraction

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title: **SITE LOCATION PLAN**

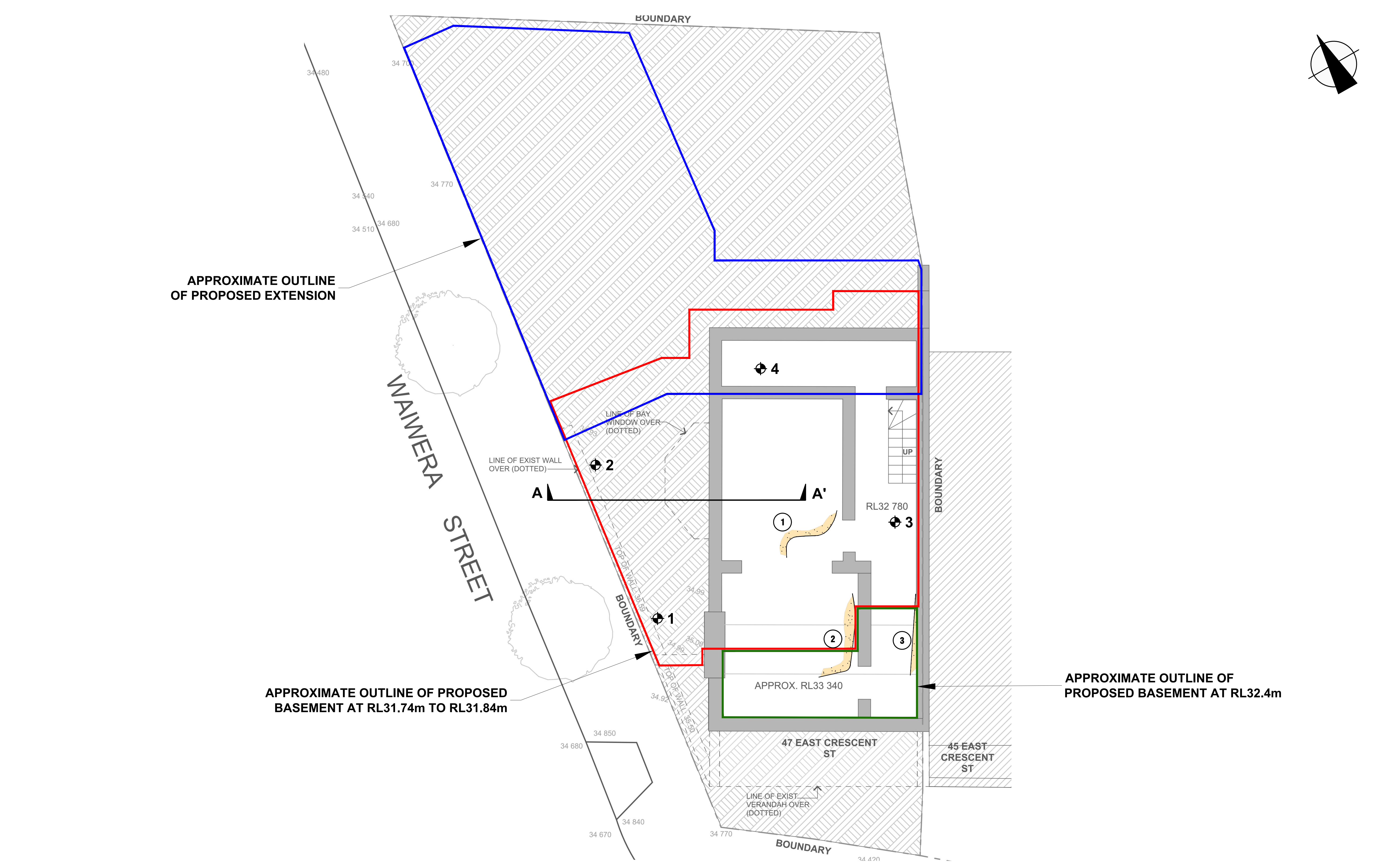
Location: 47 EAST CRESCENT STREET, LAVENDER BAY, NSW

Report No: 37572S

Figure No: 1

JKGeotechnics

PLOT DATE: 23/04/2025 2:27:11 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37572S LAVENDER BAY\CAD\37572S.DWG



LEGEND

- DCP TEST
- SANDSTONE OUTCROP
- (1) NOTE REFERENCE

NOTES:

1. VERY LOW SANDSTONE, ORANGE BROWN, WITH XW BANDS.
2. VERY LOW TO LOW SANDSTONE, ORANGE BROWN & GREY, WITH XW BANDS.
3. VERY LOW TO LOW SANDSTONE, ORANGE BROWN & GREY, WITH XW BANDS.

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title:	**INVESTIGATION LOCATION PLAN**
Location:	47 EAST CRESCENT STREET, LAVENDER BAY, NSW
Report No: 37572S	Figure No: 2

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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.

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> 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:

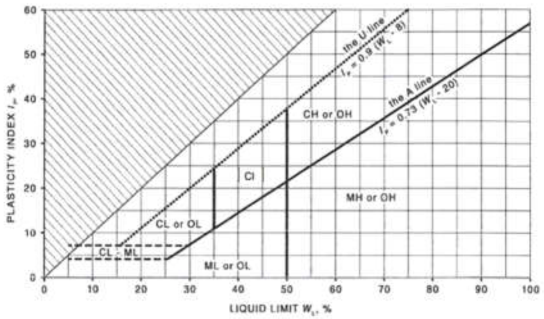
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

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