REPORT TO

SIT FAMILY PTY LTD

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

22C BURRAN AVENUE, MOSMAN, NSW

Date: 15 October 2021
Ref: 34431YJrpt

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
34431YJrpt	Final Report	15 October 2021

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1 INTRODUCTION

This report presents the results of our geotechnical investigation of the site at 22c Burran Avenue, Mosman, NSW. The location of the site is shown in Figure 1. The geotechnical assessment was commissioned by Mr Stanley Xue of HYG on behalf of Sit Family Pty Ltd and was carried out in accordance with our fee proposal (Ref: P54957Y, dated 5 September 2021).

Based on the architectural drawings prepared by PBD Architects (Job No: 1908, Full drawing list shown on Drawing Ref: DA000, Issue A, dated 28 September 2020), we understand that following demolition of the existing residence, stairways and in-ground pool, it is proposed to construct a four-storey house, terrace, swimming pool and spa.

The house will be cut into the hillside with the lower ground floor having a finished floor level of RL19.48m. At the front of the house an automated carparking system or car stacker will be located and will provide access to two carparking spots, one on the lower ground and the other on the ground floor level. The car stacker overrun will have a finished floor level of about RL17.0m. Excavation for the car stacker will result in cuts to maximum depths of about 8.8m, although typical excavation depths will be more in the order of about 3m to 6m.

To the east or rear of the house it is proposed to construct a terrace, pool, spa and deck. The terrace will be located between the house and pool with the pool coping raised above the terrace. The terrace will have a finished floor level of RL19.48m with the base of the pool at (RL19.48m) or slightly lower (RL18.78m) than the terrace finished floor level. The spa and decking around the spa will be located in front of the pool and will have a finished coping/floor level of RL19.9m. The base of the spa will be located slightly below the base of the pool (RL18.7m) and will be constructed within the existing pool excavation. Excavation for the terrace and pool is anticipated to result in cuts to maximum depths of about 2m.

Localised deeper excavations may be required for a lift core, OSD system or buried services.

We were also provided with a geotechnical feasibility 'stability' assessment prepared by GHD (Ref: 2128433-RPT-0002, dated November 2020) and a draft Coastal Engineer's Input/Advice Regarding Cliff Erosion letter prepared by James Carley (Ref: WRL2021082 JTC LR20210924, dated 24 September 2021).

The purpose of our assessment was to satisfy the Statement of Facts and Contentions set out in the Land and Environment Court of NSW Case Number 2021/00169097, which stated that the following further information was required:

1. Four boreholes drilled to a minimum of 2m to 3m below the proposed bulk excavation level to confirm the soil and rock profile. These boreholes were required adjacent to the road and properties to the north and south,
2. As part of the reporting of the results of the investigation, the geotechnical report must include advice on:

    - An excavation plan and methodology,
    - A shoring plan and methodology,
    - Measures required during excavation of the pool to protect the integrity of the cliff face,
    - A risk assessment considering the potential impact of the proposed excavation on adjoining structures, and
    - A geotechnical and vibration monitoring plan.

In addressing the above requirements, we have provided two reports. These are:

- This report which addresses all items accept the geotechnical and vibration monitor plan, and
- A Geotechnical and Vibration Monitoring Plan (Ref: 34431YJlet, dated 15 October 2023).

The purpose of the geotechnical investigation was to obtain subsurface information at the four cored borehole locations. Based on this information we have undertaken a stability assessment, which considers the risk to both life and property to the properties to both the north, south and west and provided comments and recommendations on an excavation plan and methodology, a shoring plan and methodology, footings, slabs on grade and aggression.

2 STABILITY ASSESSMENT AND INVESTIGATION PROCEDURE

2.1 Existing Stability Assessment by GHD

GHD has carried out a stability assessment of the site. The outcome of this assessment was that the proposed residential development was considered to be feasible from a geotechnical perspective and posed an acceptable risk to both life and property in accordance with AGS (2007). A number of recommendations were detailed in this report and were aimed at managing the risks associated with the proposed development.

2.2 Stability Assessment (On Adjoining Structures)

As GHD have completed a stability assessment for the site and, in particular the existing cliff line, we have not undertaken a stability assessment of the site as a whole. Our stability assessment has been limited to the risks posed by the proposed development to the adjoining properties to the north and south and Burran Avenue itself.

The stability assessment was carried out by our Associate, Mr Jarett Mones on 24 September 2021, and is based on a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

A summary of our observations and slope instability hazards for the adjoining structures are presented in Sections 3.1 and 4.1.1 below. Our specific recommendations regarding the proposed development are discussed in Section 5 following our geotechnical stability assessment of adjoining structures.

The attached Figure 3 presents a plan showing notable geotechnical site features and geotechnical hazards. We were not able to access the area between the site and the cliff-line due to safety concerns so our geotechnical mapping was carried out from safe vantage points on the site and from the base of the cliff-line. Figure 3 is based on the land survey plan prepared by Bee & Lethbridge Pty Ltd (Bee & Lethbridge, Ref: Drawing No. 21188-02, Revision 02, dated 23 July 2019). In addition to the land survey an aerial survey was carried out by Diodrone Pty Ltd (Ref: 19AU030, Revision 1.1, dated 1 August 2019). This included LiDAR data of the cliff-line and cross sections through the cliff-line. Additional features on Figure 3 have been measured by hand held clinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figures 4 to 7 present Sections A-A' to D-D', which are typical cross-sections through the site and are based on the survey data augmented by our mapping observations. Figures 8 and 9 present Section E-E' and F-F' which includes additional sections through the cliff-line at the southern and northern portions of the site. Sections A-A' and B-B' also includes the assessed potential landslide hazards for adjoining structures. Figure 10 defines the geotechnical mapping symbols adopted and used in Figure 3.

2.3 Geotechnical Investigation

Prior to the commencement of the fieldwork, we carried out the following:

- Review of the proposed development and existing site constraints to determine appropriate investigation locations. These were discussed with the property owner while Mr Mones was on site on 24 September 2021 and proposed locations were forwarded to Liang Zhang of HYG for comment,
- Liaison with the Client in regards to access for the drilling works, and
- Completion of a dial before you dig buried services search and an on-site services search using electromagnetic induction measures completed by a buried services subcontractor.

Our geotechnical investigation was carried out using portable hand operated equipment and comprised the following:

- Four boreholes, which were initially advanced by diatube core drilling through pavements (BH1 and BH2) and then using hand auger techniques to refusal depths ranging between 0.16m and 0.36m. These boreholes were then extended using portable rotary diamond rock coring techniques (TT56) to depths of between 8.73m and 12m. BH1 was advanced from a depth of 1.37m to 1.9m using wash boring techniques.
- Four DCP tests were completed adjacent to the boreholes to refusal depths of between 0.15m and 0.35m.

- Groundwater observations were made in the boreholes, during and on completion of drilling. We note that water is introduced into the borehole during core drilling and therefore the water levels measured on completion of coring are unlikely to reflect actual groundwater levels.

The purpose of the boreholes was to identify the soils present while the DCP tests were used to probe the depth to bedrock and interpret the degree of compaction of the fill. The strength of the bedrock was assessed by examination of the recovered rock core and subsequent correlation with Point Load Strength Index ($I_{S(50)}$) testing. The results of the Point Load Strength Index tests are presented in the attached Table C and on the cored borehole logs. The Unconfined Compressive Strength's (UCS's) were estimated from the Point Load Strength Index test results and are also summarised in Table C. Photographs of the recovered core are presented at the rear of this report with the borehole logs.

The investigation location plan is included as Figure 2. Due to access constraints, investigation locations were limited to areas outside the house, swimming pool and deck and other structures and buried services. The locations were set out by taped measurements from apparent surface features, as shown on the land survey plan prepared by Bee & Lethbridge and referenced above. The approximate surface levels, as shown on the borehole logs and DCP test results sheet, were estimated by interpolation between spot levels shown on the Bee & Lethbridge survey plan and are, therefore, only approximate. The datum for the levels is Australian Height Datum (AHD), as noted on the survey drawing.

Selected samples were returned to Envirolab Services Pty Ltd (Envirolab), a NATA accredited laboratory, for pH, sulphate content, chloride content and resistivity testing. These results are presented in the attached Envirolab Certificate of Analysis No. 279628.

The investigation was carried out in the full-time presence of our Geotechnical Engineer, Mr Sami Azzi, who set out the investigation locations, nominated the sampling and testing and prepared logs of the strata encountered. The borehole logs (including core photographs) and DCP test results sheet, are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques adopted and their limitations, and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site description should be read in conjunction with Figure 3, which includes a chainage system for ease of reference to the details below.

For the purpose of this site description the site is considered to be just that portion of the property that will be developed, and not the whole property. The site is located above a cliff-line/sloping ground that is approximately 15m to 20m high and drops down to Middle Harbour. The site itself is currently developed with a one and two-storey rendered house and garage that occupies much of the property and steps down from about RL25.5m at the front (western side) to about between RL19.9m and RL21.4m at the back (eastern side). The house appeared to be in good condition, however we observed an about 2mm wide crack in the render on the southern side of the house. East of the house and near the cliff-line is a tiled terrace area that has a level of about RL21.4m, which steps down to a pool and spa with tiled surrounds at about RL19.9m. Sandstone bedrock is outcropping below the tiled terrace area. Some filling was observed between the bedrock and the terrace. There is a buried sewer service to the rear of the house that runs below the terrace and the in-ground swimming pool and is shown on Figures 2 and 3. There are tiled and concrete pavements and low height sandstone masonry retaining walls supporting garden areas around the house.

The house has been cut into the hillside. As a consequence of this there is a sandstone cutting runs across the site in a north-west to south-east direction between CH. 12m and CH.15m and ranges in height from about 2m to 3m. The cut also extends to the east and rear of the site along the northern and southern boundaries between CH. 12m and CH.20m and CH. 15m and CH. 18m, respectively. The materials exposed in the face of the sandstone cutting were assessed to be slightly weathered and of medium strength. We observed 4 bedding partings, generally horizontal, with exception to one which was between about 10° and 20°. A loose boulder was observed above the crest of the cut at Ch. 15m.

There is a 0.6m to 0.9m wide plinth of rock that ranges in height from about 1m to 2m high, runs for a length of about 3m between Ch. 15m and Ch.18m along the southern site boundary and is wholly located in the adjoining property. Photographs of the sandstone cutting exposed are included as Plates 1a to 1d.

To the north of the site is a one to two-storey masonry house that, at its closest, extends to within about 1m of the common boundary. To the south the site is bound by a two to three-storey house that extends to within about 1.5m from the common site boundary. Both houses appeared in good condition when viewed from the site. Boundary walls constructed from sandstone block and rendered sandstone block and brick ran along both the northern and southern boundaries. The adjoining properties had similar landforms to that of the site with levels across the boundary similar, with the following exceptions:

- Northern boundary,
    - CH. 12.5m to CH.21.5m: As a result of the existing house being cut into the hillside, the site is lower than the adjoining property to the north. A sandstone cut has been formed along this boundary that has a maximum height of 2m and reduces in height from the west to east.
    - CH. 12.5m to CH. 15.5m: Above the cut a 0.6m to 1.1m high brick and concrete retaining wall has been constructed and supports the property to the north. The concrete wall appears to be propped by the suspended stairway and appears in good condition.
    - CH. 21.5m to CH. 28m: A 1.1m to 1.2m high sandstone masonry block retaining wall supports the neighbor’s property. The wall appeared to be in good condition.
- Southern boundary,
    - CH. 11m to CH. 16m: A 0.8m high sandstone masonry block wall supports the site. The wall appeared to be in good condition.
    - CH. 18m to CH. 21m: A 1.3m high brick retaining wall supports the property to the south. The brick wall appears to be propped by the suspended stairway. The wall appeared to be in good condition.

- From CH. 26m to CH. 29m a 0.5m high sandstone masonry block landscaped retaining wall supports the site. This wall had a 50mm wide stepped crack.

East of the site just beyond the existing pool area ground levels drop down through a number of 1m to 2m high dry stacked sandstone block retaining walls. At the toe of the walls ground surfaces then slope down at about 20° to the cliff line, which is generally about 15m high. The site is typically setback about 10m to 15m from the cliff line with the exception of two localised areas at the northern and southern ends of the site where jointing within the rock mass has resulted in preferential weathering and at these locations the site locally extends to within about 1.5m of the cliff-line. The central portion of the cliff line has been undercut forming a 3m to 4m overhang. At the base of the cliff-line is a low strength siltstone band. Where exposed this band has a height of about 1m and has been preferentially weathered and eroded to a depth of about 1m to 1.3m between the stronger sandstone units above and below (Ref: Plate 2). Jointing within the cliff line is sub-vertical and is prominent at its northern and southern ends.

To the west the site is bounded by Burran Avenue, which is a concrete paved split-level road. Midway across the road where levels step up to the northern carriageway, sandstone bedrock is exposed in the base of the sandstone block retaining wall that supports the higher carriageway.

Plate 1a - Block of Rock (Boulder size) at Crest of Sandstone Cutting (Southern Boundary)

Plate 1b – Plinth of Rock along Sandstone Cutting (Southern Boundary)

Plate 1c –Sandstone Cutting (Central)

Plate 1d –Sandstone Cutting and Retaining Wall Above (Northern Boundary)

Plate 2: Siltstone Band at Base of Cliff-Line

3.2 Geology and Subsurface Conditions

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Hawkesbury Sandstone of the Wianamatta Group. Hawkesbury Sandstone comprises medium to coarse grained quartz sandstone with very minor shale and laminite lenses.

The investigation disclosed subsurface conditions generally comprising shallow granular fill overlying weathered sandstone bedrock. This shallow sandstone bedrock is also seen in the cut that has been formed into the hillside to allow the construction of the existing house. Some of the more pertinent details of the strata encountered are described below. For further details of the conditions encountered at a particular borehole, reference should be made to the attached borehole logs.

Pavements and Fill

Concrete pavements were 105mm and 130mm thick in BH1 and BH2, respectively. In BH1, there was a 105mm thick concrete layer at a depth of 1.1m and in BH4, there was a 120mm thick concrete layer at a depth of 0.47m. In BH1 the concrete layer at depth may be from a former pavement, whereas at BH4, the concrete may be associated with the landscaped retaining wall.

Fill was encountered or inferred to extend to depths of 1.31m (BH1), 1.48m (BH2), 0.53m (BH3) and 0.47m (BH4, note that at this depth there was concrete that then extended to a depth of 0.59m). The ‘No Core’ zones above the proved sandstone unit have been inferred to represent the washing away of the fill materials. The fill encountered in the boreholes comprised sand, silty sand and gravel. Inclusions of gravel (possibly cobbles), roots and root fibres were noted within the fill.

Sandstone Bedrock

Weathered sandstone bedrock was encountered or inferred below the fill/concrete at depths of 1.31m (BH1), 1.48m (BH2), 0.53m (BH3) and 0.59m (BH4).

The sandstone bedrock in BH1 was of poor quality to a depth of about 6.1m and typically comprised highly weathered very low to low strength bedrock, with an extremely weathered sandstone (with soil strength) band. Three ‘No Core’ zones (0.13m, 0.7m and 1.01m thick) were logged within this poor quality bedrock. ‘No Core’ zones are inferred to represent poorer quality (extremely weathered) sandstone bedrock or soil bands that have been washed out during the coring process. Slightly weathered and low to medium strength sandstone was encountered at a depth of 6.1m and then improved to medium strength at a depth of 6.8m.

In the remaining boreholes, BH2 to BH4, the sandstone bedrock was generally of good quality when first encountered and was of medium strength. A number of ‘No Core’ zones were encountered within these boreholes and included 0.01m and 0.03m zones in BH2, a 0.12m zone in BH3 and a 0.59m zone in BH4. These are similarly inferred to represent weaker (extremely weathered/clay) seams within the rock mass.

With the exception of BH1, defects within the rock mass were relatively minor and typically comprised bedding partings and extremely weathered seams. Joints, dipping at between 30° and 90°, extremely weathered seams, clay seams and sub-horizontal bedding partings, were observed within the bedrock.

Groundwater

Groundwater was not encountered during or on completion of augering. Groundwater was measured on completion of coring at depths of about 2.75m (BH1), 2.5m (BH2), 2.8m (BH3) and 3.2m (BH4). We note that water is introduced into the borehole during core drilling and therefore the water levels after coring are likely to be artificially high and not representative of groundwater levels across the site. As the site is located in steeply sloping terrain that drops down to Middle Harbour through an approximately 15m high cliff line, we do not anticipate that a groundwater table will be present within the proposed depth of excavation. Seepage was not observed from the cliff-face, which confirms the above assessment.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores from each borehole correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s\,(50)}$) tests ranged from 0.04MPa to 0.9MPa. 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 from 1MPa to 18MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below. Envirolab Certificate of Analysis No. 279628 is attached and presents the results of these tests. Reference should be made to Section 5.6 for the potential impact of the soil aggression on concrete and steel structures in contact with the ground.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH1	0.15-0.35	Fill: Sand	9.0	<10	<10	25,000
BH1	2.4-2.43	Low Strength Sandstone (Bedrock)	4.9	<10	<10	66,000
BH3	1.83-1.85	Medium Strength Sandstone (Bedrock)	5.3	<10	<10	71,000
BH4	0.1-0.2	Fill: Silty Sand	6.7	<10	10	35,000

4 LANDSLIDE RISK ASSESSMENT (ON ADJOINING STRUCTURES)

4.4 Landslide Risk Assessment Criteria

The assessment of slope stability at the site on adjoining structures has been made using the guidelines presented in the Landslide Risk Management Concepts and Guidelines prepared by the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management[1]. In this regard an acceptable risk for loss of life of $1x10^{-6}$ has been adopted for natural slopes for the person most at risk. For loss to property the acceptable risk should be determined by the owner, provided loss to property only affects the owners' property and does not impact on the property of others. As a guide the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management adopts an acceptable risk to property posed by existing slopes as "low". Where risks posed by slope instability are considered unacceptable, remedial measures should be adopted to reduce the risk posed to an acceptable level.

The assessment has been made on a semi-quantitative basis with quantitative values assigned to qualitative assessments. The qualitative assessments are based on judgements made in the field by the geotechnical engineer and in this regard are subjective and formed in part by the engineers' previous experiences. The range of annual probabilities assigned to the likelihood of events occurring, the recommended vulnerability values and the qualitative risk analysis matrix are presented in Appendix A.

4.4.1 Hazards

Reference should be made to the attached Figures 3, 4 and 5, which indicate the approximate location of the potential hazards posed by this site to the adjoining properties. These hazards are described below:

- **Hazard A** – Stability of sandstone bedrock cut,
- **Hazard B** – Stability of existing retaining walls greater than 0.5m high,
- **Hazard C** – Stability of loose block of rock in plenum, and,

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[1] Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

- **Hazard D** – Stability of new engineered walls/permanent sandstone cuts.

4.4.2 Assessment of Risk to Property and Risk to Life Posed by this Site to Adjoining Properties

The attached Tables A and B detail the risks to property and the risk to life for the person most at risk for the site. Based on this the site poses an “acceptable” risk to life and property to the adjoining properties.

4.5 Assessment of Risk Posed by Proposed Development to Adjoining Properties

The design project life for this project considering adjoining structures has been taken as 50 years. This provides the context within which the geotechnical risk assessment should be made. The required 50 years baseline broadly reflects the expectations of the community for the anticipated life of a residential structure and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 50 year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, the geotechnical engineer does not warrant the development for a 50 year period, rather provides a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.

In our assessment we have made the following assumptions:

- The proposed development works will be carried out in accordance with our comments and recommendations in Section 5.
- All new retaining walls will be engineered retaining walls designed in accordance with our comments and recommendations in Section 5 and in accordance with the relevant Australian Standards/design codes.
- That no activities on surrounding land which may affect the risk on the subject site would be carried out.
- That all Council’s buried services are, and will be regularly maintained to remain, in good condition. The existing sewer that crosses below the existing tiled terrace and pool may need to be relocated.

Provided the assumptions above are correct and the recommendations below are followed, we consider that our risk analysis has shown that the risk posed by the proposed development to the adjoining properties can achieve the ‘Acceptable Risk Management’ criteria.

5 COMMENTS AND RECOMMENDATIONS

5.1 Excavation Plan and Methodology

5.1.1 Dilapidation Surveys

Dilapidation surveys of adjoining buildings/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 for damage which existed prior to excavation or demolition commencing. Consequently, prior to the commencement of works we recommend that detailed dilapidation reports be compiled on buildings that fall within the zone of influence of the excavation. The zone of influence is considered to be a distance back that extends 2H back from the crest of the excavation where H is the retained height. In this regard we recommend that dilapidation surveys be completed on the adjoining properties to both the north and south and Burran Avenue.

The dilapidation surveys should comprise detailed inspections of the adjoining properties, both externally and internally, with all defects rigorously described, i.e. defect location, defect type, crack width, crack length, etc. The respective owners should be asked to confirm in writing that the 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.

5.1.2 Excavation and Groundwater

A detailed demolition, excavation and retention methodology should be approved prior to commencement of the site works.

Excavation recommendations provided below should be complemented by reference to the latest Code of Practice ‘Excavation Work’, prepared by Safe Work Australia.

The comments and recommendations provided in this section relate to the house and not the excavation for the pool and spa. Additional recommendations with regards to excavation of the pool and spa adjacent to the cliff-line are provided below in Section 5.2.2.

Excavation for the proposed development is anticipated to extend to within 1m of the northern boundary, 2.5m of the southern boundary and 0.5m of the western boundary. Prior to the commencement of construction, as-built drawings should be sought for the existing retaining walls that are present on the boundary, particularly along the northern boundary between CH. 21.5m and CH. 28m, and on the size, location and depth of the sewer.

In the early stages of construction, we recommend that any loosened blocks of rock at the crest of the existing cut near the southern boundary be removed and that test pits be excavated along the boundaries to confirm the footing details of the existing boundary walls and retaining walls. Where these are not founded on sandstone bedrock of at least low strength, underpins may be required. The structural engineer should detail the underpinning methodology at that stage and provide detail for lateral support, if required. Prior to demolishing existing stairways that appear to be propping existing boundary retaining walls, the structural engineer must assess whether these walls are being propped and, if so, the structural engineer must detail both temporary and permanent support for these walls prior to the demolition of the house.

To achieve bulk excavation level excavation to a maximum depth of about 8.8m will be required. This will predominantly require the removal of sandstone bedrock ranging up to medium strength although a thin layer of granular fill is also anticipated to be encountered overlying the sandstone bedrock. Where sandstone bedrock is of low strength or less, we anticipate that excavation can be completed using medium sized excavators (say 15 to 20 tonnes) with buckets with "tiger teeth" attached. Where the sandstone bedrock is of greater than low strength, "hard rock" excavation techniques will be required.

"Hard rock" excavation techniques may consist of percussive or non-percussive techniques. Percussive techniques comprise the use of rock hammers while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting etc. Where percussive excavation techniques are adopted, there is the risk that transmitted vibrations may damage nearby movement sensitive structures such as the adjoining buildings structures. Consequently, we recommend that considerable caution be exercised and that the excavation procedures and the dilapidation reports be carefully reviewed prior to excavation commencing, so that appropriate equipment is used. Consequently, we recommend that the following measures be taken:

- Saw cuts should be formed along all proposed cutlines,
- During percussive excavation continuous quantitative vibration monitoring must be completed and will provide feedback to the excavation contractor on the suitability of the excavation equipment and techniques adopted. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation. Where non-percussive excavation techniques are adopted no vibration monitoring is required,
- Percussive excavation should be completed so that the excavation is progressively enlarged by breaking small wedges out of the face,
- Rock hammers should only be operated in short bursts to prevent amplification of vibrations.
- Where transmitted vibrations exceed prescribed limits, excavation techniques must be altered to reduce transmitted vibrations to within acceptable limits. This may mean that the size of percussive equipment used may need to be reduced, or non-percussive techniques adopted. Whether reducing the size of the percussive equipment is effective in controlling transmitted vibrations must be confirmed by quantitative vibration monitoring.

The prescribed vibration limits that should be adopted on this site where percussive excavation techniques are adopted are set out in the Vibration Emission Design Goals attached to the rear of this report. We have also prepared a Geotechnical and Vibration Monitoring Plan (Ref: 34431YJlet, dated 15 October 2021), which also sets out the vibration limits that we recommend be adopted for this site. Additional recommendations for excavation adjacent to the pool and spa are provided in Section 5.2.2.

Where the excavated material is disposed of offsite a waste classification will be required.

We do not anticipate a groundwater table will be encountered. Some groundwater seepage is expected to occur at the interface between the soils and bedrock and through defects within the rock mass itself such as through bedding partings, joints, etc. Higher groundwater seepage flows along the soil/rock interface are likely to occur during and immediately following periods of wet weather. Groundwater seepage can be satisfactorily controlled by using sumps and pump or gravity drainage measures. We recommend that the hydraulic engineer inspect the site during and on completion of excavation to confirm that their detailed drainage is suitable for this site.

5.1.3 Swimming Pool and Spa Excavation

As discussed, the proposed pool and spa is generally located no closer than about 10m to 12m from the eastern boundary (Ref: Figure 6). However, at its northern and southern ends it extends to within about 1.5m (Ref: Figures 3, 8 and 9) of the cliff line. The spa will be wholly located within the existing pool excavation and only limited (less than 0.5m) additional excavation is expected to be required to achieve bulk excavation in this location. The proposed new pool at its southern end will be wholly located within the existing pool excavation and consequently, over the southern half of the pool no excavation adjacent to the cliff line will be required. At its northern end, excavation to maximum depths of about 2m are required but are anticipated to be predominantly through soil.

Only very limited excavation through sandstone bedrock is anticipated to be required for either the pool or spa. Consequently, it is our opinion that the proposed pool and spa excavation can be completed without adversely affecting the sandstone bedrock exposed in the existing cliff line. While this is our opinion, we recommend that the following additional precautions be adopted:

- The excavations should be carried out by experienced professionals with safe work methods in place.
- A meeting should be held between the builder, sub-contractor and geotechnical engineer prior to any excavations to review the proposed construction methodology.
- The excavation cutlines must be completed using a diamond tipped rock saw.
- All rock excavation must be completed using non-percussive excavation techniques.
- Where the dry stacked sandstone block retaining walls are removed, measures must be put in place so that they do not roll downslope and over the cliff line.
- A geotechnical engineer is to be present full time during these excavations.

5.2 Shoring Plan and Methodology

Based on the results of the boreholes and our observation of the quality of the bedrock exposed in the rock cutting and cliff line, it appears that sandstone bedrock is present at shallow depth across the site and is of good strength and relatively free from adverse defects. Notwithstanding this, BH1 encountered poor quality bedrock to a depth of about 6.1m (RL19.4m), which appears at odds with the rest of the available information (refer to Figure 6). Consequently, it appears that, where present, this poorer quality bedrock encountered in BH1 may be quite localised.

Where the depth to bedrock is shallow and it is of good strength, it is likely that low height gravity walls can be constructed to support the soil and sandstone bedrock of low strength or poorer and that the underlying sandstone bedrock of greater than low strength may be cut vertically and left unsupported, as is the case with the cut that has been formed to allow the construction of the existing house. However, BH1 suggests that poor quality bedrock extends to significant depth at the front of the property. If this is the case a contiguous pile wall will be required to support this portion the site. Consequently, once the existing house has been demolished we recommend that further investigation be completed to confirm the most appropriate shoring solution for the site. In this regard we recommend that:

- Additional cored boreholes be drilled between the Burran Avenue and the existing cut face located at about CH. 15m to confirm the depth and extent of the poorer quality sandstone bedrock encountered in BH1, and
- Test pits be excavated along the northern, southern, and western site boundaries to confirm the depth to bedrock and whether it is sufficiently shallow to allow gravity walls to be constructed to retain the soil and poorer quality sandstone bedrock.

Where poor quality bedrock extends to depth, a propped or anchored contiguous pile wall will be required and must be founded below bulk excavation level. This will mean that piles will need to be installed through at least medium strong sandstone bedrock and consequently, we recommend that piling contractors be contacted to assess the suitability of their equipment to drill the required piles. Due to the presence of sandy fill, the small gaps between piles should progressively be dry packed as the excavation progresses. A soldier pile wall should not be attempted as the sands and gravel will likely flow between the piles.

Where good quality sandstone bedrock is present at shallow depth, the soil and poor quality bedrock may be supported by low height mass concrete walls. As a rough guide we anticipate these walls would need to be about 0.5m to 0.8m wide to support heights of 1m to 1.5m, respectively, although this will need to be confirmed by the structural engineer. Vertical rock dowels setback and inclined away from the excavation may be required to provide the required lateral support. As the mass concrete walls will be poured in potentially unsupported trenches, care will need to be taken that their construction does not potentially destabilise boundary retaining walls or other structures. In this regard, construction of these walls will need to be carefully staged and controlled and the founding conditions of adjoining boundary walls/structures determined prior to the commencement of construction.

Good quality sandstone bedrock of greater than low strength may be cut vertically and left unsupported provided it is free from adverse defects. In this regard we recommend that every 1.5m of vertical unsupported cut be inspected by a geotechnical engineer so that where adverse defects are present they may be identified and remedial measures initiated. For the proposed excavation it is likely that the proposed excavation will expose some adverse defects. Where adverse defects, such as inclined joints are present, they will need to be stabilised in the short term by the installation of shotcrete, mesh and bolts/props and in the long term by the built structure. Clay seams or bands of poor quality bedrock may also require 'dental' treatment' where the cut faces are to be left permanently exposed. Provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures. In the instance where temporary bolts or anchors will not be allowed to be installed across the site boundaries props can be substituted.

Where space permits temporary batters may be adopted. Temporary batters in soils including extremely weathered sandstone bedrock may be formed with an overall slope no steeper than 1 Vertical (V) to 1.75 Horizontal (H) and in very low to low strength bedrock may be formed with an overall slope no steeper than 1V:1H. These temporary batters assume that all surcharge loads are kept well clear from the crest of the batter, at least a distance of 2H from the crest, where H is the height of the soil and bedrock of low strength or less in metres. Where temporary batters are proposed, the geotechnical engineer should review the proposed extent and comment on the feasibility of this approach.

For the design of the contiguous pile retaining walls or mass concrete walls the following parameters may be adopted:

- Adopt a triangular lateral pressure distribution for the soils and bedrock of up to low strength. Where movement sensitive structures are located within the zone of influence of the proposed excavations or where the retaining wall will be anchored or propped and retain materials to a height of no greater 3m a coefficient of lateral earth pressure, K, of 0.6 should be adopted for the soils and bedrock of up to low strength. Where movement sensitive structures are not present within the zone of influence of the excavation, an active earth pressure coefficient, k_a, of 0.35 should be adopted for the soils and bedrock of up to low strength.
- Where soil and poor quality bedrock extends to depths in excess of 3m, a rectangular earth pressure distribution should be adopted. Where movement sensitive structures are located with the zone of influence of the excavation a pressure of 8H kPa should be adopted where H is the retained height in meters. Where movement sensitive structures are not located within the zone of influence of the excavation a pressure of 6H kPa may be adopted.
- Where a contiguous pile wall is adopted and extends to below bulk excavation level, a uniform pressure of 10kPa should be adopted for the sandstone bedrock of low strength or greater to allow for the potential presence of some adverse jointing.
- The above coefficients and pressure assume a horizontal backfill surface and any inclined backfill must be taken as a surcharge load.
- A bulk unit weight of 20kN/m^3 should be adopted for the retained soils and bedrock of up to very low to low strength. A bulk unit weight of 24kN/m^3 should be adopted for sandstone bedrock of greater than very low to low strength.

- Any surcharge affecting the walls should be allowed in the design.
- The retaining walls should be designed for appropriate hydrostatic pressures. If they are designed as drained they must be provided with complete and permanent drainage. The drainage could comprise vertical strip drains at 1m to 1.5m spacings formed along the gaps between piles or by installing weepholes through the piles. The drains should incorporate a non-woven geotextile fabric, e.g. Bidim A34, to act as a filter against subsoil erosion.
- Toe restraint of the wall may be achieved by keying the footing into bedrock below bulk excavation level. A value of 500kPa may be adopted where the wall is below bulk excavation level, including localised nearby excavations that may be required for the lower level for the automated car system, lift core, OSD system, buried services, etc. and is socketed into sandstone bedrock of at least low to medium strength. When calculating the required depth of embedment needed for lateral restraint the first 0.5m of the socket below bulk excavation should be ignored. The suitability of the sandstone bedrock must be confirmed by a geotechnical engineer during installation of piles.
- Anchors or bolts may be designed based on an allowable bond strength of 100kPa in low strength sandstone bedrock and 250kPa in medium strong sandstone bedrock. Temporary anchors used for lateral support should be bonded beyond a line drawn up at 45° from the bulk excavation level. All 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.
- Where temporary anchors extend below adjoining properties permission from the respective property owners/Council must be obtained before installation.
- Long term support should be provided by the built structure. Once constructed temporary anchors could then be destressed.

We have found that detailed retaining wall designs using geotechnical software such as WALLAP and PLAXIS can produce more economical wall designs than by using the apparent earth pressure recommendations above. However, WALLAP, does not have input parameters to review scenarios where there are adverse joints, large wedge failures, etc and therefore specific checks need to be carried to review these. Otherwise, these can be reviewed using PLAXIS. We consider that the following preliminary geotechnical design parameters could be adopted for shoring wall design using such software packages.

Preliminary Shoring Wall Design Parameters

Material Type	Unit Weight (above GWL) (kN/m^3)	Effective Friction Angle (degrees)	Effective cohesion (kPa)	Elastic Modulus (MPa)
Fill: Sand and Gravel	18	27	0	8
Extremely Weathered Sandstone and Very Low to Low Strength Sandstone (1)	22	32	10	75
Sandstone Bedrock of Low to Medium Strength or Better (1)	24	30	50	400

Notes on table above:

(1) The presence of potential adverse defects must be considered and included in the design of retaining walls.

The shoring design should be forwarded to the geotechnical engineer for review prior to the commencement of construction.

5.3 Footings

Due to the presence of uncontrolled granular fill extending to a depth of greater than 0.8m, 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.

We recommend that all footings be uniformly founded on the underlying sandstone bedrock. With the exception to the eastern portion of the site, excavation is anticipated uniformly to expose sandstone bedrock at bulk excavation level and shallow pad/strip footings would likely be adequate for the building loads. At the eastern portion of the site where fill soils will be exposed or where there is an existing sewer that will remain, piles may be required. Generally, where these soils are greater than a depth of about 1m piles are considered to be more suitable than pads/strips. Otherwise, deeper pad footings could be constructed. Footings near the sewer line will likely need to be founded below the sewer invert level and maintain minimum set-backs as required by Sydney Water.

Pad and strip footings founded in sandstone bedrock of at least low to medium strength may be designed for an allowable bearing pressure (ABP) of 1500kPa. Similarly, piles socketed a minimum of 0.5m into sandstone bedrock of at least low to medium strength may also be designed for an ABP of 1500kPa. For that part of the pile that extends below this nominal socket a shaft adhesion of 150kPa and 75kPa may be adopted for compressive and tensile loads, respectively. This assumes that the rock socket is suitably roughened. All footings should be founded below a line drawn upwards up at 1V:1H from the base of adjoining excavations, such as the automated car system, lift pit, OSD tank, services, etc. Care will be required where footings are located close the cliff line and, in this instance, further advice on the proposed footing locations will be provided once bulk excavation is completed. This may require footings that are located in close proximity to the cliff line to either be moved or deepened.

Prior to pouring concrete all footings must be free from all loose and softened materials. All footing excavations must be inspected by a geotechnical engineer to confirm that the design ABP’s have been achieved prior to pouring concrete.

5.4 Slabs on Grade

We recommend that where fill or soil is exposed at subgrade level, such as at the eastern and western ends of the site, that the slabs be fully suspended and supported on the underlying sandstone. All on-grade slabs for the building that are poured directly over sandstone should be provided with an underfloor drainage blanket that will act both as a debonding and drainage layer. The underfloor drainage should comprise a strong, durable, single sized washed aggregate, such as ‘blue metal’ gravel. The underfloor drainage should collect groundwater seepage and direct it by gravity flow to the stormwater system.

5.5 Aggressivity

The above results indicate that the materials 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 and Installation’’. Any concrete exposed to these conditions (e.g. footings, shoring piles, etc.) should have a characteristic concrete strength and cover as recommended in Table 6.4.3.

The above results indicate that the materials 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) AS2159-2009. Any steel exposed to these conditions should have a uniform corrosion allowance as recommended in Table 6.5.3.

5.6 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:

- Dilapidation reports on the properties to the north, south and west of the site.
- Approval of a detailed demolition, excavation and retention methodology prior to commencement of the site works.
- Site meeting between the builder, architect, hydraulic, structural engineer and geotechnical engineer prior to any site works to review the demolition, excavation and retention methodology.
- Following the demolition of the existing house, drilling of additional cored boreholes over the front of the site to confirm the extent of the poor quality bedrock identified in BH1 and the excavation of test pits along the northern and southern boundaries to both determine the depth to bedrock and details of the footings of the walls located on the boundaries.
- Site meeting between the builder, subcontractor and geotechnical engineer prior to any site works to review the excavation methodology near the cliff-line.
- Geotechnical engineer to review geotechnical elements of structural drawings including the shoring design.
- If temporary batters are proposed the geotechnical engineer is to review and comment whether they are considered appropriate.
- A geotechnical is to be present during excavation works along the proposed pool and spa.
- Monitoring of transmitted vibrations where percussive excavation techniques are used.
- Regular inspection of all unsupported vertical cuts formed through bedrock by a geotechnical engineer at depth intervals of no more than 1.5m to check for the presence of adverse defects and initiate remedial measures where required.
- Inspection of hydraulic conditions during and on completion of bulk excavation.
- Inspection of all footing excavations and pile drilling to confirm that bedrock of adequate quality for the design allowable bearing pressures has been encountered.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are 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.

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.

TABLE A

SUMMARY OF RISK ASSESSMENT TO ADJOINING PROPERTIES

POTENTIAL LANDSLIDE HAZARD	A Stability of sandstone bedrock cut	B Stability of existing retaining walls greater than 0.5m high	C Stability of loose block of rock in plenum	C Stability of new engineered walls/permanent cuts
Assessed Likelihood	Barely Credible	Barely Credible	Possible	Barely Credible
Assessed Consequence	Insignificant	Insignificant	Insignificant	Insignificant
Risk	Very Low	Very Low	Very Low	Very Low

* The above consequences are based on an assumed property value of $14.5M (Source: **www.realestate.com.au**, last sold on 8 Nov 2018)

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE ON ADJOINING PROPERTIES

POTENTIAL LANDSLIDE HAZARD	**A** **Stability of existing sandstone bedrock cut**	**B** **Stability of existing retaining walls greater than 0.5m high**	**C** **Stability of loose block of rock in plenum**	**D** **Stability of new engineered walls/permanent cuts**
Assessed Likelihood	Barely Credible	Barely Credible	Possible	Barely Credible
Indicative Annual Probability	10^{-6}	10^{-6}	10^{-3}	10^{-6}
Duration of Use of area Affected (Temporal Probability)	(i) Above cut 8 hours/day 3.33×10^{-1} (ii) Below cut 8 hours/day 3.33×10^{-1}	(i) Above wall 1 minute/day 6.94×10^{-4} (ii) Below wall 1 minute/day 6.94×10^{-4}	Below 1 minute/month 2.28×10^{-5}	(i) Above wall/cut 8 hours/day 3.33×10^{-1} (ii) Below wall/cut 8 hours/day 3.33×10^{-1}
Probability of not Evacuating Area Affected	(i) 0.9 (ii) 1.0	(i) 0.9 (ii) 1.0	1.0	(i) 0.9 (ii) 1.0
Spatial Probability	6m length of failure over ˜25m full length, 6/25 = 0.24	4m length of failure over ˜24m full length, 4/24 = 0.17	1.0	6m length of failure over ˜67m combined lengths, 6/67 = 0.090
Vulnerability to Life if Failure Occurs Whilst Person Present	(i) 0.1 (fall from above) (ii) 1.0 (likely to be buried)	(i) 0.1 (low fall from above) (ii) 1.0 (buried, fence above wall)	1.0	(i) 0.8 (fall from above) (ii) 1.0 (likely to be buried/hit)
Risk for Person most at Risk	(i) 7.1×10^{-9} (ii) 7.99×10^{-8}	(i) 1.06×10^{-11} (ii) 1.18×10^{-10}	2.28×10^{-8}	(i) 2.16×10^{-8} (ii) 3.0×10^{-8}
Combined total Risk	2.2×10^{-7}			

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: sit family Pty Ltd
Ref No: 34431YJ

Project: Proposed Residential Development
Report: A

Location: 22C Burran Avenue, MOSMAN, NSW
Report Date: 30/09/21

Page 1 of 3

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	2.50 - 2.54	0.2	4	A
	2.75 - 2.79	0.2	4	A
	3.80 - 3.84	0.04	1	A
	5.55 - 5.59	0.07	1	A
	6.10 - 6.14	0.4	8	A
	6.55 - 6.59	0.2	4	A
	6.80 - 6.84	0.5	10	A
	7.10 - 7.14	0.4	8	A
	7.70 - 7.74	0.4	8	A
	8.30 - 8.34	0.6	12	A
	8.80 - 8.84	0.7	14	A
	9.30 - 9.34	0.7	14	A
	9.80 - 9.84	0.4	8	A
	10.20 - 10.23	0.3	6	A
	10.70 - 10.74	0.5	10	A
	11.40 - 11.44	0.5	10	A
	11.90 - 11.93	0.7	14	A
2	1.55 - 1.58	0.4	8	A
	1.92 - 1.95	0.6	12	A
	2.09 - 2.13	0.6	12	A
	2.72 - 2.75	0.7	14	A
	3.28 - 3.32	0.5	10	A
	3.82 - 3.85	0.6	12	A
	4.16 - 4.20	0.9	18	A
	4.78 - 4.81	0.5	10	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: sit family Pty Ltd

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Project: Proposed Residential Development

Report: A

Location: 22C Burran Avenue, MOSMAN, NSW

Report Date: 30/09/21

Page of 3

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	5.24 - 5.27	0.4	8	A
	5.92 - 5.94	0.4	8	A
	6.17 - 6.21	0.7	14	A
	6.80 - 6.83	0.6	12	A
	7.08 - 7.12	0.6	12	A
	7.91 - 7.95	0.4	8	A
	8.16 - 8.20	0.5	10	A
	8.85 - 8.89	0.6	12	A
3	0.62 - 0.66	0.4	8	A
	0.92 - 0.95	0.5	10	A
	1.08 - 1.12	0.6	12	A
	1.84 - 1.88	0.6	12	A
	2.29 - 2.33	0.7	14	A
	2.78 - 2.81	0.8	16	A
	3.16 - 3.20	0.9	18	A
	3.66 - 3.70	0.7	14	A
	4.12 - 4.15	0.7	14	A
	4.71 - 4.74	0.6	12	A
	5.21 - 5.25	0.8	16	A
	5.81 - 5.83	0.6	12	A
	6.03 - 6.07	0.5	10	A
	6.60 - 6.62	0.5	10	A
	7.32 - 7.35	0.4	8	A
	7.72 - 7.75	0.8	16	A
	8.09 - 8.12	0.4	8	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: sit family Pty Ltd

Ref No: 34431YJ

Project: Proposed Residential Development

Report: A

Location: 22C Burran Avenue, MOSMAN, NSW

Report Date: 30/09/21

Page 3 of 3

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
3	8.52 - 8.55	0.7	14	A
4	0.67 - 0.70	0.5	10	A
	0.94 - 0.98	0.6	12	A
	1.12 - 1.15	0.4	8	A
	1.77 - 1.81	0.2	4	A
	2.66 - 2.70	0.5	10	A
	3.22 - 3.26	0.8	16	A
	3.84 - 3.88	0.7	14	A
	4.16 - 4.19	0.9	18	A
	4.84 - 4.88	0.9	18	A
	5.07 - 5.10	0.9	18	A
	5.72 - 5.75	0.6	12	A
	6.26 - 6.29	0.4	8	A
	6.68 - 6.72	0.4	8	A
	7.12 - 7.14	0.6	12	A
	7.93 - 7.96	0.8	16	A
	8.21 - 8.24	0.7	14	A
	8.66 - 8.69	0.7	14	A

NOTES

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

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CERTIFICATE OF ANALYSIS 279628

Client Details

Client	JK Geotechnics
Attention	Sami Azzi
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34431YJ, 22C Burran Avenue, Mosman, NSW**
Number of Samples	2 Soil, 2 Sandstone
Date samples received	05/10/2021
Date completed instructions received	05/10/2021

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	12/10/2021
Date of Issue	07/10/2021

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference		279628-1	279628-2	279628-3	279628-4
Your Reference	UNITS	BH1	BH4	BH3	BH1
Depth		0.15-0.35	0.1-0.2	1.83-1.85	2.4-2.43
Type of sample		Soil	Soil	Sandstone	Sandstone
Date prepared	-	06/10/2021	06/10/2021	06/10/2021	06/10/2021
Date analysed	-	06/10/2021	06/10/2021	06/10/2021	06/10/2021
pH 1:5 soil:water	pH Units	9.0	6.7	5.3	4.9
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	<10	<10
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	<10	<10
Resistivity in soil*	ohm m	250	350	710	660

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. 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	-			06/10/2021	1	06/10/2021	06/10/2021		06/10/2021	[NT]
Date analysed	-			06/10/2021	1	06/10/2021	06/10/2021		06/10/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	9.0	9.1	1	100	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	95	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	94	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	250	240	4	[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.

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.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ **Method:** HAND AUGER **R.L. Surface:** ~25.5 m
Date: 28/9/21 **Datum:** AHD
Plant Type: **Logged/Checked By:** S.A./J.M.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING				■	REFER TO DCP TEST RESULTS					CONCRETE: 105mm.t				NO OBSERVED REINFORCEMENT
									-	FILL: Sand, fine to coarse grained, light grey and yellow brown, trace of fine to coarse grained sandstone gravel, and possibly sandstone cobbles.	M			
						25				REFER TO CORED BOREHOLE LOG				
							1							
						24								
							2							
						23								
							3							
						22								
							4							
						21								
							5							
						20								
							6							
						19								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34431YJ MOSMAN.GPJ <<DrawingFile>> 06/10/2021 17:11 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
2 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~25.5 m
Date: 28/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
				START CORING AT 0.35m					
		25	1	NO CORE 0.75m					Hawkesbury Sandstone
				CONCRETE: 105mm.t				(1.25m) Cuttings include sand with gravel sized cemented sand nodules.	
				NO CORE 0.11m					
		24		SANDSTONE: fine to medium grained, red brown, orange brown and grey, indistinct rock fabric.	HW	VL - L		(1.37-1.90m) COMMENCE WASHBORING	
			2	NO CORE 1.01m					
ON COMPLETION OF CORING		23	3	SANDSTONE: fine to coarse grained, red brown, grey and orange brown, indistinct rock fabric.	HW	L	0.20 0.20	(2.96m) Be, 15°, P, R, Clay Vn	Hawkesbury Sandstone
80% RETURN		22		NO CORE 0.70m					
		21	4	SANDSTONE: fine to coarse grained, light grey and orange brown, indistinct rock fabric.	HW	VL - L	0.040	(4.30m) J, 40 - 90°, P, R, Clay Vn (4.45m) XWS, 15°, 10 mm.t	Hawkesbury Sandstone
				NO CORE 0.13m					
			5	Extremely Weathered sandstone, clayey SAND, fine to coarse grained, orange brown, light grey and red brown.	XW	D			Hawkesbury Sandstone
		20 19	6	SANDSTONE: fine to coarse grained, light grey, orange brown mottled red brown, indistinct rock fabric.	HW SW	VL - L L - M M	0.070 0.40 0.20 0.50	(5.70m) J, 70 - 80°, P, Cn (5.90m) CS, 0°, 90 mm.t (6.23m) CS, 0°, 5 mm.t (6.71m) J, 45°, Clay FILLED, 5 mm.t	

Graphic Log; Strength scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH; SPACING (mm): 600, 200, 60, 20

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 34431YJ MOSMAN.GPJ <<DrawingFile>> 06/10/2021 17:12 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
1
3 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~25.5 m
Date: 28/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

Water Loss\Level	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness – Specific	General	Formation
80% RETURN	18		SANDSTONE: fine to coarse grained, light grey, orange brown mottled red brown, indistinct rock fabric. *(continued)*	SW	M	0.40 0.40	(7.34m) Be, 15°, P, R, Cn (7.37m) Be, 20°, P, R, Cn (8.15m) J, 45°, P, R, Cn		Hawkesbury Sandstone
	17 16 15	8 9 10 11	SANDSTONE: fine to medium grained, light grey, indistinct rock fabric.	FR		0.60 0.70 0.70 0.40 0.30 0.50 0.50	(10.29m) CS, 0°, 5 mm.t		
	14		as above, but distinctly bedded at 0-15°.			0.70			
	13 12	12 13	END OF BOREHOLE AT 12.00 m						

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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Job No: 34431YJ
Borehole No: BH1
Depth: 0·35m – 12·0m
KODAK Gray Scale
KODAK Color Control Patches
JOB No. 34431YJ BH1 CORING STARTS AT
0
0·35m
NO CORE 0·75m
1
NO CORE 0·105m
NO CORE 1·01m
2
3
NO CORE 0·69m
4
NO CORE 0·13m
5
6
7
8
9
10
11
12
END OF BH 12·0m
JK Geotechnics

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Method: HAND AUGER
R.L. Surface: ~21.8 m
Date: 29/9/21
Datum: AHD
Plant Type:
Logged/Checked By: S.A./J.M.

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					CONCRETE: 130mm.t				8mm DIA. REINFORCEMENT, 120mm TOP COVER
				■					-	FILL: Gravel, fine to medium grained, grey, angular, igneous, trace of fine to coarse grained sand.	M			
										REFER TO CORED BOREHOLE LOG				
						21								
							1							
						20								
							2							
						19								
							3							
						18								
							4							
						17								
							5							
						16								
							6							
						15								

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
2
2 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~21.8 m
Date: 29/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

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
					START CORING AT 0.36m							
		21	1		NO CORE 1.12m					(0.36-1.48m) Possibly Fill		
ON COMPLETION OF CORING		20 19	2 3		SANDSTONE: fine to medium grained, red brown and light grey, indistinct rock fabric.	MW	M	0.40 0.60 0.60 0.70 0.50	600 200 60 20			Hawkesbury Sandstone
100% RETURN					NO CORE 0.01m					(3.57m) Jh, 20°, P, R, Cn		
		18	4		SANDSTONE: fine to medium grained, orange brown and grey, indistinct rock fabric.	MW	M	0.60 0.90				Hawkesbury Sandstone
		17 16 15	5 6		as above, but light grey.	FR SW FR		0.50 0.40 0.40 0.70 0.60	600 200 60 20			

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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CORED BOREHOLE LOG

Borehole No.
2
3 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~21.8 m
Date: 29/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

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
100% RETURN		14	8		as above, but light grey. *(continued)*	FR	M	0.60 0.40 0.50			Hawkesbury Sandstone
					NO CORE 0.03m						
		13			SANDSTONE: fine to medium grained, grey, indistinct rock fabric.	FR SW	M	0.60		(8.76m) XWS, 0°, 10 mm.t	
		12 11 10 9 8	9 10 11 12 13		END OF BOREHOLE AT 8.97 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 34431YJ
Borehole No: BH2
Depth: 0.16-8.97

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BOREHOLE LOG

Borehole No.
3
1 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Method: HAND AUGER
R.L. Surface: ~21.5 m
Date: 29/9/21
Datum: AHD
Plant Type:
Logged/Checked By: S.A./J.M.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING				■	REFER TO DCP TEST RESULTS			☒		FILL: Silty sand, fine to medium grained, grey brown, trace of organic mulch.	M			-
										REFER TO CORED BOREHOLE LOG				
						21								
							1							
						20								
							2							
						19								
							3							
						18								
							4							
						17								
							5							
						16								
							6							
						15								

CORED BOREHOLE LOG

Borehole No.
3
2 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~21.5 m
Date: 29/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

Water Loss\Level	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
	START CORING AT 0.16m					
	NO CORE 0.37m					
	SANDSTONE: fine to medium grained, yellow brown and grey, indistinct rock fabric, trace of siltstone clasts.	MW	M	0.40 0.50 0.60	(1.59m) Be, 10°, P, R, Cn	Hawkesbury Sandstone
	NO CORE 0.12m					
	SANDSTONE: fine to medium grained, yellow brown and grey, trace of siltstone and quartz clasts, indistinct rock fabric.	MW	M	0.60 0.70		Hawkesbury Sandstone
ON COMPLETION OF CORING 100% RETURN	SANDSTONE: fine to medium grained, light grey, trace of siltstone clasts, indistinct rock fabric.	FR		0.80 0.90 0.70 0.70 0.60 0.80 0.60 0.50 0.50	(2.88m) Be, 5°, P, R, Cn	

Point load strength index scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20

RL (m AHD): 21, 20, 19, 18, 17, 16, 15
Depth (m): 1, 2, 3, 4, 5, 6

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

CORED BOREHOLE LOG

Borehole No.
3
3 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~21.5 m
Date: 29/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

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
		14 13	8	⁚⁚⁚	SANDSTONE: fine to medium grained, light grey, trace of siltstone clasts, indistinct rock fabric. *(continued)*	FR	M	0.40 0.80 0.40 0.70			Hawkesbury Sandstone
		12 11 10 9 8	9 10 11 12 13		END OF BOREHOLE AT 8.73 m				600 200 60 20 600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 34431YJ

Borehole No: BH3

Depth: 0.16-8.72m

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BOREHOLE LOG

Borehole No.
4
1 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Method: HAND AUGER
R.L. Surface: ~20.2 m
Date: 30/9/21
Datum: AHD
Plant Type:
Logged/Checked By: S.A./J.M.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING				■	REFER TO DCP TEST RESULTS	20		▨		FILL: Silty sand, fine to medium grained, grey brown, trace of root fibres.	M			GARDEN BED AREA
										REFER TO CORED BOREHOLE LOG				
							1							
						19								
							2							
						18								
							3							
						17								
							4							
						16								
							5							
						15								
							6							
						14								

CORED BOREHOLE LOG

Borehole No.
4
2 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ
Core Size: TT56
R.L. Surface: ~20.2 m
Date: 30/9/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: S.A./J.M.

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 $I_s(50)$	DEFECT DETAILS SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		20			START CORING AT 0.34m							
					NO CORE 0.13m							
					CONCRETE: 120mm.t							
100% RETURN		19	1		SANDSTONE: fine to medium grained, red brown, orange brown and grey, indistinct rock fabric.	MW	M	0.50 0.60 0.40				Hawkesbury Sandstone
					as above, but distinctly bedded at 0-15°.							
					as above, but indistinct rock fabric.		L	0.20		(1.61m) Be, 0°, P, R, Cn		
		18	2		NO CORE 0.59m							
					SANDSTONE: fine to medium grained, orange brown and light grey, indistinct rock fabric.	MW	M	0.50				Hawkesbury Sandstone
ON COMPLETION OF CORING		17	3		as above, but light grey, trace of orange brown.	SW		0.80				
0% RETURN		16	4		as above, but distinctly bedded at 0-15°.			0.70 0.90				
		15	5		as above, but grey and orange brown.			0.90 0.90				
		14	6		as above, but indistinctly rock fabric.			0.60 0.40 0.40		(6.47m) J, 30°, P, S, Fe Cn		

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 34431YJ MOSMAN.GPJ <<DrawingFile>> 06/10/2021 17:12 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JKGeotechnics

CORED BOREHOLE LOG

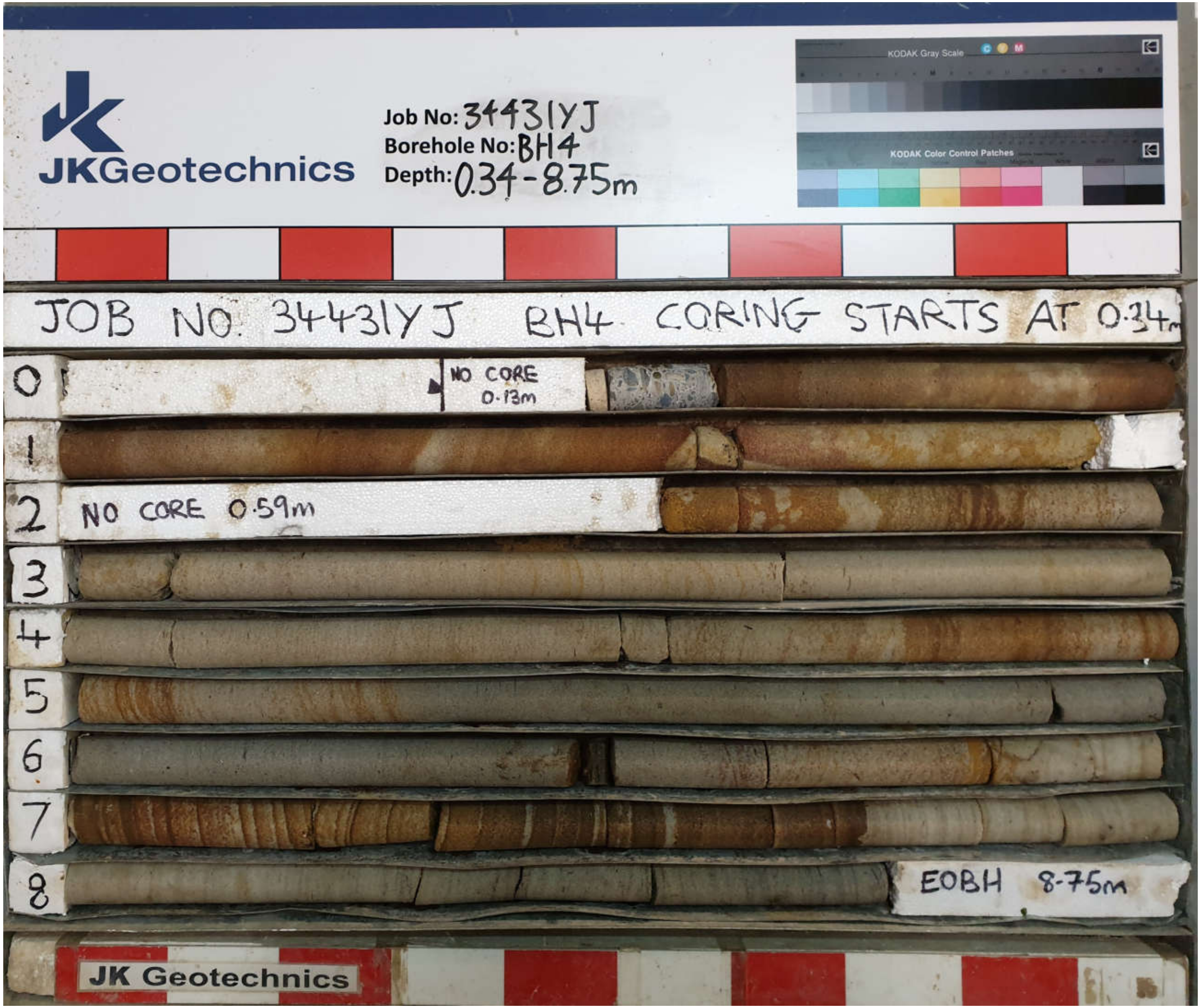
Borehole No.
4
3 / 3

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No.: 34431YJ **Core Size:** TT56 **R.L. Surface:** ~20.2 m
Date: 30/9/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: MELVELLE **Bearing:** N/A **Logged/Checked By:** S.A./J.M.

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
0% RETURN		13			as above, but distinctly bedded at 0-15°.	MW	M	0.60			Hawkesbury Sandstone
						FR					
			8					0.80			
		12						0.70			
								0.70			
					END OF BOREHOLE AT 8.75 m						
			9								
		11									
			10								
		10							600 200 60 20		
			11								
		9									
			12								
		8									
			13								
		7							600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Job No: 34431YJ
Borehole No: BH4
Depth: 0.34-8.75m

DYNAMIC CONE PENETRATION TEST RESULTS

Client: SIT FAMILY PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Job No. 34431YJ
Date: 28-9-21
Tested By: S.A.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	**1**	**2**	**3**	**4**			
Surface RL	≈25.5m	≈21.8m	≈21.5m	≈20.2m			
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	EXCAVATED	EXCAVATED	2	3			
100 - 200	5	5	4/50mm	2			
200 - 300	8	7/80mm	REFUSAL	3			
300 - 400	8/30mm	REFUSAL		4/50mm			
400 - 500	REFUSAL			REFUSAL			
500 - 600							
600 - 700							
700 - 800							
800 - 900							
900 - 1000							
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

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

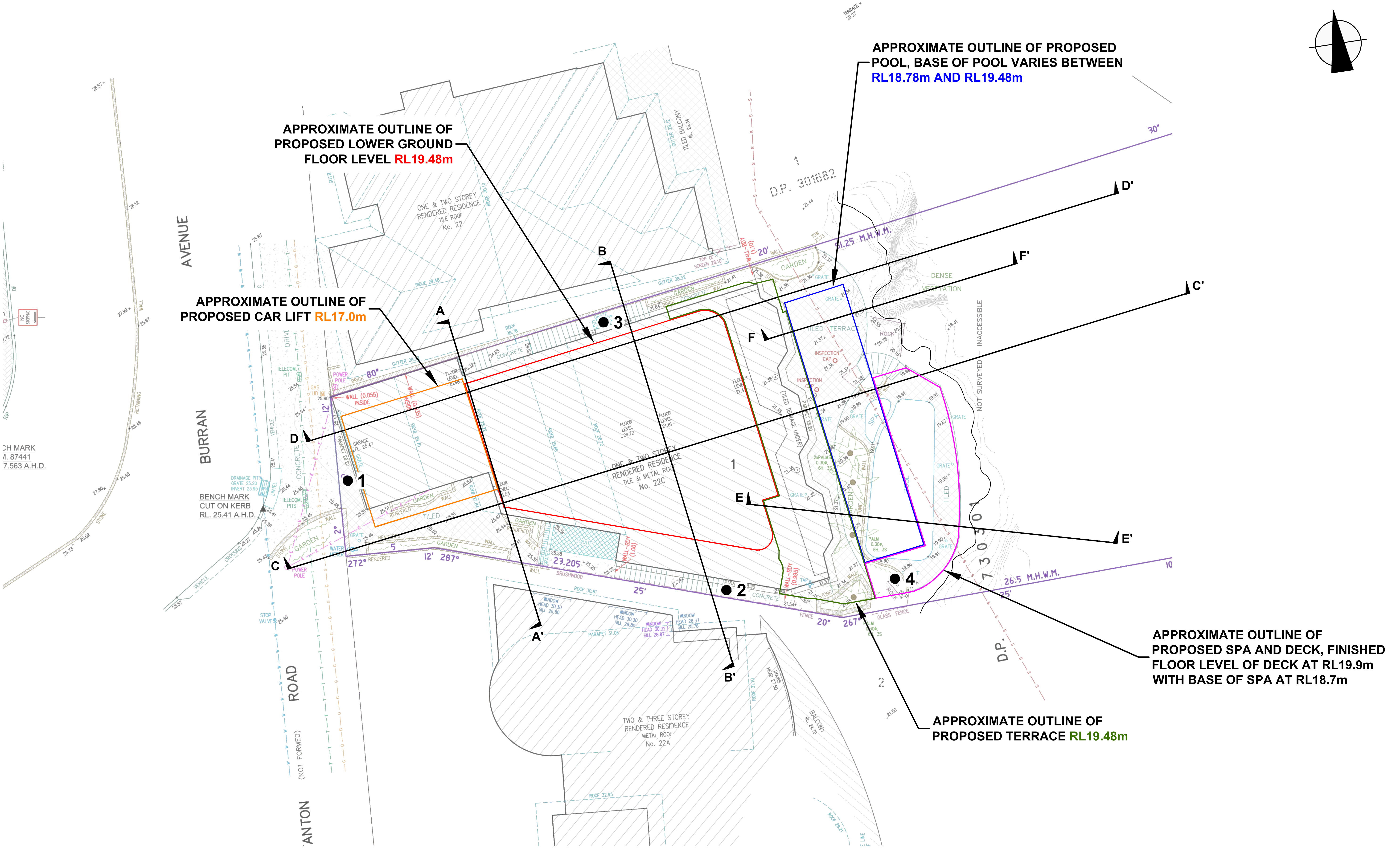
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 1

JKGeotechnics

PLOT DATE: 7/10/2021 4:35:34 PM DWG FILE: Y:\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

● BOREHOLE

A⌴A' CROSS SECTION

NOTES:
1. REFER TO FIGURE 4 FOR CROSS SECTION A-A'.
2. REFER TO FIGURE 5 FOR CROSS SECTION B-B'.
3. REFER TO FIGURE 6 FOR CROSS SECTION C-C'.
4. REFER TO FIGURE 7 FOR CROSS SECTION D-D'.
5. REFER TO FIGURE 8 FOR CROSS SECTION E-E'.
6. REFER TO FIGURE 9 FOR CROSS SECTION F-F'.
7. REFER TO FIGURE 9 FOR GEOTECHNICAL MAPPING SYMBOLS.

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

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

Title: **BOREHOLE LOCATION PLAN**

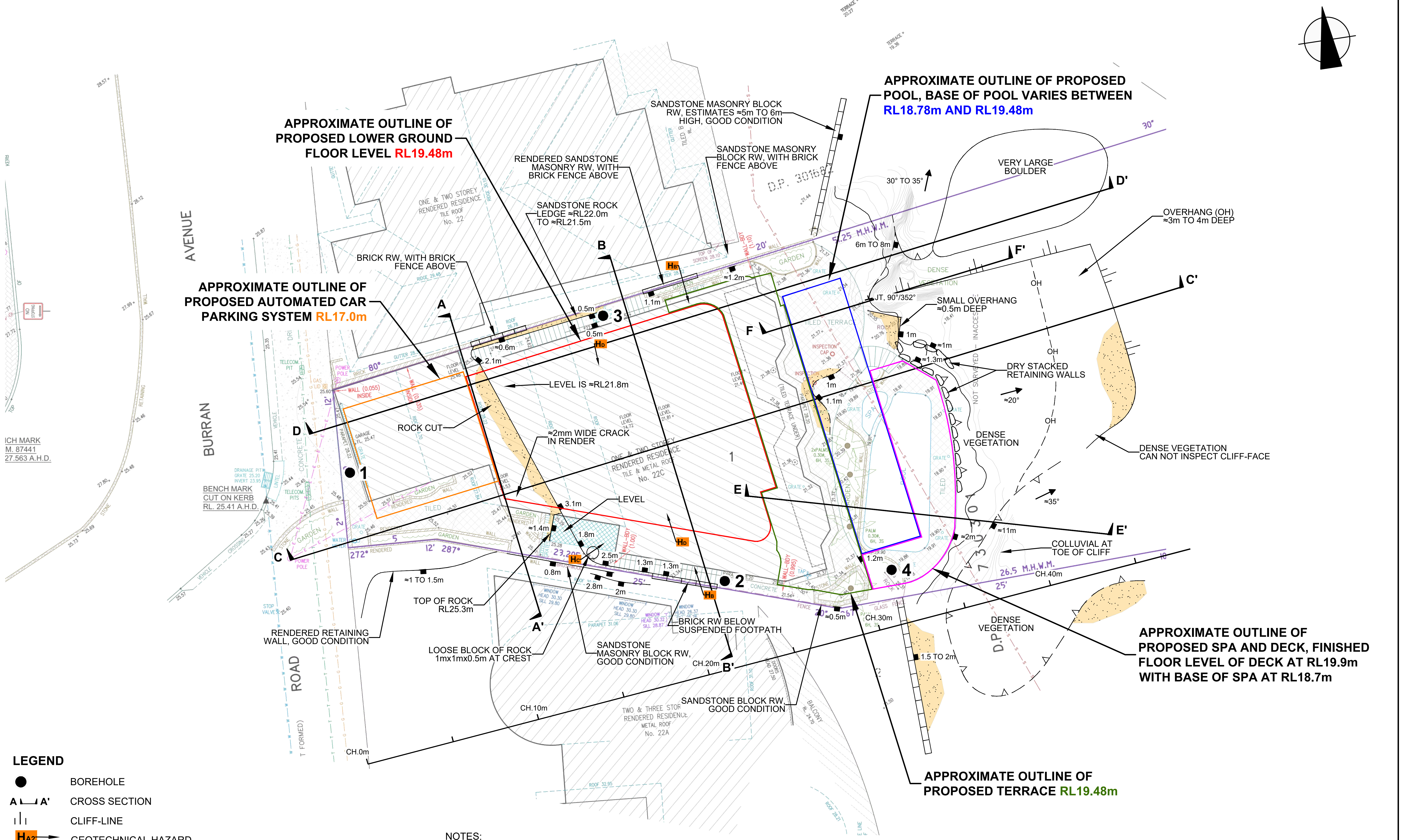
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 2

JKGeotechnics

PLOT DATE: 15/10/2021 4:03:54 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000's\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

●	BOREHOLE
A ⌊⌋ A'	CROSS SECTION
ı|ı	CLIFF-LINE
HA2 →	GEOTECHNICAL HAZARD
JT-90°/082	JOINT - DIP ANGLE+DIP DIRECTION
RW	RETAINING WALL
SS	SANDSTONE
— OH —	APPROXIMATE OVERHANG BOUNDARY
▨	SANDSTONE BEDROCK

NOTES:
1. REFER TO FIGURE 4 FOR CROSS SECTION A-A'.
2. REFER TO FIGURE 5 FOR CROSS SECTION B-B'.
3. REFER TO FIGURE 6 FOR CROSS SECTION C-C'.
4. REFER TO FIGURE 7 FOR CROSS SECTION D-D'.
5. REFER TO FIGURE 8 FOR CROSS SECTION E-E'.
6. REFER TO FIGURE 9 FOR CROSS SECTION F-F'.
7. REFER TO FIGURE 9 FOR GEOTECHNICAL MAPPING SYMBOLS.

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

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

Title: **PLAN OF NOTABLE GEOTECHNICAL FEATURES AND HAZARDS**

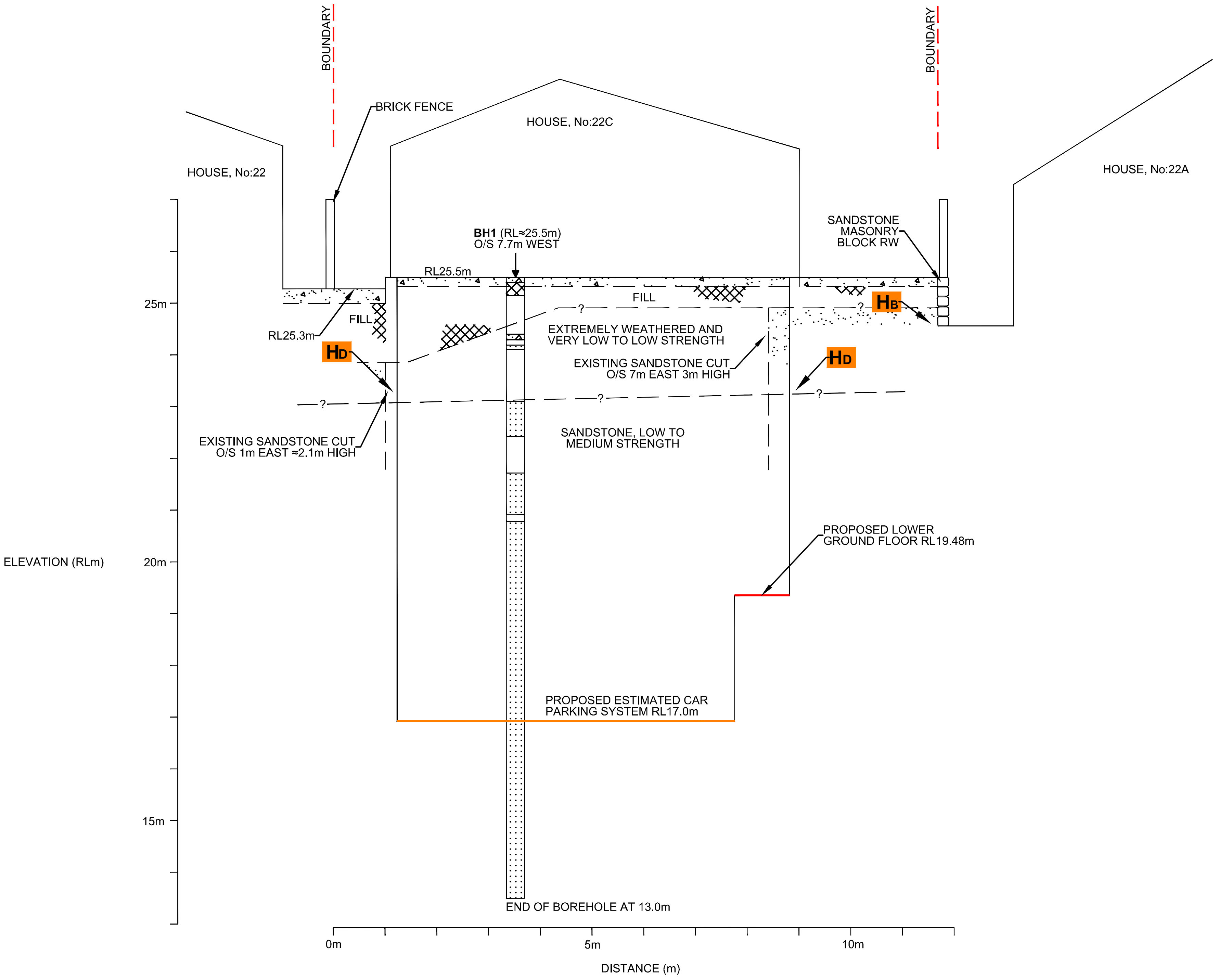
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 3

JKGeotechnics

PLOT DATE: 15/10/2021 4:04:16 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

▨	CONCRETE
—?— —	INFERRED GEOTECHNIAL UNIT
HA2 ⟶	GEOTECHNICAL HAZARD
RW	RETAINING WALL

MATERIAL GRAPHIC

□	NO CORE
⬚	SANDSTONE
▩	FILL

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

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

Title: **SECTION A-A'**

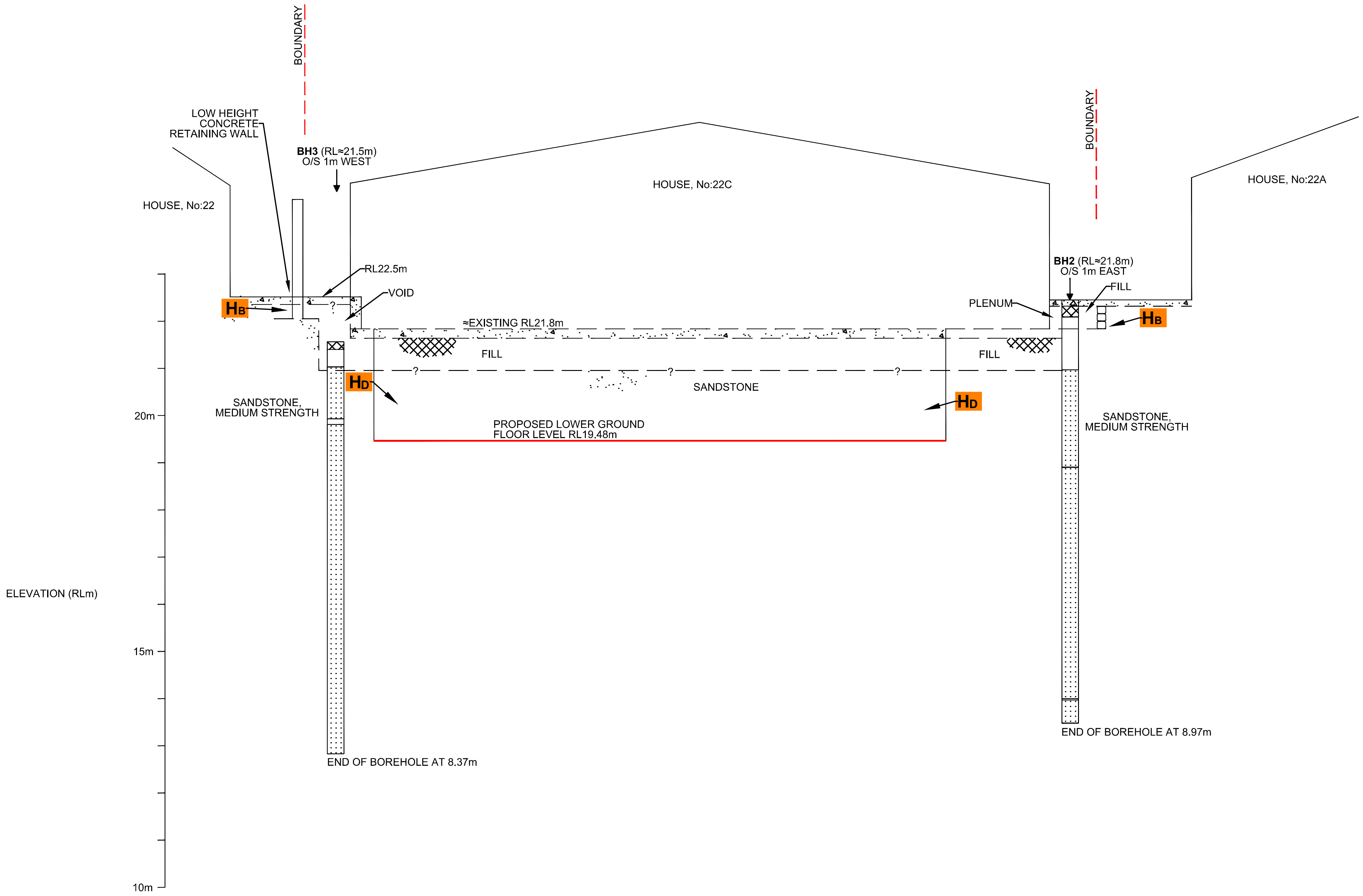
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 4

JKGeotechnics

PLOT DATE: 13/10/2021 3:21:22 PM DWG FILE: Y:\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

▨	CONCRETE
—?— —	INFERRED GEOTECHNIAL UNIT
HA2 →	GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

☐	NO CORE	▨	SANDSTONE
☒	FILL		

0 1 2 3 4 5

SCALE 1:100 @A3 METRES

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

Title: **SECTION B-B'**

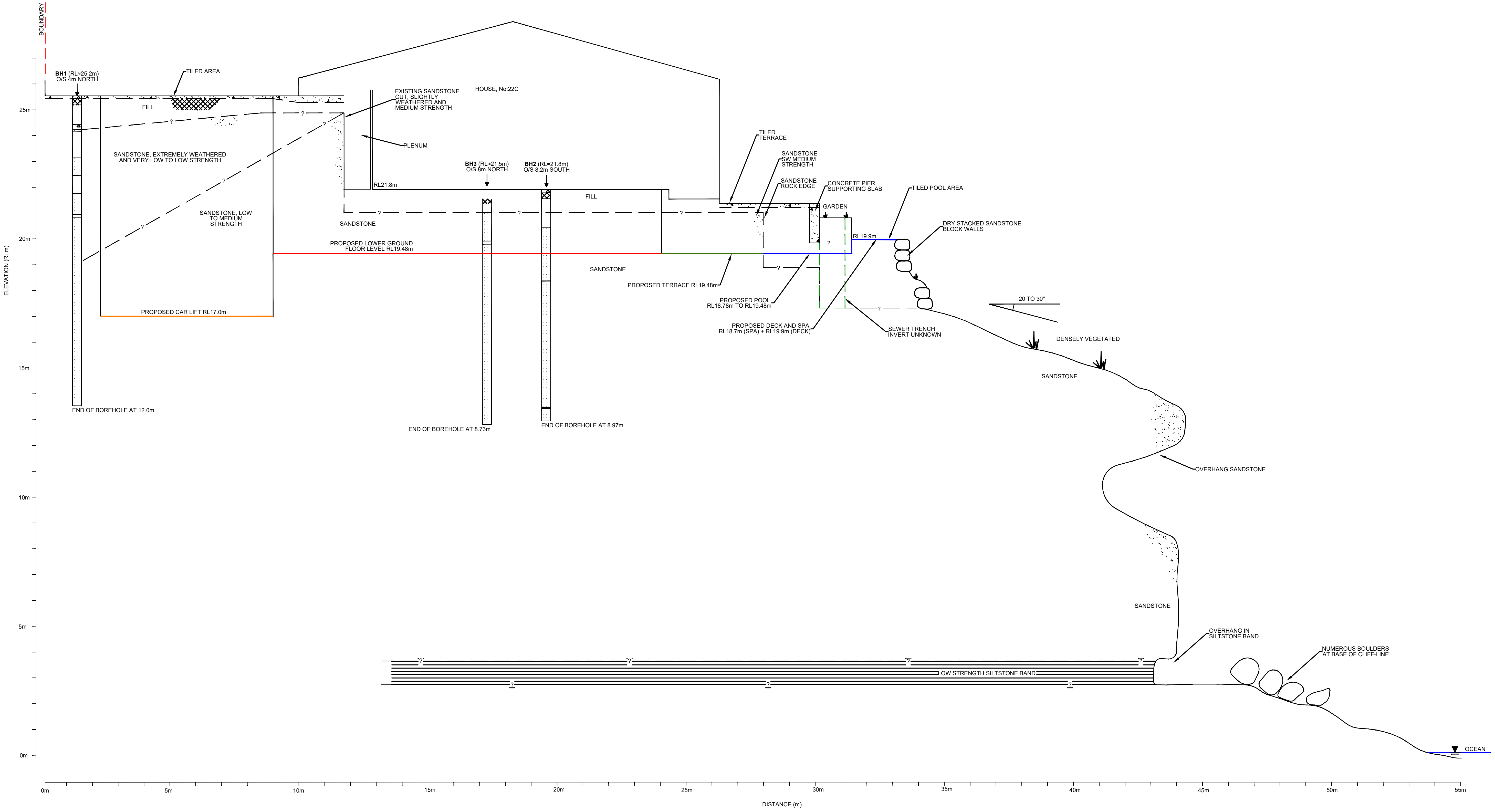
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 5

JKGeotechnics

PLOT DATE: 13/10/2021 3:21:36 PM DWG FILE: Y:\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

▨ CONCRETE

—?— — INFERRED GEOTECHNIAL UNIT

HA2 ⟶ GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

☐ NO CORE

▨ FILL

▨ SANDSTONE

▤ SILTSTONE

0 1.5 3 4.5 6 7.5

SCALE 1:150 @A3 METRES

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

Title:
SECTION C-C'

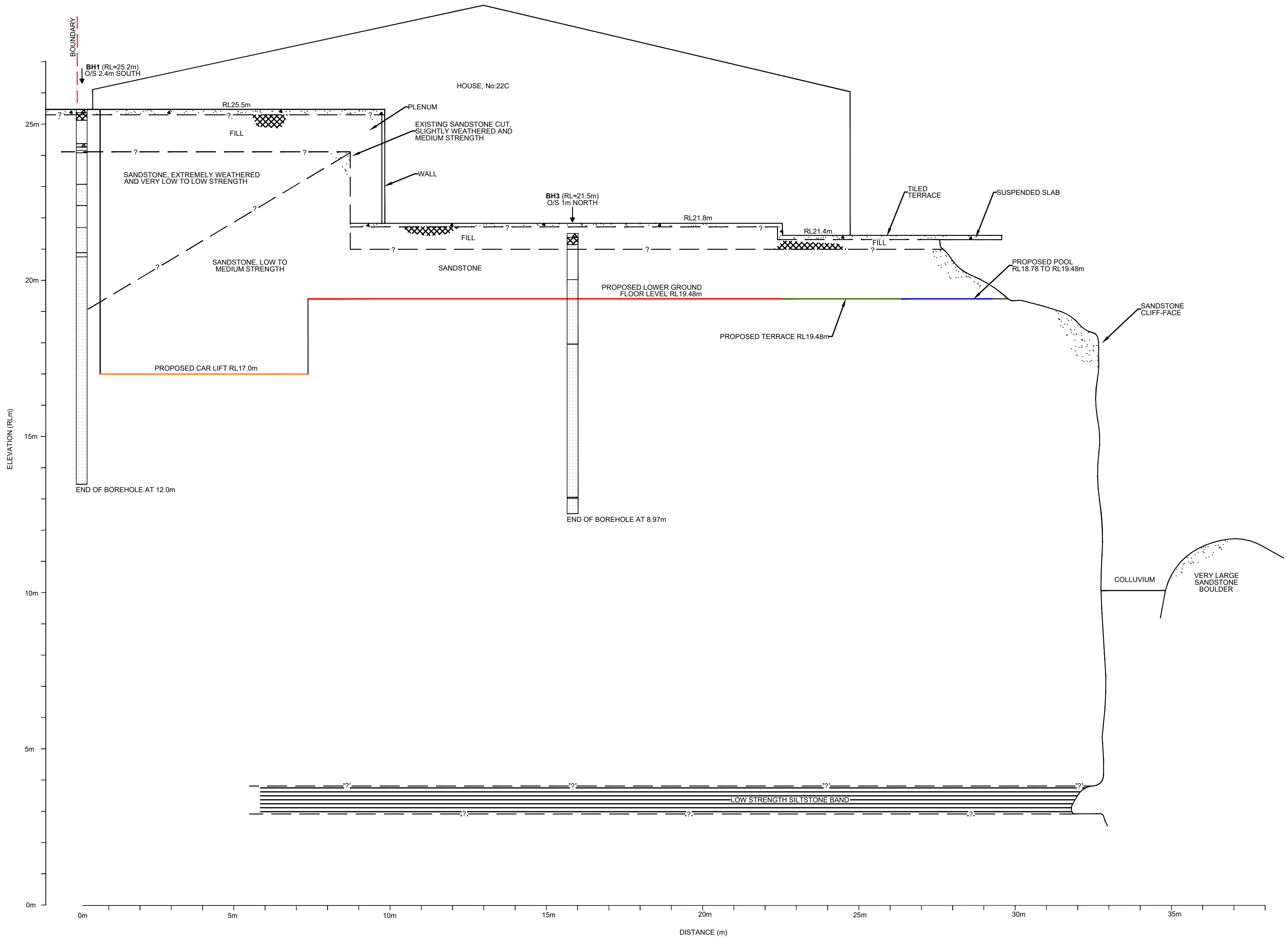
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 6

JKGeotechnics

PLOT DATE: 15/10/2021 4:04:25 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



PLOT DATE: 15/10/2021 4:04:47 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG

LEGEND

▨ CONCRETE

—?— — INFERRED GEOTECHNIAL UNIT

HA2 → GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

☐	NO CORE	░	SANDSTONE
▩	FILL	☰	SILTSTONE

0 1.5 3 4.5 6 7.5

SCALE 1:150 @A3 METRES

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

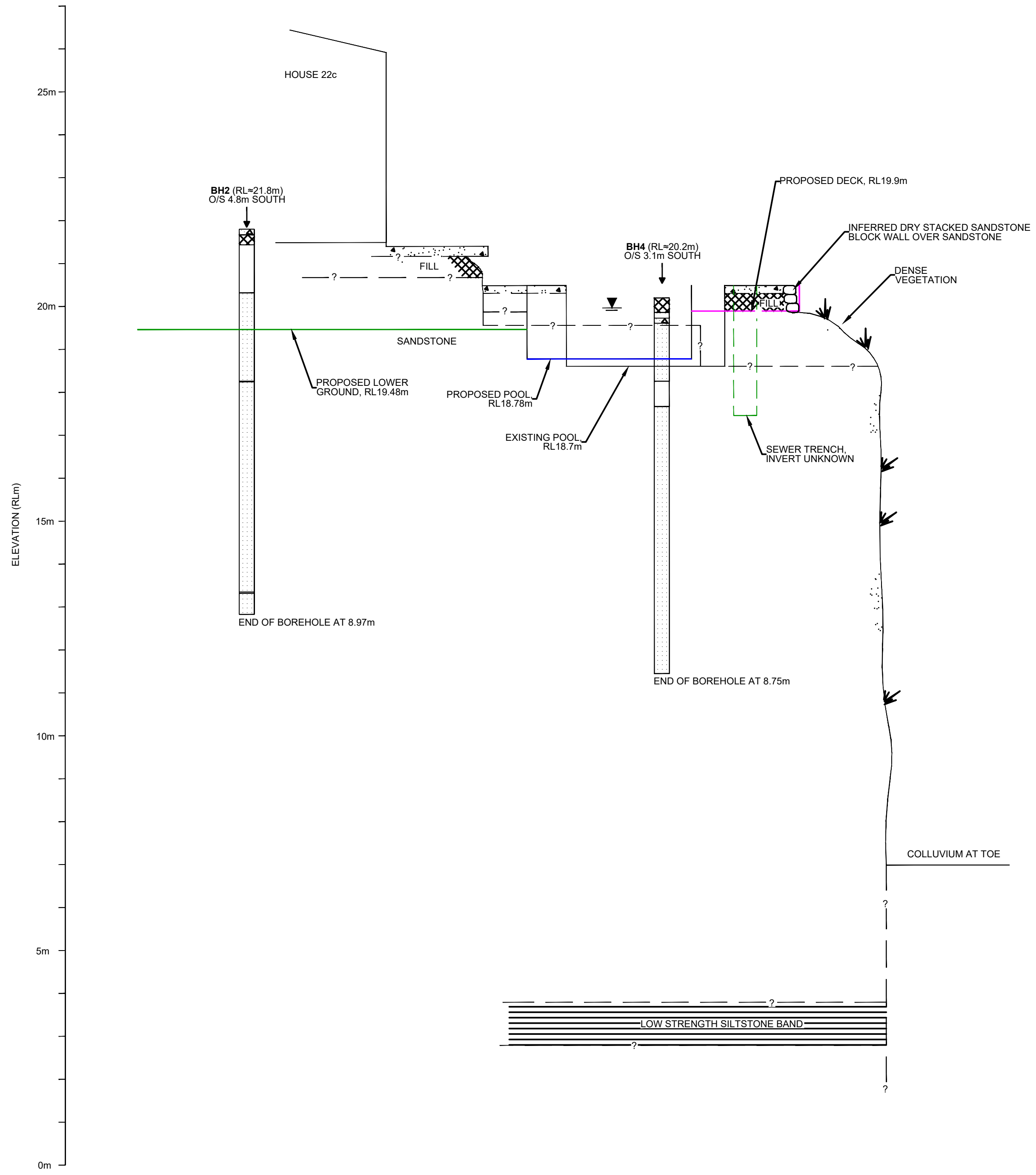
Title: **SECTION D-D'**

Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 7

JKGeotechnics



LEGEND

▨ CONCRETE

—?— — INFERRED GEOTECHNIAL UNIT

HA2 ➝ GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

☐ NO CORE

▨ SANDSTONE

☒ FILL

☰ SILTSTONE

0 1.5 3 4.5 6 7.5

SCALE 1:150 @A3 METRES

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

Title: **SECTION E-E'**

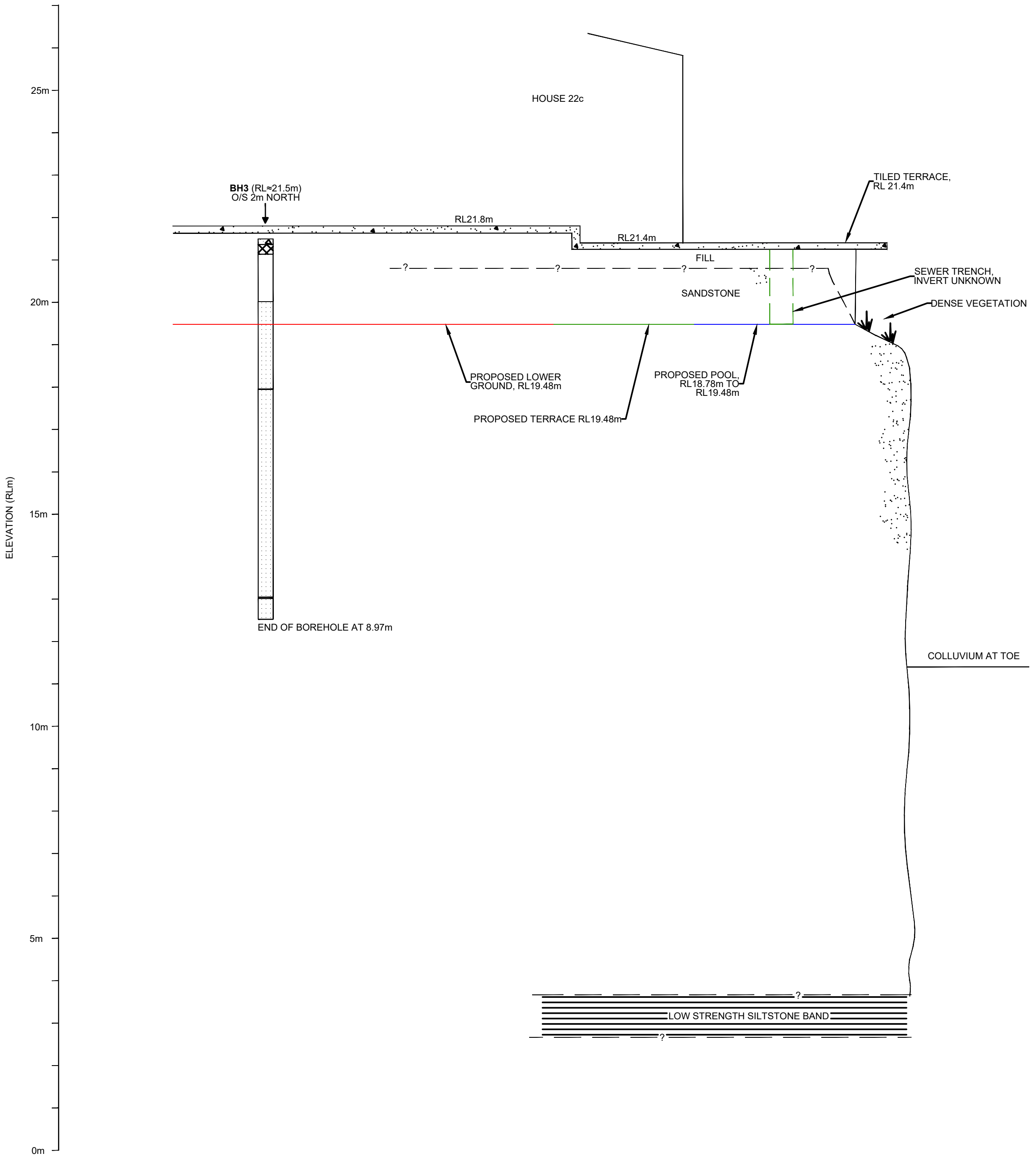
Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 8

JKGeotechnics

PLOT DATE: 15/10/2021 4:05:12 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG



LEGEND

▨ CONCRETE

—?— — INFERRED GEOTECHNIAL UNIT

HA2 → GEOTECHNICAL HAZARD

MATERIAL GRAPHIC

☐ NO CORE

▨ SANDSTONE

☒ FILL

☰ SILTSTONE

0 1.5 3 4.5 6 7.5

SCALE 1:150 @A3 METRES

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

Title:

SECTION F-F'

Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 9

JKGeotechnics

PLOT DATE: 15/10/2021 4:06:13 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG

TOPOGRAPHY

Symbol *Ground Profile*

convex
concave
} well defined or angular break of slope

convex
concave
} poorly defined or smooth change of slope

breaks of slope
changes of slope
} convex and concave too close together to allow the use of separate symbols

sharp
rounded
} ridge crest

3 Cliff or escarpment or sharp break 40° or more (estimated height in metres)

15 Uniform Slope
10 Concave Slope
8 Convex Slope
} Slope direction and angle (Degrees)

Top
Bottom
} Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

OTHER FEATURES

Boulder

Seepage/spring

Swallow hole for runoff

Natural water course

Open drain, unlined

Open drain, lined

Fenceline

Property boundary

Dry Stone Wall

J —— J 200 Major joint in rock face (opening in millimetres)

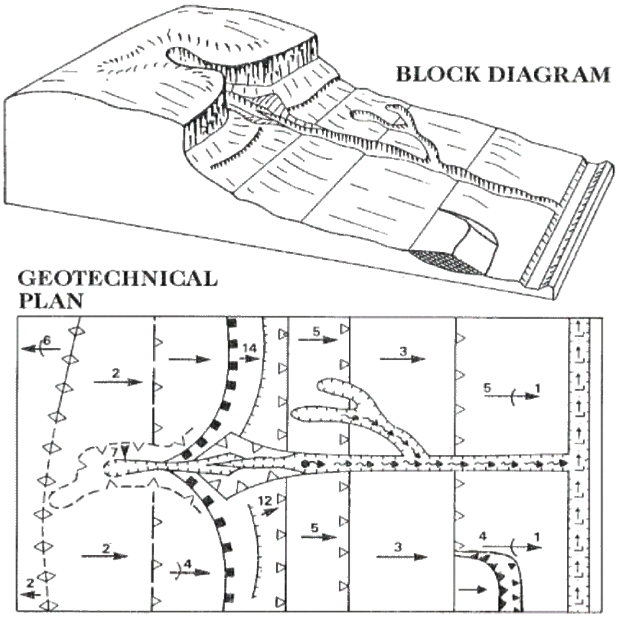
– T – T – 10 Tension crack (opening in millimetres)

Masonry or concrete wall

Ponding water

Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



(After Gardiner, V & Dackombe, R. V. (1983), Geomorphological Field Manual; George Allen & Unwin).

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

Title:
GEOTECHNICAL MAPPING SYMBOLS

Location: 22C BURRAN AVENUE, MOSMAN, NSW

Report No: 34431YJ

Figure No: 10

JKGeotechnics

PLOT DATE: 15/10/2021 4:09:35 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\34000'S\34431YJ MOSMAN\CAD\34431YJ.DWG

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 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	

				Field Classification of Silt and Clay			Laboratory Classification
Major Divisions		Group Symbol	Typical Names	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:

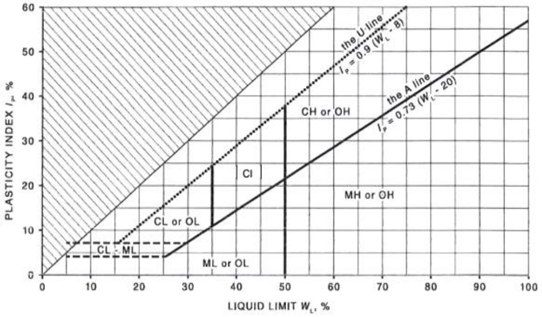
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

Abbreviations Used in Defect Description

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

APPENDIX A

LANDSLIDE RISK MANAGEMENT TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

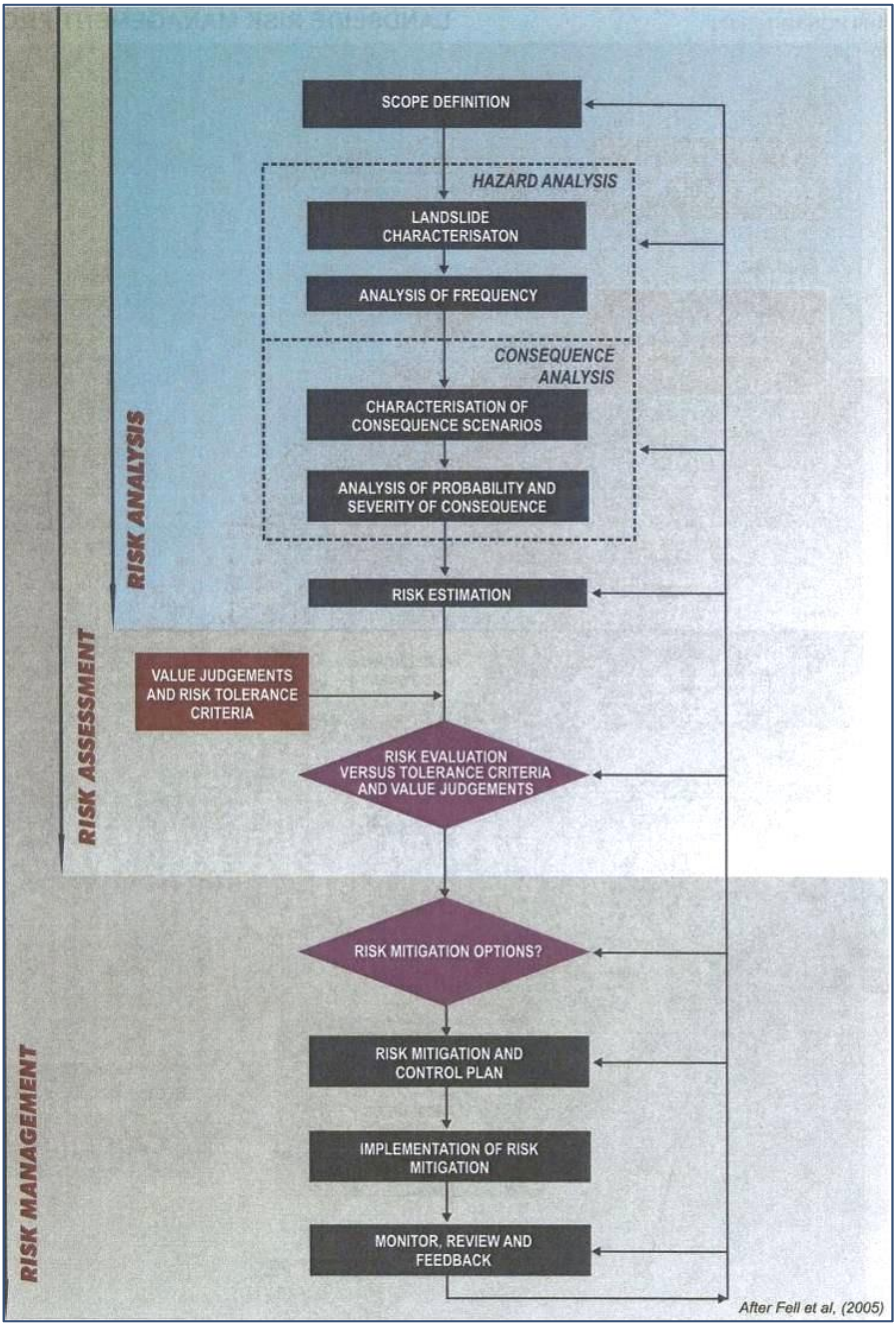
Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
Consequence	The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
Hazard	A condition with the potential for causing an undesirable consequence (the landslide). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.
Individual Risk to Life	The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
Landslide Intensity	A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. These are two main interpretations: (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability ***(continued)***	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

NOTE: Reference should be made to Figure A1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.



After Fell et al, (2005)

FIGURE A1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-2}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (continued)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		**CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)**				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book “Guideline Document Landslide Hazards” published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board’s website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both “potential” and “actual” landslides must be taken very seriously. The present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with series consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can “run-out” from above, “regress” from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else’s land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

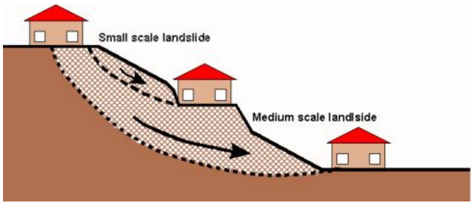


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

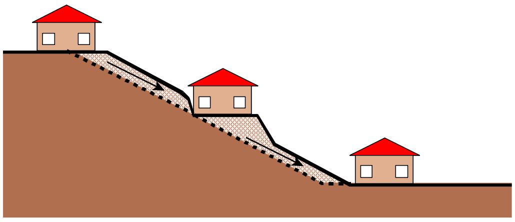


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

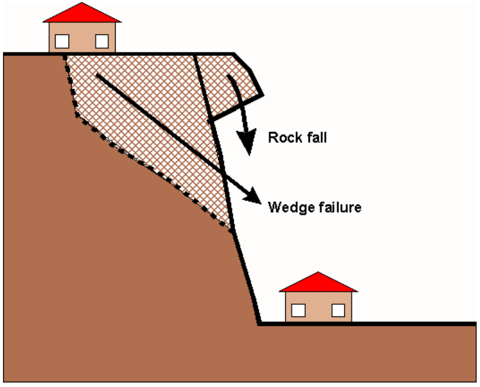


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

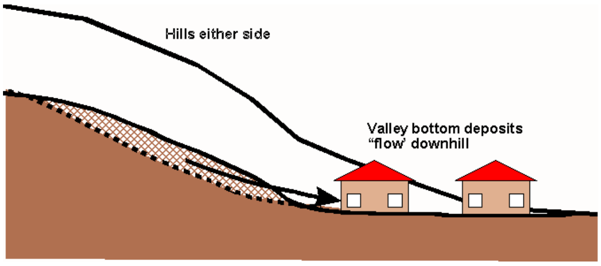


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of the repairs and temporary loss of use if the landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2 – LIKELIHOOD

Likelihood	**Annual Probability**
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable risk level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		**Significance - Geotechnical engineering requirements**
Very high	VH	**Unacceptable** without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**May be tolerated** in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding, ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other Australian GeoGuides:

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APPENDIX B

SOME GUIDELINES
FOR
HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

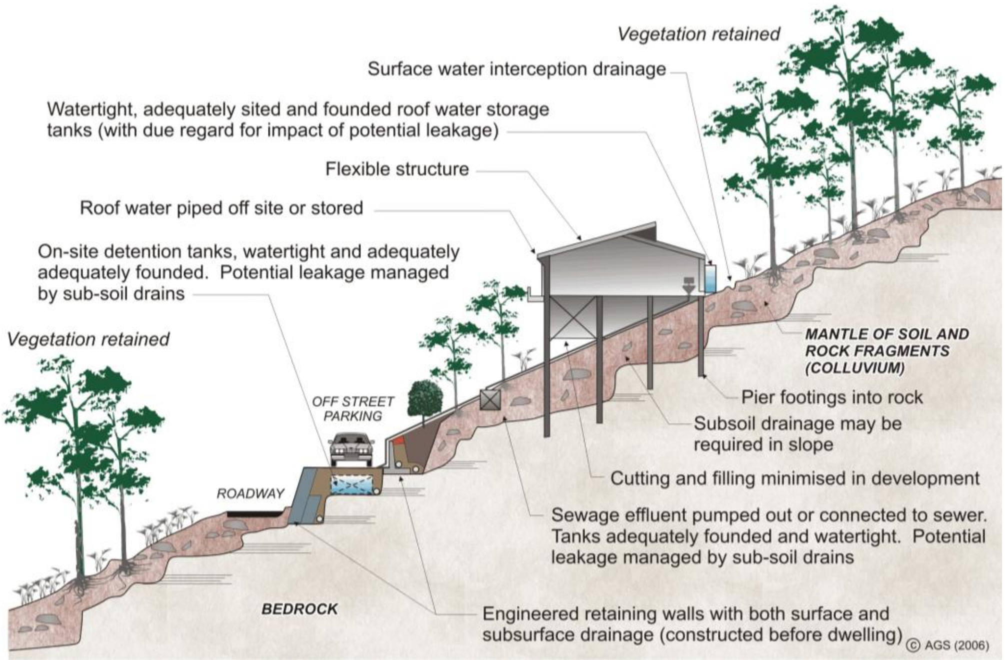
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminant bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements.
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance (including onto properties below). Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc. in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on bedrock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within bedrock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide generous falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge of roof run-off into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use of absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by a geotechnical consultant.	
SITE VISITS	Site visits by consultant may be appropriate during construction.	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is extracted from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in *Australian Geomechanics*, Vol 42, No 1, March 2007 which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES FOR **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that due to level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfill the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

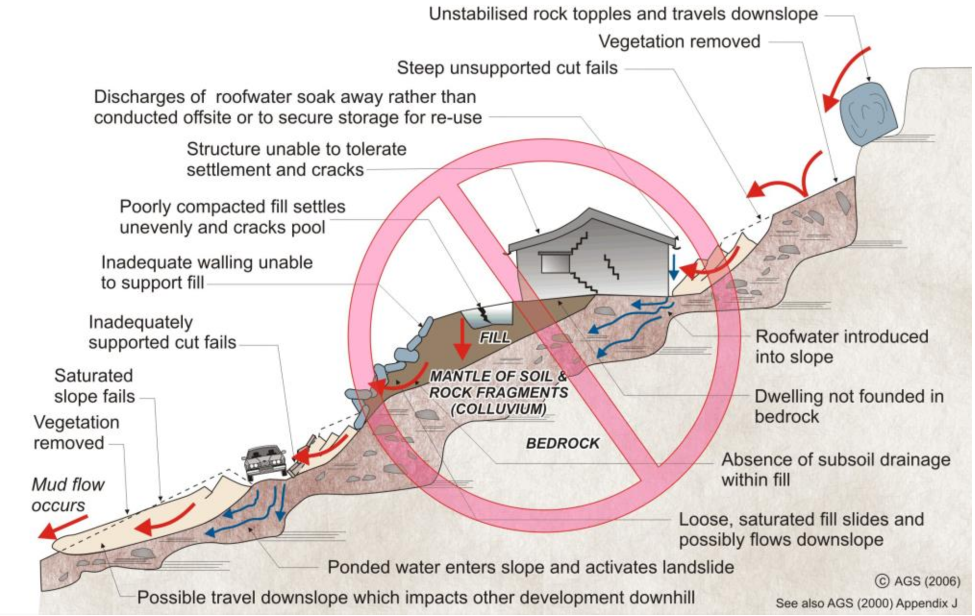
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soaks into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herringbone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.