REPORT TO

TRUSTEE OF THE NORTHERN BEACHES CENTRAL BUSINESS PARK NO.2 UNIT TRUST

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED SUBDIVISION

AT

120 OLD PITTWATER ROAD, BROOKVALE, NSW

Date: 16 October 2025
Ref: 37745SFrpt Rev1

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical assessment for the proposed land subdivision at 120 Old Pittwater Road, Brookvale, NSW. The location of the site is shown in Figure 1. The assessment was engaged by signed Acceptance of Proposal form dated 3 July 2025 and was carried out in general accordance with our fee proposal, Ref: P72157SF, dated 25 June 2025.

We understand the proposed development includes subdivision of the site into three separate lots (Lots A, B and C), as detailed in the drawing prepared by Reid Campbell (Sheet No. 1240041_ASK_00, Rev B dated 30 April 2025) which will then be sold as industrial zoned lots. Concepts of the proposed development on Lot C have been provided, as shown in the architectural drawings prepared by Reid Campbell (per the Cover Page, Sheet No. 1240041-A001, various revisions dated 22 September 2025 and 10 October 2025), where it is proposed to demolish the existing on-grade car park and construct a multi-storey storage facility. The lowest floor level is expected to be at RL30.5m which will require only minor cut and fill on the eastern side of Lot C, however will require excavations about 14m deep on the western side given the existing steeply sloping hillside. Details of development on Lot A and B are not available but we understand will involve demolition of the existing structures and construction of a similar development as Lot C, although we expect minimal/reduced excavations given the existing rock faces. Regardless, this report is intended to address the general subdivision of the site and the geotechnically related aspects and must be updated once details of the proposed developments are known.

The purpose of the assessment was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on stability risk assessment, excavation conditions, hydrogeological considerations, retention systems, footings, subgrade preparation, engineered fill and pavement design parameters.

This geotechnical assessment was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E37744Brpt, for the results of the environmental site assessment.

2 ASSESSMENT PROCEDURE

The assessment comprised:

- A site walkover by our Senior Associate Geotechnical Engineer on 8 July 2025.
- A search of the JK Geotechnics project database to identify relevant geotechnical investigations completed on the site and nearby.
- A review of aerial photography and digital street view (NearMap and Google Earth).
- A review of the regional geology maps.

No subsurface investigations were carried out as part of the assessment.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site lies in an area of topography changing from relatively steep hilly area to a relatively flat low lying area. The site itself appears to extend up from the toe of an eastward facing hill sloping down between 15° and 60°.

At the time of fieldwork, the site contained a multi-storey concrete building that generally appeared in good condition with no visible defects based upon a cursory external inspection. The external areas generally comprised of asphaltic concrete (AC) and concrete paved car parks and loading zones. The pavements appeared in moderate condition with cracking up to 5mm observed in some areas based upon a cursory inspection.

The eastern portion of the site was relatively level with only a gentle easterly slope down at 1° to 2°. The western portion of the site was steeply sloping and, except for the driveway, stepped up via two sub-vertical sandstone cut faces. The eastern cut face adjacent to the existing building was 8m to 10m high and the western cut face varied from being level with the existing cut face to 3m high; these cuts are shown approximately as Rock Bank B and Rock Bank A on Figure 2. The sandstone cut faces were assessed as being at least medium strength, distinctly weathered to fresh and were generally left unsupported although some areas contained shotcrete and rock bolt supports. Further discussion on the rock bank slopes are provided in Section 4.

The site has an eastern street frontage onto Old Pittwater Road which runs relatively level and comprises of AC pavement that appears in moderate to good condition with some cracking observed based upon a cursory inspection from the side of the road. The neighbouring southern property comprises of multiple two to three storey concrete buildings that generally appear in good condition. The neighbouring buildings abut the common boundary and appear to be raised from the subject site by about 1m to 2m towards the eastern end of the boundary increasing to about 10m towards the western end.

The northern adjacent properties contained two to three storey concrete buildings that generally appear in good condition based upon a cursory external inspection. The adjacent properties are separated by an AC paved right of carriageway road and as a result the properties are offset by about 10m from the site boundary. The western end of the neighbouring property is bounded by a sub-vertical sandstone cut face up to at least 15m high or higher. To the west the site is bounded by steeply sloping and heavily vegetated bushland with sandstone boulders and outcrops evident.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by Hawkesbury Sandstone of the Wianamatta Group comprising medium to coarse grained quartz sandstone, very minor shale and laminite lenses. It should also be noted that the site lies approximately 100m south-west from the deep deposits of stream alluvial and estuarine sediments.

The boreholes disclosed a subsurface profile generally comprising pavements and fill overlying natural sands and clayey sand with sandstone bedrock at varying depths across the site; significant sandstone exposures are evident in the rear western parts of the site within the steep cliff cuts (Rock Bank A and Rock Bank B shown approximately on attached Figure 2).

Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations. A summary of the subsoil conditions, as encountered, is presented below:

Pavements and Fill

All the boreholes, except BH3, penetrated AC or concrete pavement of varying thicknesses. The concrete in BH4 was 155mm thick with no observed reinforcement. The remaining boreholes encountered AC pavement of thickness varying between 15mm and 40mm.

The fill initially encountered was generally a sandy gravel/gravelly sand ‘road base’ type material below the existing pavements. The ‘road base’ type fill extended to depths of 0.1m to 0.25m below existing surface levels. A fine to medium grained sandy fill was then generally encountered containing varying amounts of fine to coarse grained igneous and sandstone gravel and clay. The fill was generally assessed as moderately to well compacted, where possible.

The fill in BH6 extended deeper to 2.4m depth which is significantly deeper when compared to the other boreholes. We believe the deeper fill profile may be due to colluvium/slope wash material that was deposited in the past. In BH1 at approximately 0.52m depth it appears concrete was encountered. We do not know the thickness of the concrete as the drilling was unable penetrate through the concrete.

Natural Soils

Natural soils were only encountered in BH2, BH3 and BH5 at 0.8m, 0.9m and 1.4m depth, respectively. The natural soils generally comprised an upper layer of fine to coarse grained silty sand, containing gravel, and then clayey sand as well as sandy clay layers of varying thickness and depths. The sands were generally assessed as medium dense to dense relative density and the clays were of hard strength. The sandy clay encountered at 2.8m depth in BH3 was assessed as having a moisture content less than the plastic limit.

Bedrock

Sandstone bedrock was encountered in all boreholes (except BH3 which could not penetrate the buried concrete obstruction) at varying depths. The sandstone bedrock at the central and western portions of the site was relatively shallow with depth ranging from 0.4m to 2.4m depth below existing surface levels. The sandstone on initial contact was generally of at least medium strength with ‘TC’ bit refusal on high strength bedrock occurring in all boreholes. Significant sandstone exposures are also evident in the rear western parts of the site within the steep cliff cuts (Rock Bank A and Rock Bank B shown approximately on attached Figure 2).

The eastern portion of the site encountered deeper sandstone bedrock with extremely weathered and extremely low strength sandstone encountered at 5.5m to 5.7m depth improving to slightly weathered to fresh medium to high strength at 7.6m to 8.3m depth, or about RL9.0m.

Groundwater

Groundwater seepage during drilling was encountered only in BH8 at 1.3m depth. The remaining boreholes were all dry during and on completion of drilling. BH3, BH6 and BH8 were left open to allow for further groundwater monitoring and the groundwater level was measured at 6.1m depth in BH3 and 0.4m depth in BH8 up to 24 hours after completion. BH6 was still dry up to one hour after completion of drilling. Unfortunately not all boreholes were able to be left open upon completion of drilling due to public safety concerns and access constraints.

3.3 Laboratory Test Results

The results of the moisture content tests (refer to attached Table A) of fragment samples of the sandstone generally correlate well with the field logging assessments of rock strength.

The results of the Atterberg Limits and Linear Shrinkage test (refer to attached Table A) on sample of the sandy clay confirmed the clay to be of medium plasticity and indicated the clay to have a moderate potential for shrink/swell movements with changes in moisture content.

Two bulk samples of the upper natural soil subgrade (i.e. the silty sand with gravel) from BHs 2 and 3 in the eastern part of the site were tested for compaction and four day soaked CBR, and the results are shown on attached Table B. The CBR results were significantly different, but in both case high values. BH2 sample had a CBR of 60% and BH3 sample had a CBR 19%, with both samples compacted to achieve 98% of Standard Maximum Dry Density. We attribute the much higher value of BH2 silty sand sample to the presence of more and perhaps coarser gravel (note the high SPT N value of 42), whilst BH3 silty sand had perhaps less or finer gravel (note the much lower SPT N value of 15).

4 STABILITY ASSESSMENT

4.1 Geotechnical Assessment

The majority of the site is relatively level with the exception of two rock banks, which have been determined to be the critical hazards present onsite. For the purposes of this report the western-most cut face has been designated as 'Rock Bank A' and the existing cut face adjacent to the existing building as 'Rock Bank B' (please refer to the attached Figure 2).

The stability of rock cut faces exposed during proposed excavations to cut back the prevailing rock banks can be affected by the presence of adversely oriented defects, which may not have become evident during the inspection due to obscuring cover of vegetation or distance of observation vantage points. Our assessment of rock bank slope conditions is based on information available during the site inspection and borehole investigation and is limited by the feasibility of interpolation between data locations. In some situations it was impracticable to identify specific defect using available observations methods and vantage points at the time of the fieldwork. It is possible that the stability of the proposed rock bank excavations could depend on the presence of a randomly orientated defect or zone which was not identified during the site inspection. We recommend that the rock face be progressively mapped during excavation to confirm rock conditions and identify any features which may require special treatment. Further discussion on rock cut slope stability measures and precautions are provided in Section 5 below.

Rock Bank (Slope) A

This heavily vegetated slope initially slopes down from the west at about 20° before steeply sloping down at generally 60° to 90° adjacent to the existing car park. The upper slope appears to comprise of clayey gravelly colluvium with large sandstone boulders, most likely rock falls from further up the slope. The colluvium appears to be at least 3m thick in some areas of the slope but observations were limited due to the dense vegetation.

The sandstone bedrock is exposed in the sub-vertical cut face adjacent to the existing cut face and, based upon a tactile examination using a geo-pick, was assessed to be of at least medium to high strength sandstone. The sandstone appears to be bedded at approximately 0° to 10° from the horizontal with a north to north-west dip direction. Extremely weathered seams and joints were occasionally observed in the cut face but are considered to be relatively minor. The slope shows no signs of slope movement based upon observations made from the car park.

Rock Bank (Slope) B

This existing cut face behind the existing building is approximately 10m high and comprises of a roughly 7m high sub-vertical sandstone cut face and then a 2m to 3m high steep slope. Based upon a tactile examination using a geo-pick the sandstone bedrock was assessed as being at least medium strength. A number of infilled joints appear to have been covered in shotcrete with weep holes installed. In parts the existing shotcrete has deteriorated with visible cracks and lost sections.

The majority of the cut face comprises of sandstone bedrock but there are a number of sections with shotcrete and concrete installed. A section south of the overhead bridge appears to have had a rock block slide occur in the past and rock bolts have since been installed to stabilise the cut face. The overhead bridge itself is supported on concrete columns that appear to be founded near the crest of the cut face, although may be supported by a pile under but this was not observed. The upper slopes above the cut face are generally heavily vegetated except under the overbridge where soil is exposed with a net covering to prevent debris fall. The cut face adjacent to the southern side of the existing building was covered in shotcrete and the area behind the crest heavily vegetated.

4.2 Potential Landslide Hazards

We consider that the potential landslide hazards associated with the site to be the following:

A Stability of existing rock faces:
  (i) Rock Face A, and;
  (ii) Rock Face B.

B Stability of the natural hillside slope:
  (i) In front of the existing site;
  (ii) Beneath the existing site; and
  (iii) Upslope of the existing site.

C Stability of proposed rock cut faces on Lot C

These potential hazards are indicated in schematic form on the attached Figure 2.

4.3 Risk Analysis

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Use has been made of data in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A indicates that the assessed risk to property varies between Low or Very Low, which would be considered 'acceptable' in accordance with the criteria given in Reference 1.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 10^{-6}. This would be considered to be 'acceptable' in relation to the criteria given in Reference 1.

4.4 Risk Assessment

The typical requirements is to adopt suitable measures 'to remove risk'. It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site and/or development cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners are made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, where it is required that 'reasonable and practical measures have been identified to remove risk', it means that there has been an active process of reducing risk, but it does not require the geotechnical engineer to warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

In preparing our recommendations, we have assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all Council's buried services are, and will be regularly maintained to remain, in good condition.

We consider that our risk analysis has shown that the site and existing and proposed development can achieve the 'Acceptable Risk Management' criteria provided that the recommendations given in Section 5 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

5 COMMENTS AND RECOMMENDATIONS

We consider the site suitable for the proposed sub-division and concept built form on Lot C, particularly given the performance to date of the existing site structures. The following provides preliminary advice of a general nature only given the details of the proposed developments are not known, other than early concept drawings for Lot C. As such, once further details are known, we recommend the comments and recommendations contained within this report are reviewed and further advice provided. Additional geotechnical investigation are expected to be required and further geotechnical reports for the lots.

5.1 Dilapidation Surveys

Prior to commencement of demolition and excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the north and south of the site that fall within a distance of 20m from the proposed rock excavation areas.

The dilapidation surveys should include internal and external inspections of the buildings, where all defects including defect location, type, length and width are described and photographed. The respective owners of the neighbouring buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

5.2 Excavation Conditions

A major part of the proposed development will entail ‘hard rock’ excavations in the rear western parts of the site for Lot C and potentially other areas of the site. Excavations may be to remove unstable fill and soils, especially in the eastern parts of the site to allow undercroft pavement and driveway pavement construction. This would result in minimal excavations towards the eastern boundary (i.e. with Old Pittwater Road), but up to about 7m to 14m of excavation to achieve the required finished floor and undercroft parking levels towards the western boundary for Lot C.

All excavation recommendations should be complemented by reference to Safe Work Australia’s ‘Excavation Work Code of Practice’, dated July 2015.

An assessment of the excavation characteristics of the various main geotechnical strata of the site is presented below. The excavatability of the sandstone bedrock and the selection of appropriate excavation equipment have been assessed on the basis of augered borehole logs (attached) and our visual and tactile inspections of the rock bank slopes, where possible, taking into consideration the obscuring vegetation cover with few vantage points. It should be noted that rock strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Assessment of excavation characteristics and productivity is not an exact science and contractors must make their own evaluation based on experience with specific equipment, and their own study of the borehole information and their inspection of the exposed rock in the rock bank slopes; they might also request for rock cored boreholes to be completed at the site (note: we recommend the completion of rock cored boreholes). The ease with which excavation of rock is achieved depends upon the equipment used, the skill, and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.

Depending on final elected development and earthwork levels, the materials to be excavated will comprise uncontrolled fill in the eastern parts of the site and in the western parts of the site, for the most part, the sandstone bedrock, but with some removal or soil cover (or retention by batters or walls) and removal of boulder or stabilization of boulders in the rear rock bank slope A (refer to Figure 2). The soils can be readily excavated by buckets of a large hydraulic excavator. Excavation into the rock bank slopes will mostly present ‘hard rock’ excavation conditions, requiring due care regarding equipment to be used to reduce vibration transmission during rock excavation that could affect adjoining infrastructure. Due care will also be required during cutting into the rock banks to maintain their continuing stability and ensure safety of personnel and equipment. The works will have to be completed with regular geotechnical inspections and by experienced contractors in rock slope excavations, to ensure rock faces are supported where necessary, boulders at top of slopes that may become unstable are removed or stabilised.

Excavation in extremely low to very low strength sandstone can be achieved using a Caterpillar D9 tractor or equivalent, probably with some light to medium ripping. Some of this material can probably also be excavated with a large excavator bucket. Localised stronger bands/zones may require heavy ripping or the use hydraulic rock hammers.

For the most part the sandstone will be of medium or greater strengths, which will present hard or heavy ripping or “hard rock” excavation conditions, and may require a higher capacity and heavier bulldozer for effective production. Most likely rock bank slopes will be cut back by using hydraulic rock breakers, although with the large areas to be excavated, productivity is expected to be lower. This equipment would also be required for detailed excavations such as footings or services in the rock.

The use of excavators with hydraulic impact hammer attachments, for rock excavation should be approached with considerable caution, as there will likely be direct transmission of ground vibrations to nearby structures and buildings. Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. We recommend that the acceptable limit for transmitted vibrations be set at quite a low peak particle velocity of 5mm/s for frequencies of less than 10Hz at foundation level. If it is found that transmitted vibrations are unacceptable, then it may then be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary rock grinder, rock saws, or jackhammers.

If rock hammers are to be used, we recommend that the initial excavation in rock should preferably be commenced away from likely critical areas and instrument vibration monitoring undertaken. The monitoring program should be confirmed when details of the contractor’s excavation methods and sequence are known. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

In addition, we recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. The contractor should also be provided with a copy of this report to make his own judgement on the most appropriate excavation equipment.

5.3 Hydrogeological Considerations

Groundwater seepage is not expected to be a significant issue to proposed excavations since this was not encountered in most boreholes and only minor seepage flows were observed on the rock bank slopes, unless significantly deeper excavations compared to current site levels are proposed. We would expect seepage rates to be manageable using conventional sump and pump or gravity drainage systems due to the most likely low permeability of the soils and bedrock. Notwithstanding, surface drainage must be provided at the tops of all cut (and any fill) slopes. Discharge of surface runoff over top and faces of cuts or fill slopes or ponding of water on bench areas of the slopes must not be allowed.

5.4 Retention Systems

We would expect that all soil and extremely weathered excavation sides would have to be cut at batter slopes or retained, or otherwise entirely removed. A method of maintain excavation stability within soil and extremely weathered rock excavation areas would be to cut the excavations sides at shallow batter slopes if there is sufficient space for accommodating temporary batters. Batter slopes of no steeper than 1 Vertical in 1 Horizontal (1V:1H) are recommended for temporary stability of excavation sides within the natural clayey sand and sandy clays and extremely weathered sandstone of the site, subject to geotechnical inspection as the excavation progresses. The retaining walls would then be constructed at the toe of the batters and subsequent backfilling undertaken. Provision of benches may be required depending on the quality of the materials on the slope.

The medium or greater strength sandstone may be cut vertically and unsupported subject to regular geotechnical inspections as the excavations progresses. The rock cuts should be progressively mapped during excavation by experience engineering geologists or geotechnical engineers to confirm site conditions and identify any features which may require special treatment. However, some allowance should be made for the potential larger scale instability that occasionally exists within bedrock. Should these joints exist, flatter batters (possibly of the order of 1V in 1H or flatter) or large capacity rock anchors can be required; the cost of the latter would be relatively high and delays to the excavation process with consequential cost implications would occur. To manage the potential risks associated with any continuous joints and other defects in the rock faces, we recommend that the excavation should not be allowed to advance more than 1.5m vertically between inspections and the excavation should be staged or stepped so that a whole face is not excavated 1.5m vertically between visits.

Some allowance should be made for stabilisation measures. It would be unusual to complete such an excavation without some form of support being required to the rock faces, though this may take forms other than rock bolting. Any potentially unstable blocks in the exposed sandstone face should be clearly identified with paint markings so that proposed remedial work can be easily communicated to, and undertaken, by the contractor. We expect that face works probably need to include the following:

- Scaling down of some small blocks using a crowbar etc.
- In critical areas of the rock face, localised support (e.g. shotcrete, rock bolts, dowels, buttresses) may be required.
- Mesh may need to be draped and fixed to the face to prevent small blocks from falling and endangering site personnel during construction.

If batters cannot be formed or are not desired then shoring or retained walls would be required and should be designed to support the excavation in the fill, soil and any poorer quality (extremely low to very low strength) sandstone, with the toe of the wall founded at least 0.5m into sandstone of low to medium strength. We advise that cantilevered concrete block or reinforced concrete walls may be constructed adjacent to the crest of the vertical sandstone face where it is to support retained heights of less than around 1.5m and only where some higher lateral and vertical movements of adjoining ground can be tolerated.

Any retaining wall required at the site which is capable of some movement may be designed for lateral earth pressures that are estimated using conventional triangular distributions employing an active coefficient of earth pressure, K_a, of at least 0.35 and the bulk unit weight of $20kN/m^3$. An allowable coefficient of passive resistance, K_p, of 2 may be considered for design of retaining walls, which are embedded below the base of the excavation. Bulk unit weights of $20kN/m^3$ and $22kN/m^3$ should be adopted for the soil and weathered bedrock profiles, respectively. The aforementioned earth pressure parameters apply to a horizontal backfill surface and, if inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill taken as a surcharge load. All applicable surcharge loadings (e.g. sloping backfill) should be taken into account in the wall design. All retaining walls should incorporate permanent drainage provisions or be designed to withstand hydrostatic pressures

For the potential development on Lot C, the existing Rock Bank A will be excavated, however it is expected that Rock Bank B will remain. The existing development may have been constructed in the 70's and therefore if the existing stabilisation measures were constructed around the same time, then the rock bolts and shotcrete would likely be at the end of their design life. Furthermore, there was evidence that some of the shotcrete is deteriorating. As such, we recommend that once details of the developments on Lots A and B is known, that a joint inspection by a geotechnical and structural engineer is undertaken of Rock Face B to assess the condition of the existing stabilisation and provide an agreed approach for replacement of existing stabilisation measures and any additional measures.

5.5 Footings

Based on the available information, our recommendation is that all structures are uniformly supported on the sandstone bedrock foundation strata. Footings are likely to comprise a combination of shallow spread type footings where the rock is shallow or exposed in the western part of the site, and pile type footings where the rock occurs deeper in the eastern parts of the site. Where footings approach existing cut faces, high level footings may be feasible but will be dependent on the rock face stability but piled footings may be necessary.

The following criteria are recommended for design and construction of footings founded on the sandstone bedrock.

1. The spread or pile footings should be proportioned for Allowable Bearing Pressure (ABP) not exceeding 1,000kPa when founded into the sandstone of at least very low strength. Bored pile or CFA pile footings may be used to reach the deeper sandstone bedrock of the eastern parts of the site. We recommend that the piles or pile have a nominal embedment of about 0.3m into the sandstone foundation stratum.
2. If bored piles or piles are used to reach the sandstone of at least medium strength then the ABP used for design may be increased to 1,500kPa. In the shallow rock areas of the western parts of the site spread footings may be possible on the rock of medium strength. It is likely that greater bearing pressures (e.g. 3,500kPa) may be adopted on this quality sandstone subject to completing recommended cored boreholes with strength testing of recovered rock cores and assessment of rock defects.

3. An allowable shaft adhesion (ASA) of equivalent to 10% of the above ABP values may be adopted for design of pile sockets, in compression, through the sandstone. For uplift or tension, the aforementioned ASA value should be halved. The shaft adhesion values are recommended on condition that cleanliness and roughness of pile sockets and bases are achieved.
4. For piles socketed into the sandstone, we recommend that large capacity drilling rigs with rock augers be used to drill the piles. We recommend that rock socket lengths be kept as short as practical due to the difficulties and expense of drilling into deeper sandstone of high or greater strengths, which cause TC auger bit refusal during drilling of the boreholes. Piles should be poured immediately or at very latest, on the same day, as drilling, cleaning and inspection. Special tools should be used to roughen the sides of load bearing pile sockets into the sandstone. Dewatering of deep bored piles may be required using sump pumps. All loose or softened debris should be cleared from the base of all piles prior to concreting.

The initial stages of footing excavation, particularly if piles are adopted, should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

The site has been altered by fill and hence, is classified as Class 'P' in accordance with AS2870-2011. We note that the underlying natural sandy clays are considered to be moderately reactive equivalent to Class ‘M’. Although the behaviour of reactive clays and its effects on a building or other movement sensitive structures is very complex, the prediction of ground movements may be undertaken in accordance with the method suggested in AS2870-2011 “Residential Slabs and Footings – Construction”. We advise that in the strict sense AS2870 site classification does not apply to this size structure but it is a useful guide in highlighting the potential foundation problems of the soils of this site.

5.6 Subgrade Preparation

The main geotechnical issues with other earthworks, including subgrade preparation, under floor slab (and external pavement areas) are to do with any existing fill that may remain after excavation for the proposed developments on the subdivision. The existing fill appears to be variably compacted. The fill is deemed unsuitable as a bearing stratum for warehouse footings and floor slabs. The fill is also considered a ‘moderate risk’ (of poor performance) as a supporting subgrade under external pavements. Again, we prefer that the fill be removed and replaced with engineered fill.

Earthworks recommendations provided in this report should be complemented by reference to AS3798. In summary, we recommend the following subgrade preparation that:

1. After reaching the bulk excavation level for the undercroft car parking levels, all exposed soil subgrade should be proof rolled with at least 8 passes of a heavy (not less than 12 tonne) smooth drum vibratory roller. The purpose of the proof rolling is to detect any soft or heaving areas. Caution is required when proof rolling near any neighbouring improvements and buried services.

2. The final pass should be undertaken in the presence of a geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction.
3. Unstable subgrade detected during proof rolling should be locally excavated down to a stiff or sound base and replaced with engineered fill or further advice should be sought. Allowance should be made for either, tyning, aerating and drying the subgrade, or removal and replacement with a select imported fill, or lime/cement stabilisation.
4. It is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

5.7 Engineered Fill

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in layers not greater than 200mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density (SMDD).

The existing fill and natural soils at the site are acceptable for re-use on condition that the soils used are clean (i.e. free of organics and inclusions greater than 75mm size), free of contaminants (in this respect refer to the JKE report). The natural clay soils should be compacted in maximum 200mm loose layers to a density strictly between 98% and 102% of SMDD and at moisture content within 2% of Standard Optimum. All clay fill should preferably be used in the lower fill layers. Thus, the use of clay materials for engineered fill will entail more rigorous earthwork supervision and compaction control, time for drying out the soils and hence, possibly a greater eventual cost for earthworks.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. We recommend that full time Level 1 control of fill compaction, as defined in AS3798-2007, be adhered to on this site. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

During construction of the fill platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program.

5.8 Floor Slabs

Whilst column loads will likely be required to be supported on footings, it may be feasible for a slab on ground uniformly supported on the natural clays or engineered fill between the columns as a 'floating' slab provided it can be designed to accommodate potential shrink-swell movements. Where not considered practical, the slabs should be designed as a fully suspended floor slab. We note that where floor slabs approach rock faces, such as Rock Face B, it may be required to suspend a portion of the slab regardless if the rock face is deemed not suitable to support the slab. We expect the floor slabs will span over fill, slopewash, residual clay and sandstone bedrock which should be considered in the slab design. As discussed above, given the depth of fill the site is classified as Class 'P' in accordance with AS2870-2011, however we would expect shrink-swell movements in the order of that defined by a Class 'M' site. If existing clayey soils are re-used as engineered fill, then this will also increase the shrink-swell potential of the soils.

In addition to the potential shrink-swell movements, there is potential for differential movements to occur as a result of the differing founding conditions of the slab spanning over possibly fill, residual clay and sandstone bedrock. The slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints. If the slab will directly overlie bedrock, we recommend that an underfloor drainage blanket be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate, such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump or gravity drain for disposal to the stormwater system. Once further slab design details are known, we recommend the potential impact of the reactivity of the clayey soils and differential founding conditions are reviewed and discussed further.

Where a fully suspended floor slab is adopted, no particular subgrade preparation would be required, but any vegetation, root affected soils or deleterious fill material should be stripped. Fill may then be placed as 'form fill' with only nominal compaction and without the need for density testing of the fill during placement. Where soils are present, void formers below the suspended slab of at least 50mm thickness would be required to account for the potential shrink-swell reactivity of the underlying clayey soils.

5.9 Pavement Design Parameters

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Refer to Section 5.6 on subgrade preparation and other earthwork procedures including engineered fill specifications and compaction control.

Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as truck turning and manoeuvring. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice.

The upper natural soil profile of silty sand subgrade have soaked CBR values of 19% and 60%, both very high values probably due to the presence of gravel. However, based on the Atterberg Limits of the sandy clay lower values of CBR probably of the order of 5% are likely in areas where the sandy clay may form the pavement subgrade. As a prudent, cautious measure we recommend on a preliminary basis that pavement thickness designs be based on this estimated CBR of 5%.

Further soaked CBR tests may be carried out on representative samples of the subgrade. If the existing fill is removed and replaced with imported fill, the CBR of the imported material may be taken into account. These design values should be confirmed by inspection and DCP testing of the subgrade following proof rolling.

All upper (base) course are recommended to be crushed rock to TfNSW QA specification 3051 (2022) unbound base and compacted to at least 100% of Standard Maximum Dry Density. All lower (sub-base) course are recommended to be crushed rock to TfNSW QA specification 3051 (2022) unbound base or ripped/crushed sandstone with CBR greater than 40%, maximum particle size of 60mm, well graded and Plastic Index less than 10. All lower course material should be compacted to an average of no less than 100% of SMDD, but with a minimum acceptance value of 98% of SMDD.

Concrete pavements are recommended to have a sub-base layer of at least 100mm thickness of crushed rock to TfNSW QA specification 3051 (2022) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD. Concrete pavements should be designed with an effective shear transmission of all joints by way of either doweled or keyed joints.

Careful attention to subsurface and surface drainage is required in view of the effect of moisture on the clay soils. Pavement levels will need to be graded to promote rapid removal of surface water so ponding does not occur on the surface of pavements.

5.10 Further Geotechnical Input

The full detailed built form DA required for the site will require specific geotechnical reports based on the detailed design, rather than the preliminary intent of this report which addresses the subdivision of the site into Lots A, B and C, as well as the concept built form on Lot C. We recommend that prior to excavation commencing that several rock cored boreholes are completed over the western portion of the site, and the eastern portion for profiling the rock quality with more comprehension and if higher footing bearing pressures are required.

We summarise below the recommended additional geotechnical input that is anticipated needs to be carried out:

- At least eight rock cored boreholes to be completed across the site. A meeting of the design team, once the design has been further advanced, would be of benefit to discuss the geotechnical issues in more detailed and determine the scope of the further detailed investigations.
- Dilapidation surveys for the neighbouring structures, especially as rock breakers/hammers are almost certain to be used.
- Monitoring of groundwater seepage into the excavation to confirm drainage requirements.

- At least initial and periodic vibration monitoring during bulk excavation.
- Progressive inspection of excavated cut faces to confirm if additional support or treatment is required.
- Foundations inspections and testing.
- Proof rolling, density testing and inspection of any fill subgrade to remain place and of any new fill to be places at the site (should comprise engineered fill with compaction control).

We also recommend a review of proposed earthworks and structural drawings and section detail (once available) in order to confirm that these follow the guidelines of this report.

Furthermore, it will be essential during earthworks and construction that regular geotechnical inspections and testing be commissioned to check initial assumptions about earthworks and foundation conditions and likely variations that may occur between borehole/test locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for such variations in subsurface conditions and for excavation and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections.

The preliminary recommendations provided in this report should be reviewed following the additional geotechnical investigation as well as after these inspections. Furthermore, the recommendations provided herein should also be reviewed once exact development details, such structural layout, earthwork levels, floor levels, structural loads etc., are determined.

It is likely that further advice/input will be required during the structural design to address issues that may not have been addressed in this report. To some degree, this is an "iterative" process between evaluation of the geotechnical site conditions and the structural design. For the earthworks, piling and other foundation works, we strongly recommend that only competent contractors be considered, and that they are provided with a full copy of this report.

6 GENERAL COMMENTS

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

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

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 PROPERTY

POTENTIAL LANDSLIDE HAZARD	**A – Stability of Existing Rock Faces**		**B – Stability of Natural Hillside Slope**			**C – Stability of Proposed Cut Faces**
	Rock Face A	Rock Face B	In front of the existing site	Beneath the existing site	Upslope of the existing site	Excavation on Western Side of Lot C
Assessed Likelihood	Unlikely	Unlikely	Barely Credible	Barely Credible	Rare	Rare
Assessed Consequence	Minor	Medium	Minor	Medium	Minor	Minor
Risk	Low	Low	Very Low	Very Low	Very Low	Very Low
Comments	The above risk assessment is on the basis the stabilisation recommendations are implemented	The above risk assessment is on the basis the stabilisation recommendations are implemented			Existing area upslope of Rock Face A, assumed to comprise shallow colluvial/slopewash soils	Assumes regular inspections by a geotechnical engineer during excavation

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A - Stability of Existing Rock Faces**		**B – Stability of Natural Hillside Slope**			**C – Stability of Proposed Cut Faces**
	Rock Face A	Rock Face B	Excavation on Western Side of Lot C	Beneath the existing site	Upslope of the existing site	Excavation on Western Side of Lot C
Assessed Likelihood	Unlikely	Unlikely	Barely Credible	Barely Credible	Rare	Rare
Indicative Annual Probability	10^{-4}	10^{-4}	10^{-6}	10^{-6}	10^{-5}	10^{-5}
Persons at risk	Persons at toe of rock face parking car	Persons using fire escape	Persons at front of site	Persons within building	Persons accessing upslope	Persons at toe of rock face
Number of Persons Considered	1	2	2	10	1	1
Duration of Use of area Affected (Temporal Probability)	5mins/day (0.0035)	5mins/week (0.0005)	2 hours/day (0.083	12 hours/day Mon-Sat (0.429)	1hr/month (0.0014)	10hr/day (0.417)
Probability of not Evacuating Area Affected	0.1	1.0	0.1	0.5	0.5	0.2
Spatial Probability	0.5	1.0	0.1	1.0	0.8	0.8
Vulnerability to Life if Failure Occurs Whilst Person Present	0.1	1.0	0.1	0.9	0.1	0.9

POTENTIAL LANDSLIDE HAZARD	**A - Stability of Existing Rock Faces**		**B – Stability of Natural Hillside Slope**			**C – Stability of Proposed Cut Faces**
	Rock Face A	Rock Face B	Excavation on Western Side of Lot C	Beneath the existing site	Upslope of the existing site	Excavation on Western Side of Lot C
Risk for Person most at Risk	1.8×10^{-9}	5.0×10^{-8}	8.3×10^{-11}	1.9×10^{-7}	5.6×10^{-10}	6.0×10^{-7}
Total Risk	1.8×10^{-9}	1.0×10^{-7}	1.7×10^{-10}	1.9×10^{-6}	5.6×10^{-10}	6.0×10^{-7}
Combined total Risk	2.6×10^{-6}					

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

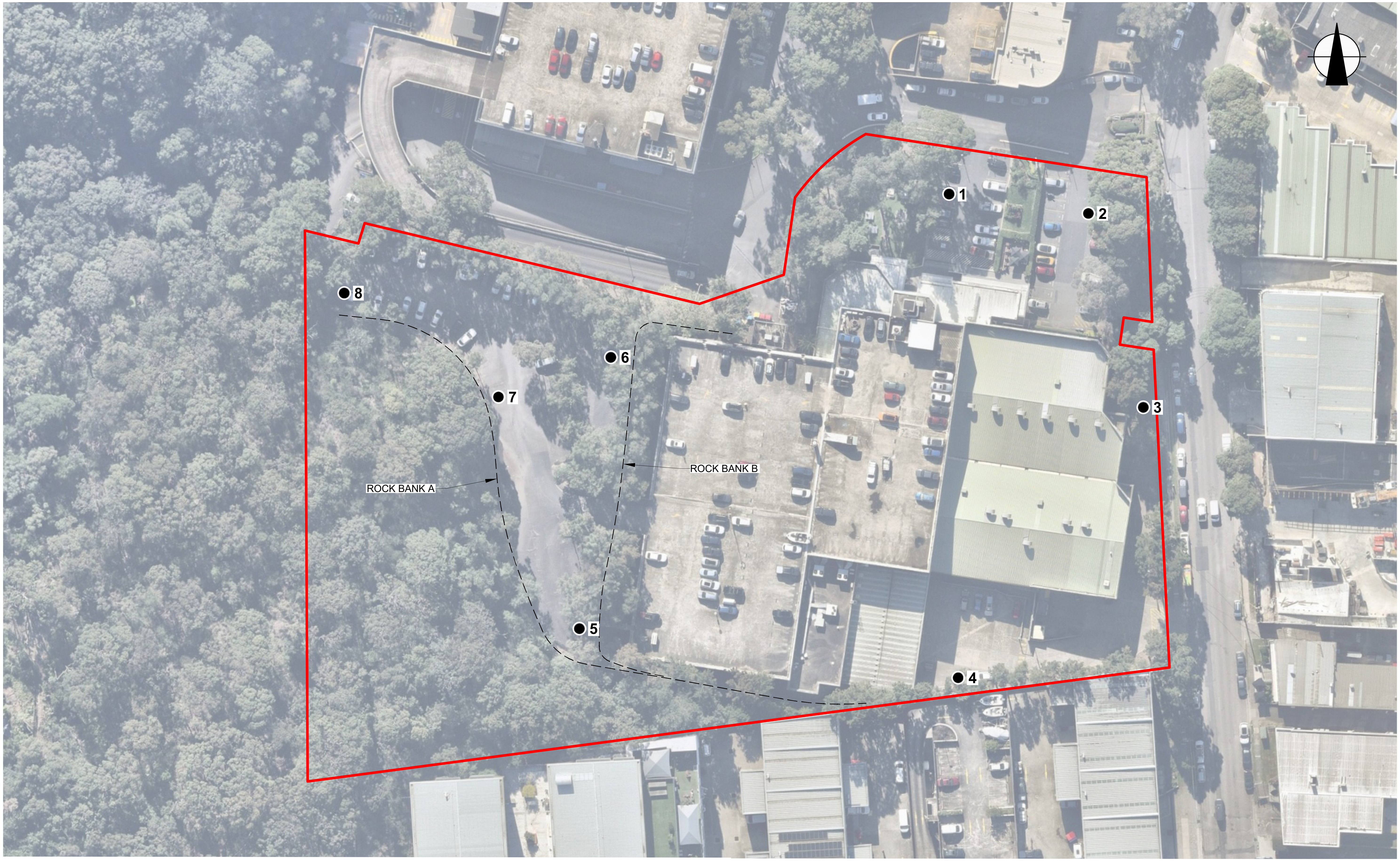
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Report No: 37745SF

Figure No: 1

JKGeotechnics

PLOT DATE: 14/08/2025 1:19:19 PM   DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37745SF BROOKVALE\CAD\37745SF.DWG



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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SCALE 1:800 @A3 METRES

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

Title:

BOREHOLE LOCATION PLAN

Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Report No: 37745SF

Figure No: 2

JKGeotechnics

PLOT DATE: 14/08/2025 1:19:31 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37745SF BROOKVALE\CAD\37745SF.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.

APPENDIX A

BOREHOLE LOG

Borehole No.
1
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 18-4-16
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 18.8m
Datum: AHD
Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	ASPHALTIC CONCRETE: 40mm.t	D			APPEARS WELL COMPACTED
									FILL: Sandy gravel, fine to coarse grained, dark grey, fine to coarse grained sand.				
				▬	N = SPT 10/20mm REFUSAL			-	FILL: Silty sand, fine to medium grained, light brown, trace of fine to coarse grained igneous gravel.				VERY HIGH 'TC' BIT RESISTANCE
						1			CONCRETE				'TC' BIT REFUSAL ON CONCRETE OBSTRUCTION
									END OF BOREHOLE AT 0.7m				
						2							
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
2
1/2

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 19-4-16
Method: SPIRAL AUGER JK350
Logged/Checked by: O.F./F.V.
R.L. Surface: ≈ 17.3m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	ASPHALTIC CONCRETE: 30mm.t	D			APPEARS WELL COMPACTED
									FILL: Sandy gravel, fine to coarse grained igneous, dark grey, fine to coarse grained sand.				
				■	N = SPT 15/90mm REFUSAL				FILL: Silty sand, fine to coarse grained, light brown, with fine to coarse grained sandstone gravel.				
			■			1		SM	SILTY SAND: fine to medium grained, brown and orange brown, with gravel.	D	(MD)		
				■	N = 42 12,16,26						D		
						2		CL	SANDY CLAY: medium plasticity, light brown, orange brown, red brown and light grey.	MC<PL	H		RESIDUAL
				■	N = 20 8,9,11	3							
				■	N = 21 8,8,13	4							
						5			as above, but light grey, with XW seams and fine to coarse grained ironstone gravel.				
								-	SANDSTONE: fine to coarse grained, light grey, with clay seams and iron indurated bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
				■	N > 46 13,24, 22/140mm REFUSAL	6							
						7							

BOREHOLE LOG

Borehole No.
2
2/2

Client: BUNNINGS GROUP & PRIMEWEST FUNDS

Project: PROPOSED WAREHOUSE DEVELOPMENT

Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V

Date: 19-4-16

Method: SPIRAL AUGER
JK350

R.L. Surface: ≈ 17.3m

Datum: AHD

Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■		8			SANDSTONE: fine to coarse grained, light grey, with clay seams and iron indurated bands.	XW-DW	EL-VL		
				■		9			SANDSTONE: fine to medium grained, light grey.	SW	H		MODERATE TO HIGH RESISTANCE
						10 11 12 13 14			END OF BOREHOLE AT 9.4m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/2

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 18-4-16
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 16.6m
Datum: AHD
Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Silty sand, fine to medium grained, dark brown, trace of roots and root fibres.	M			GRASS COVER
				■	N = 34 9,17,17				FILL: Silty sand, fine to coarse grained, brown, with clay, trace of fine to medium grained igneous gravel and steel fragments.				APPEARS WELL COMPACTED
			■			1		SM	SILTY SAND: fine to coarse grained, brown and orange brown, trace of clay and cemented sand nodules and gravel.	M	MD		
				■	N = 15 6,7,8								
						2		SC	CLAYEY SAND: fine to coarse grained, brown, orange brown and red brown.	M	MD		RESIDUAL
				■	N = 18 7,8,10	3		CL	SANDY CLAY: medium plasticity, grey, red brown and orange brown.	MC<PL	H	420 420 430	
				■	N = 37 8,19,18	4 5		SC	CLAYEY SAND: fine to coarse grained, light grey, grey, orange brown and red brown.	M	D		
▼ AFTER 24 HRS				■	N = SPT 19/100mm REFUSAL	6 7		-	SANDSTONE: fine to medium grained, light grey and red brown, trace of L strength bands.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE

BOREHOLE LOG

Borehole No.
3
2/2

Client: BUNNINGS GROUP & PRIMEWEST FUNDS

Project: PROPOSED WAREHOUSE DEVELOPMENT

Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 18-4-16

Method: SPIRAL AUGER
JK350

Logged/Checked by: O.F./F.V.

R.L. Surface: ≈ 16.6m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SANDSTONE: fine to medium grained, light grey and red brown, trace of L strength bands.	DW	VL		
				■					SANDSTONE: fine to medium grained, red brown and brown.	SW	M-H		MODERATE RESISTANCE
				■		8					VH		HIGH RESISTANCE
									END OF BOREHOLE AT 8.1m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						9							
						10							
						11							
						12							
						13							
						14							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 19.3m
Date: 18-4-16
Datum: AHD
Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			CONCRETE: 155mm.t				NO OBSERVED REINFORCEMENT
								-	FILL: Sandy gravel, fine to coarse grained igneous, dark grey, fine to coarse grained sand.	M			APPEARS WELL COMPACTED
				■	N = SPT 14/110mm REFUSAL				FILL: Clayey sand, fine to coarse grained, light brown, with fine to coarse grained sandstone gravel.				
				■		1		-	SANDSTONE: fine to coarse grained, light grey, orange brown and red brown.	SW	L-M		LOW 'TC' BIT RESISTANCE
										SW-FR	H-VH		VERY HIGH RESISTANCE
									END OF BOREHOLE AT 1.6m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						2							
						3							
						4							
						5							
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS

Project: PROPOSED WAREHOUSE DEVELOPMENT

Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 19-4-16

Method: SPIRAL AUGER
JK350

R.L. Surface: ≈ 29.3m
Datum: AHD

Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	ASPHALTIC CONCRETE: 15mm.t	D			
									FILL: Gravelly sand, fine to coarse grained, dark grey, fine to coarse grained igneous gravel.	M			APPEARS MODERATELY COMPACTED
				■	N = 12 8,7,5	1			FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained sandstone gravel and clay.				
									FILL: Silty sand, fine to medium grained, dark brown.				
				■	N = 16 7,8,8	2		SC	CLAYEY SAND: fine to coarse grained, orange brown.	M	MD		RESIDUAL
				■				-	SANDSTONE: fine to coarse grained, orange brown, red brown and brown.	SW	VH		HIGH 'TC' BIT RESISTANCE
						3 4 5 6 7			END OF BOREHOLE AT 2.6m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE

BOREHOLE LOG

Borehole No.
6
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 19-4-16
Method: SPIRAL AUGER JK350
Logged/Checked by: O.F./F.V.
R.L. Surface: ≈ 31.0m
Datum: AHD

Groundwater Record	SAMPLES				Field Tests	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 & AFTER 1 HR						0	▨	-	ASPHALTIC CONCRETE: 15mm.t FILL: Gravelly sand, fine to coarse grained, dark grey, fine to coarse grained igneous gravel.	D			APPEARS WELL COMPACTED
				■	N = 28 11,15,13		▨		FILL: Silty sand, fine to medium grained, light brown, trace of fine to medium grained sandstone gravel.				POSSIBLY COLLUVIUM
						1	▨						
				■	N = 25 11,10,15		▨						
						2	▨						
							⋮		SANDSTONE: fine to coarse grained, brown and orange brown.	SW	M-H		MODERATE 'TC' BIT RESISTANCE
				■		3	⋮						
						4	⋮						
				■			⋮				VH		HIGH RESISTANCE
						5			END OF BOREHOLE AT 4.7m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						6							
						7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
7
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 18-4-16
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 30.7m
Datum: AHD
Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	ASPHALTIC CONCRETE: 30mm.t FILL: Sandy gravel, fine to coarse grained igneous, dark grey, fine to coarse grained sand.	D			APPEARS WELL COMPACTED
					N > 8 8,8/10mm REFUSAL			-	FILL: Silty sand, fine to medium grained, light brown, trace of fine to medium grained sandstone gravel.	XW	EL		VRY LOW 'TC' BIT RESISTANCE
						1			SANDSTONE: fine to medium grained, light grey. as above, but light grey and red brown.	SW	H		HIGH RESISTANCE
						2							
									END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
8
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS
Project: PROPOSED WAREHOUSE DEVELOPMENT
Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V
Date: 18-4-16
Method: SPIRAL AUGER
JK350
R.L. Surface: ≈ 31.3m
Datum: AHD
Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0		-	ASPHALTIC CONCRETE: 30mm.t	D			
									FILL: Sandy gravel, fine to coarse grained igneous, dark grey, fine to coarse grained sand.	M			
AFTER 24 HRS								-	FILL: Silty sand, fine to medium grained, brown and light brown, trace of clay.	SW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE
ON COMPLET-ION						1			SANDSTONE: fine to coarse grained, light grey and orange brown.				
						2			SANDSTONE: fine to medium grained, grey and light grey.		H-VH		HIGH RESISTANCE
									END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						3							
						4							
						5							
						6							
						7							

BOREHOLE LOG

Borehole No.
9
1/1

Client: BUNNINGS GROUP & PRIMEWEST FUNDS

Project: PROPOSED WAREHOUSE DEVELOPMENT

Location: 120 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No. 29340V

Date: 18-4-16

Method: SPIRAL AUGER
JK350

R.L. Surface: ≈ 22.0m

Datum: AHD

Logged/Checked by: O.F./F.V.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	ASPHALTIC CONCRETE: 40mm.t	D			
									FILL: Sandy gravel, fine to coarse grained igneous, dark grey, fine to coarse grained sand.				
				■	N > 4 18,4/10mm REFUSAL				FILL: Silty sand, fine to coarse grained, light brown, with fine to coarse grained sandstone gravel.				APPEARS WELL COMPACTED
								-	SANDSTONE: fine to medium grained, brown and orange brown.	SW	H		HIGH 'TC' BIT RESISTANCE
						1			END OF BOREHOLE AT 1.0m				'TC' BIT REFUSAL ON HIGH STRENGTH SANDSTONE
						2							
						3							
						4							
						5							
						6							
						7							

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

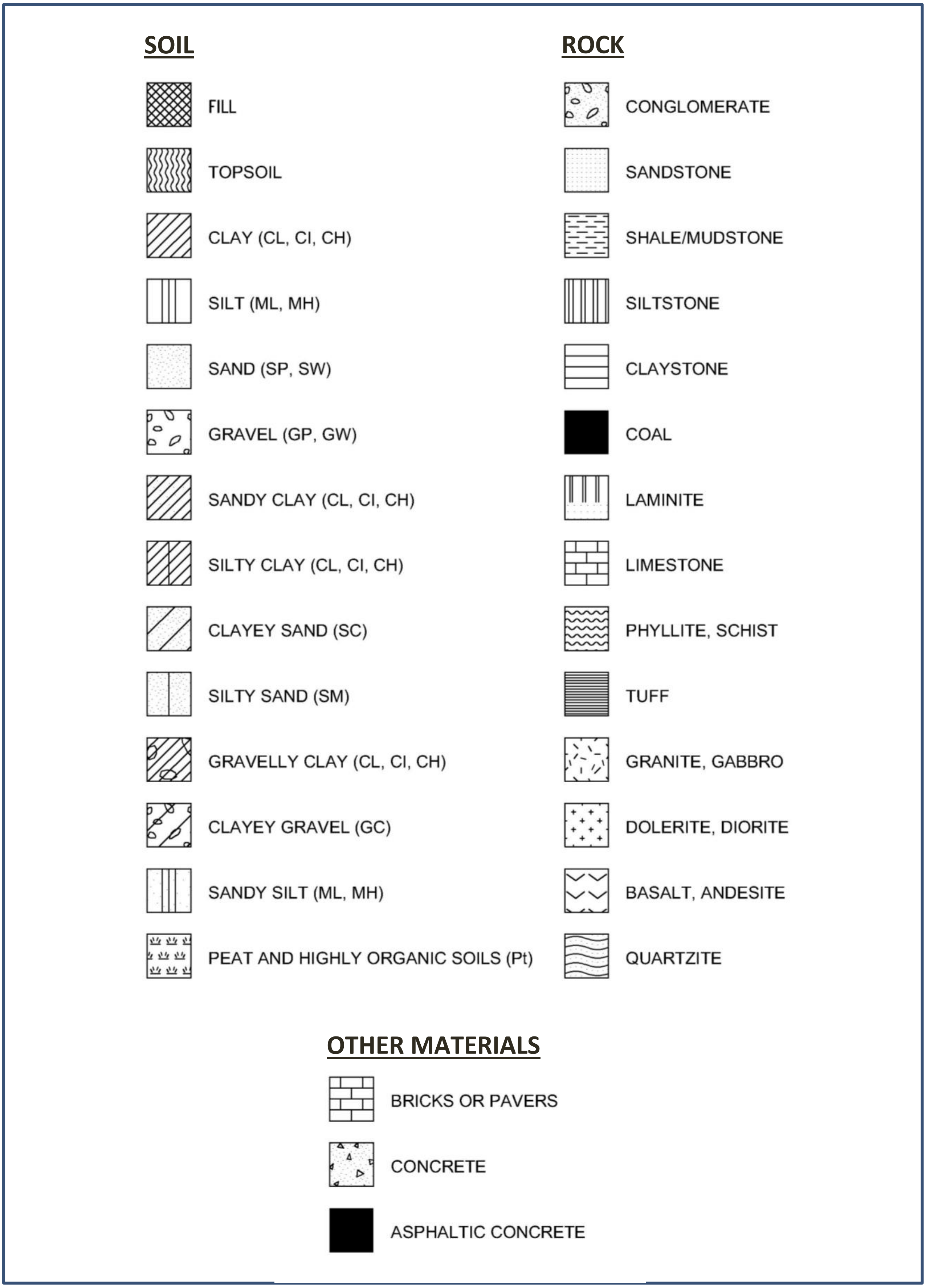
Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS



SOIL

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

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- ■ COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ■ ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory **Classification** Criteria

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

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

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:

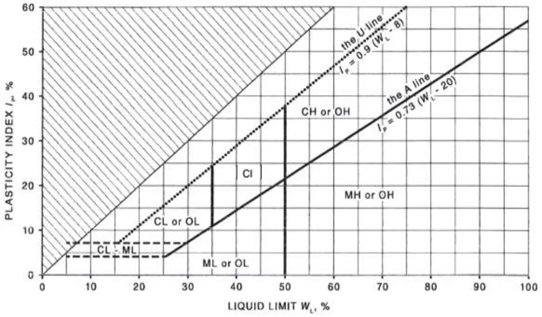
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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