REPORT TO

INTEGRATED PROJECTS

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SENIORS LIVING DEVELOPMENT

AT

55 FOX HILLS CRESCENT, PROSPECT, NSW

Date: 11 December 2020

Ref: 33524SNrpt

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Table of Contents

ATTACHMENTS

STS Table A: Moisture Content, Atterberg Limits & Linear Shrinkage Test Report

STS Table B: Four Day Soaked California Bearing Ratio Test Report

Table C: Point Load Strength Index Test Report

Borehole Logs 1 to 10 Inclusive (With Core Photographs)

Figure 1: Site Location Plan

Figure 2: Borehole Location Plan

Vibration Emission Design Goals

Report Explanation Notes

1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed Seniors Living Development at 55 Fox Hills Crescent, Prospect, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Acceptance of Proposal form signed by Mr David Griffin of Integrated Developments and dated 10 September 2020. The commission was on the basis of our proposal Ref: P52474PN dated 21 August 2020 and our email dated 10 September 2020.

From the supplied unreferenced masterplan drawings prepared by Altis Architecture, we understand the proposed development will occupy the south-west corner of the current Fox Hills Golf Club, and will comprise 14 separate residential apartment buildings over basement carparks of one or two levels, along with a refurbished Golf Club and Facilities building, tennis courts and bowling green, internal roads, and an upgraded on-grade carpark for the golf club.

As specific details for the proposed buildings have not been finalised, the depth of excavation for the basements is unknown, however, excavation to depths between about 3m and 6m below existing levels is expected to be required. As no structural loads have been advised, typical loads for these types of structures have been assumed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at broadly spaced borehole locations, and to use this as a basis for providing preliminary comments and recommendations on excavation conditions, shoring options, retaining wall design, footing design, on grade floor slabs, and recommendations for detailed geotechnical investigations.

This geotechnical investigation 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: E33524BArpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

Ten boreholes, BH1 to BH10, were drilled to depths between 5.8m (BH2) and 7.5m (BH1, BH3 to BH8, and BH10) using spiral augering techniques with our track mounted JK300 and JK305 drill rigs. BH2 and BH9 were subsequently extended to depths of 8.64m and 7.37m respectively using rotary diamond coring techniques with water flush. The compaction of the fill and strength of the natural soils were assessed from the results of Standard Penetration Tests (SPTs) completed in the boreholes, and the results of hand penetrometer tests completed on recovered cohesive soil samples. The strength of the augered bedrock was assessed from observation of the drilling resistance of a tungsten carbide drill bit attached to the augers, tactile examination of recovered rock chips, and correlation with the results of subsequent laboratory moisture content tests. We note that the assessment of strength in this manner is approximate, and variation by about one order of strength should not be unexpected. The strength of the cored bedrock was assessed from tactile examination of the recovered rock core and the results of subsequent laboratory point Load Strength Index ($I_{S(50)}$) tests.

The results of the Point Load Strength Index tests are presented on the attached Table C and are plotted on the cored borehole logs.

Groundwater observations were made during and on completion of drilling each borehole. PVC standpipe piezometers were installed in BH1, BH3 and BH8 to allow for ongoing measurement of groundwater levels, and measurements were made in these piezometers at the end of the drilling program. No longer term groundwater monitoring has been completed.

The borehole locations, as shown on the attached Investigation Location Plan (Figure 2) were set out using a differential GPS, the order of accuracy of which is better than 50mm in all directions. Figure 2 is based on available Nearmap imagery, with the proposed masterplan overlain. The eastings and northings of the boreholes to Map Grid of Australia (MGA) and reduced levels (RL) to Australian Height Datum (AHD) are shown on the borehole logs.

Our geotechnical engineer, Ben Sheppard, was on site full time during the fieldwork and set out the borehole locations, nominated the sampling and testing, directed the standpipe installation, and prepared the borehole logs. The borehole logs are attached to this report along with our Report Explanation Notes which describe the investigation techniques adopted and define the logging terms and symbols used.

Selected soil and rock chip samples were returned to Soil Test Services Pty Ltd (STS), our NATA accredited laboratory, for moisture content, Atterberg Limits, linear shrinkage, Standard compaction, and 4-day soaked CBR tests. The results of the tests are presented on the attached STS Tables A and B.

3 RESULTS OF INVESTIGATION

3.1 Site Description

Most of the golf course is located on the western side, and towards the base, of a north-south trending gulley. The site itself predominantly graded to the east and south at a maximum of about 3°, but with many locally steeper areas due to local cut and fill earthworks. Fox Hills Crescent and Great Western Highway bound the site to the south.

At the time of the fieldwork, the site was a portion of an active golf course (Fox Hills Golf Club) and was predominantly grassed with trees between fairways and around greens/tees. At the southern end of the site was a two-storey clubhouse of concrete construction, which appeared to be in good condition based on a cursory inspection. To the east and south of the clubhouse was an asphaltic concrete surfaced carpark which was in fair condition. Towards the north-west corner of the site was the golf course maintenance depot, with several metal clad sheds. In the north-east corner of the site was a small dam, approximately 50m in diameter, which appeared to have been constructed by excavating below natural surface levels, with no dam wall observed.

To the north and east of the site, the golf course continued.

To the west of the site were numerous residential properties with buildings of 1, 2 and 3 stories, some of which abut the site boundary.

3.2 Subsurface Conditions

The 1:100,000 geological Map of Penrith indicates the site is underlain by Bringelly Shale of the Wianamatta Group, however, alluvial soils overlie the Bringelly Shale along the base of the gully, i.e. just to the east of the site.

The boreholes disclosed a subsurface profile comprising topsoil and fill to moderate depth, over predominantly residual soils, then weathered siltstone (shale) bedrock. For details of the encountered subsurface profile, reference should be made on the attached borehole logs. A summary of the encountered conditions is presented below.

Fill

Fill, predominantly comprising clays but with some sands and gravels also present, was encountered from the surface in all of the boreholes and extended to depths ranging from 0.2m (BH6) to 3.3m (BH9), and was assessed as being poorly to well compacted where tested. The upper fill in all of the boreholes was a topsoil material.

Natural Soils

Alluvial silty clay of high plasticity was encountered below the fill in BH1 only, to a depth of 3.7m. Residual silty clay of high plasticity was encountered below the fill in all of the borehole except BH1, where it was encountered below the alluvial clay. The clays were predominantly of very stiff and hard strength, however, clays of stiff or stiff to very stiff strength were encountered in BH1, BH5, and BH8.

Siltstone Bedrock

Siltstone bedrock with some interbedded sandstone was encountered in all of the boreholes, from depths between 1.2m (BH6 and BH8) and 4.7m (BH1), and extended to the borehole termination depths. The siltstone bedrock was predominantly extremely weathered and of hard (soil) strength on first contact, and improved to between very low and medium to high strength with depth. Over the cored portion of BH2 and BH9, numerous defects, including extremely weathered seams and inclined joints, were encountered.

Groundwater

All of the boreholes were ‘dry’ during and on completion of auger drilling. Whilst standing water was measured in BH9 on completion of coring, as water is injected into the borehole during coring, this is not considered representative of natural groundwater. In the standpipes installed in BH1, BH3 and BH8, standing water was measured at depths of 7.2m, 7.4m, and 3.8m between 2 hours and about 24 hours following installation.

3.3 Laboratory Test Results

The results of the moisture content tests correlated well with our field assessment of the bedrock strength. The results of the Atterberg Limits tests confirmed the clayey fill and natural clays to be of high plasticity ,and the linear shrinkage tests indicated the clayey soils to be highly reactive to moisture content change.

The 4 day soaked CBR tests returned CBR values of between 1.5% and 3% for the clayey soils, when compacted to 98% of their respective Standard Maximum Dry Density (SMDD) at close to their Standard Optimum Moisture Content (SOMC). The in-situ moisture content of the clays was between 2.2% 'dry' and 10.3% 'wet' of their respective SOMC. The clayey soils swelled by between 1% and 3% during soaking which also indicates the clays are reactive to moisture content change.

The results of the Point Load Strength Index tests correlated well with our field assessment of the bedrock strength.

4 COMMENTS AND RECOMMENDATIONS

As the design for the proposed development has not yet been finalised, the comments and recommendations which follow are general in nature. Further, whilst predominantly consistent subsurface conditions have been encountered, the boreholes have been drilled at large centres, and variability could occur between the boreholes

Once the planning has been completed, but prior to detailed structural design, supplementary investigations must be completed with at least 2 boreholes over the footprint of each building, possibly more if higher bearing pressures are desired for footing design. Following completion of the additional drilling, the comments and recommendations in the following sections of this report must be reviewed and updated if required. We can complete the supplementary investigation(s) if commissioned to do so.

4.1 Geotechnical Opinion

The subsurface conditions encountered in the boreholes are considered 'typical' for the region, and no particular issues of geotechnical concern were identified. From a geotechnical perspective, we do not anticipate any non-standard design or construction practices will be required to facilitate the proposed development, and we consider the site to be geotechnically suitable for the proposed development.

4.2 Site Preparation and Earthworks

The following recommendations are applicable for areas where on-grade buildings, floor slabs, and pavements are proposed. Where basements are proposed, no particular site preparation would be required.

The following comments and recommendations must be complemented by reference to AS3798-2007: 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.2.1 Dilapidation Surveys

Prior to any earthworks or excavation commencing within 20m of existing structures, we recommend that detailed dilapidation reports be prepared for such structures. The dilapidation surveys should comprise a detailed inspection both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of the existing conditions. We note that Council may also require that dilapidation reports be prepared for adjoining Council assets.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works.

4.2.2 Existing Fill

The existing fill was variably compacted, including portions which were poorly compacted. No documentation on the fill placement or compaction is available, and given the nature of the site, we anticipate the fill was most likely placed in an uncontrolled manner. As such, the fill cannot be considered to be engineered fill, and we consider it is unsuitable to support any new buildings or pavements. Where new buildings or pavements are proposed, the fill must be stripped off across the entirety of the building/pavement footprint, and for a minimum 2m envelope around the building/pavement footprint.

4.2.3 Site Drainage and Stripping

The clayey fill and residual clay materials at the site are expected to undergo substantial loss in strength when wet, as evident from the low CBR values and high swells during soaking. Furthermore, the clay subgrade is expected to have a high shrink-swell reactive potential. Therefore, 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 untrafficable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

Good surface and subsurface drainage must also be provided post construction to improve the long-term performance of the structures and pavements.

4.2.4 Site Preparation and Minor Excavation

Any grass and other vegetation, as well as any topsoil or root affected soils must be stripped from the site.

Stripped topsoil/root affected soils must be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes. Reference should be made to the JKE report for guidance on the offsite disposal of soil.

Where required, excavation may then be carried out to achieve design subgrade levels, or to the base of the existing fill below proposed building and pavement footprints (whichever is the deeper). Based on the investigation results and an assumed maximum depth of excavation of about 1m, the excavations are expected to encounter the soil profile only and are expected to be readily completed using buckets fitted to hydraulic excavators, or blades on dozers, graders and scrapers. The upper extremely weathered bedrock, if encountered, is also expected to be readily excavated using such equipment.

Temporary batter slopes through the soil profile appear be feasible and should be cut no steeper than 1 Vertical in 1 Horizontal for a maximum height of 4m, and provided all surcharge loads are kept at least 2m clear of the crest of such batters. Where the excavation depth exceeds 4m, a horizontal mid-slope bench at least 1.5m wide must be adopted. We note that where poorly compacted fill is present, localised slumping of such batter slopes may occur. Where temporary batters cannot be accommodated within the site geometry, or are not preferred, then further geotechnical advice should be sought regarding the use of a shoring system.

Any permanent batter slopes should be graded no steeper than 1V in 2H, provided the slopes are protected from erosion by quickly establishing a vegetative cover or by applying a reinforced shotcrete facing, together with surface drains at the crests of the batters to intercept surface water flows and prevent them from flowing over the face. Where access for mowing etc. is required, permanent batters will likely need to be flattened to about 1V in 4H.

4.2.5 Basement Excavation

Excavation for the proposed basements is expected to extend to a maximum depth of about 6m below existing levels, with locally deeper excavations required for lift overrun pits and services. Excavation to such depths will extend through the fill and natural soil profiles then into the underlying siltstone bedrock.

Excavation of the fill and natural soils, as well as any extremely low and very low strength bedrock is expected to be readily achievable using conventional techniques such as the buckets of medium to large sized hydraulic excavators.

Excavation through bedrock of low or greater strength could likely be achieved using ripping tynes on a large excavator (at least 30 tonne in size) and or ripping with a large dozer (D9 or larger). If rock of high strength is encountered, it would require the use of rock breaker attachments to large excavators, or hard ripping with D11 size dozers. Grid sawing techniques with ripping and/or hammering could also facilitate excavation of the higher strength bedrock.

4.2.6 Subgrade Preparation

Following excavation to design levels, the exposed soil subgrade should be proof rolled with at least eight passes of a smooth drum roller of at least 12 tonnes deadweight. Proof rolling would not be required where a siltstone bedrock subgrade is present. The final passes of proof rolling should be witnessed by an experienced geotechnical engineer or earthworks superintendent for the detection of unstable or soft areas.

If soft or heaving areas are detected during proof rolling, then the heaving areas should be locally removed to a stable base and replaced with engineered fill, as outlined in Section 4.2.7 below, or further geotechnical advice should be sought. Further guidance on the treatment of heaving areas must be provided by the geotechnical engineer during or following the proof rolling inspection. Based on the investigation results, heaving may occur where the in-situ soil moisture content is elevated, or in areas of the site where water has ponded on the ground surface), assuming the earthworks are not carried out immediately following a period of wet weather. In the vicinity of and downslope of the dam, elevated soil moisture contents may occur

If soil softening occurs after rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. Conversely, if the clayey subgrade exhibits shrinkage cracking, then the surface should be lightly watered and rolled until the shrinkage cracks are no longer evident.

4.2.7 Engineered Fill

Preferably, engineered fill should comprise an imported well graded granular material, e.g. crushed sandstone, with a maximum particle size not exceeding 75mm. Such materials are less susceptible to softening that clayey soils, and have reduced reactive movement to moisture content change. From a geotechnical perspective, the existing fill materials, which were predominantly clayey, residual soils and any excavated bedrock are also considered suitable for reuse as engineered fill on condition that they are ‘clean’, free of organic matter and contain a maximum particle size not exceeding 75mm. Any imported fill must also have a maximum particle size not exceeding 75mm.

All clayey fill, including excavated weathered siltstone bedrock, should be compacted in maximum 200mm thick loose layers to a density ratio strictly between 98% and 102% of SMDD and within 2% of SOMC. Given the moisture contents of the materials on site, some moisture conditioning of the site won clay soils will be required in order to meet this specification. Engineered fill comprising well graded granular materials, such as imported crushed sandstone, should be compacted in maximum 200mm thick loose layers to achieve a density ratio of at least 98% of SMDD.

In order to achieve adequate compaction at the edge of fill platforms, the outer edge of each fill layer should extend a horizontal distance of at least 1m beyond the design geometry. The roller must extend over the edge of each placed layer in order to seal the batter surface. On completion of filling, the excess under-compacted edge fill should be trimmed back to the design geometry.

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of the compaction plant that can be placed in trenches, backfilling should be carried out in maximum 100mm thick loose layers and compacted using a trench roller, a pad foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a ‘Wacker Packer’). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also be reduced to not more than 50mm. The compaction specifications provided above are applicable.

As for services trenches, retaining wall backfilling must also be carried out using engineered fill in order to reduce post-construction settlements. Compaction of the engineered backfill should be carried out using a trench roller or hand operated vertical rammer compactor for the lower layers and immediately behind the wall in the upper layers. Elsewhere a small static roller could be used. As per service trenches, backfilling should be carried out in 100mm thick loose layers and the maximum particle size of the backfill material should be reduced to not more than 50mm. The compaction specifications provided above are applicable.

Compaction of engineered fill behind retaining walls is difficult and not commonly undertaken. The more common method comprises using a single sized hard, durable free draining aggregate, such as 'Blue Metal' or crushed concrete aggregate (free of fines, brick and tile fragments), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion into the gravel. Provided the aggregate backfill is placed as recommended above, density testing would not be required in that material. The geotextile should then be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of clayey engineered fill, to reduce the potential for surface water to enter the retaining wall drainage.

In-situ density tests must be carried out on the engineered fill to confirm the above specifications are achieved, as outlined below:

- The frequency of density testing for general engineered fill on a large scale lot should be at least one test per layer per 2,500m^2 or one test per 500m^3 distributed reasonably evenly throughout the full depth and area, and at least 3 tests per earthworks lot (as defined in Clause 1.2.8 of AS3798-2007), whichever requires the most tests (assumes maximum 200mm thick loose layers).
- The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 100mm thick loose layers), with the tests fully penetrating both layers.
- The frequency of density testing for retaining wall backfill should be at least one test per two layers per 50m^2 (assumes maximum 100mm thick loose layers), with the tests fully penetrating both layers.

As the fill will support pavements and may support buildings, we recommend that Level 1 control of fill placement and compaction in accordance with AS3798-2007 be carried out, including for the trench backfill. Due to a potential conflict of interest, we strongly recommend that the geotechnical inspection and testing authority (GITA) be directly engaged by the client or builder, and not by the earthworks contractor or sub-contractors.

4.2.8 Earthworks Testing Overview

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. This should be detailed in the tender documents.

We also recommend that the GITA be requested to provide a summary of test results, including a test location plan, and daily site reports on a fortnightly basis for review by the Project Superintendent and/or JK Geotechnics. On completion of the earthworks, the GITA should be requested to provide a Level 1 'sign off' report for our review, including a statement that the fill has been placed and compacted in accordance with the specification.

4.2.9 Potential Vibration Risks

When vibratory compaction or rock excavation is to be completed within 20m of the residential properties to the west, or any other structures, we recommend that continuous vibration monitoring be carried out during vibratory compaction, piling, and excavation works. Provisionally, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec for nearby houses, as well as for any other structures, subject to the structural engineers review of the dilapidation survey report(s). If higher vibrations are recorded, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be tolerable depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use a smaller plant or alternative techniques.

For all areas within 20m of where vibration damage is of concern, excavation with hydraulic rock hammers should be completed using a hydraulic excavator fitted with a moderate energy hydraulic rock hammer no larger than say, a Krupp 900 size or equivalent. In addition, a vertical saw cut slot should be provided along the perimeter of the excavation and the base of the slot maintained at a lower level than the adjoining rock excavation at all times.

The use of a rotary grinder or grid sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

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

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.

- When operating more than one hammer at a time, operate hammers in different areas of the site and in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

4.2.10 Seepage

Standing water was encountered within the bedrock profile during our investigation, however, this was predominantly below the expected level of the proposed basements. Additional seepage may be encountered in the vicinity of the dam.

Based on the above, we consider that design and construction of drained basements is most appropriate for the proposed development, however, additional investigation will be required to confirm if this is the case during the detail design phase. Following the measurement of groundwater inflows at the completion of excavations, an accurate estimate of long term groundwater inflows will be made to assess long term storage and pumping requirements (if any). This assessment may have to be delayed until basement construction is complete if persistent wet weather occurs as rainfall inflows into the open excavation will vastly exceed likely seepage volumes and the use of water in construction will further complicate assessment.

Pumping of groundwater from the basement should not result in significant drawdown of groundwater in the vicinity of the site, as the groundwater is within the bedrock profile. We therefore consider draining the basement will have a negligible impact on any nearby structures or infrastructure.

4.3 Retaining Walls

4.3.1 Basement Batter Slopes and Shoring

Temporary batter slopes through the soil and weathered bedrock profiles appear be feasible and should be cut no steeper than 1 Vertical in 1 Horizontal for a maximum height of 4m, and provided all surcharge loads are kept at least 2m clear of the crest of such batters. Where the excavation depth exceeds 4m, a horizontal mid-slope bench at least 1.5m wide must be adopted. We note that where poorly compacted fill is present, localised slumping of such batter slopes may occur. In areas where such batter slopes are not feasible, or are not preferred, excavations will need to be supported by engineer designed shoring systems installed prior to individual excavations commencing. We note that in areas where battered excavations are adopted for basement constructions, backfilling of the basement shoring walls with engineered fill, as detailed in Section 4.2.7 above, would need to be completed after the walls are constructed.

Whilst some of the low strength siltstone bedrock may be suitable to stand unsupported in the short term, given the limited depth of excavation anticipated, we consider the most feasible solution would be for the shoring system to extend to bulk excavation level.

The effect of ground movements on any structures and services that lie within the influence zone of the excavation must also be taken into account. The influence zone of excavations may be defined as a horizontal distance of 2H (where 'H' is the depth of the excavation in metres) behind shoring walls. A suitable retention system for the proposed basements would comprise soldier pile walls with reinforced shotcrete infill panels. Conventional bored piles are considered suitable for use on this site.

For shallower basements (i.e. about 3m excavation depth) which are not located close to any movement sensitive structures or services, cantilevered shoring walls would likely be feasible. However, for deeper basements (i.e. greater than about 4m excavation depth), or where movements are to be limited, the shoring systems will need to be anchored or braced as the excavation progresses. Approval from neighbouring land owners would be required prior to the installation of anchors into their property (if required). If permission cannot be obtained to install anchors, it would be possible to use internal braces instead.

Drilling of rock sockets will be difficult through medium and greater strength rock, if required, and will require the use of high capacity drilling rigs equipped with rock augers and possibly coring buckets.

Given the expected depth of shoring piles, pouring using tremie methods is recommended, particularly if groundwater inflow occurs into bored pile holes such that the piles would be tremie poured as 'wet' piles.

4.3.2 Lateral Earth Pressures

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of temporary or permanent retention systems.

Free standing cantilever walls of no more than about 3m height, where minor wall movements are tolerable, should be designed using a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient (K_a) of 0.35 for the soil and bedrock profiles, as well as for any backfill materials, assuming a horizontal retained surface.

Cantilever walls which will be propped or restrained by structures and subsequently backfilled, or where wall movements are to be limited, should be designed using a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient, K_0, of 0.5 for the soil and bedrock profiles, as well as for any backfill materials, assuming a horizontal retained surface.

For anchored or propped soldier pile walls where minor movements can be tolerated, e.g. landscaped areas or similar, we recommend the use of a trapezoidal earth pressure distribution with a maximum lateral pressure of 4HkPa for the soil and bedrock profiles, where 'H' is the retained height in metres. These pressures should be assumed to be uniform of the central 50% of the support system, tapering to zero at the crest and toe. Where movements are to be limited, e.g. where neighbouring structures or movement sensitive services are located within 2H of the wall, the maximum lateral pressure should be increased to 8HkPa.

A bulk unit weight of 20kN/m^3 should be adopted for the soil and bedrock profiles.

Any surcharge affecting the walls (e.g. traffic loading, construction loads, adjacent high level footings, etc.) should be allowed for in the design using the above 'at-rest' earth pressure coefficient.

All retaining walls should be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

Lateral toe restraint of low height cantilever walls may be achieved by passive resistance of the soil profile in front of the wall using a triangular lateral earth pressure distribution and a 'passive' earth pressure coefficient, K_P, of 3.0 for the soil profile. We note that significant movement is required in order to mobilise the full passive pressure of a soil, and therefore a factor of safety of at least 2 should be adopted to reduce such movements. Any localised excavations in front of the wall should be taken into account in the embedment design. Friction on the base of a wall founded in the natural soil or bedrock profiles can be calculated using a friction angle of 30˚ between the retaining wall base and the soil/bedrock below, provided the base is clean, rough and dry when the retaining wall footing is poured.

Lateral toe restraint of the shoring walls extending below basement excavation level may also be achieved by embedding the piles into the bedrock below bulk excavation level. An allowable lateral resistance of 200kPa can provisionally be adopted for bedrock over very low or greater strength, though the upper 0.5m of socket must be ignored to allow for disturbance or possible over excavation. For piles embedded into bedrock below bulk excavation level, a minimum embedment depth (ignoring the 0.5m allowance above) of 1.0m should apply. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, tanks, buried services, etc. must be taken into account in the wall design. Any soil above the bedrock profile must be ignored when calculating the lateral resistance of such piles due to strain incompatibility between the soil and bedrock profiles.

Anchors bonded into consistent very low or greater strength bedrock may be provisionally designed on the basis of an allowable bond stress of 100kPa. All anchors should be proof loaded to at least 1.3 times their working load and then locked off at approximately 85% of their working load. Proof loading should be carried out in the presence of an engineer independent of the anchor contractor. Anchors must be bonded behind a line drawn up at 45° from the base of the excavation, with all anchors having a free length and bond length of at least 3m each. Lift off tests should be carried out on at least 25% of all anchors about 4 days after initial lock off to confirm that they are maintaining their load. Anchor design and construction is best managed as a design and construct sub-contract to avoid disputes if anchors fail test load.

Alternatively, retaining walls could be designed using computer based soil structure interaction analysis methods (e.g. Wallap or Plaxis), which could result in cost savings compared to a design based on the above simplified earth pressure assumptions. Analysis software treating the soil as 'equivalent springs' should not be used for this design. Analysis using soil structure interaction methods can model the actual excavation stages, including progressive anchoring/shoring, and outputs include structural actions in the piles, anchor/prop loads, and wall movements. The analysis should be completed by an engineer with a good understanding of soil-structure interaction behaviour, including an understanding of when soil wall friction should and should not be used etc. We can provide appropriate soil and bedrock parameters for such analysis if requested, or undertake the analysis on behalf of the wall designer.

4.4 Footing Design

Following completion of basement excavation weathered siltstone bedrock is expected to be exposed at basement excavation level across the majority of the basements. We therefore recommend that all structures be supported on footings founded in the bedrock profile for uniformity of support.

Where the bedrock is exposed or is at shallow depth conventional pad or strip footings could be adopted. Where the bedrock is at greater than about 1m depth, piles would be required. Conventional bored piles are expected to be feasible for the subject site, however, allowance must be made for pumping pile holes dry before pouring concrete and pouring using tremie techniques.

Pad or strip footings, or piles socketed a nominal 0.3m into bedrock of at least very low strength may be designed for an allowable end bearing pressure of 700kPa, based on serviceability criteria. For pile rock sockets longer than this 0.3m, an allowable shaft adhesion of 70kPa in compression and 35kPa in tension (uplift) may be adopted provided the socket is satisfactorily cleaned and roughened. For pad/strip footings founded on and piles socketed into consistent low or greater strength bedrock, the above values could be increased to 1,000kPa, 100kPa and 50kPa respectively. Higher bearing pressures, possibly to about 3,000kPa may be achievable following further investigation with cored boreholes.

For any on-grade buildings, or portions of buildings, supported on footings to the bedrock profile, void formers must be incorporated below any on grade suspended elements, e.g. ground beams, floor slabs (if suspended) etc. to protect such elements against uplift pressures resulting from swelling of the clayey soils. Such void formers should be suitable for swell movements of at least 75mm.

For pad/strip footings and piled footings founded within the weathered siltstone bedrock, representative footings/piles, say one in five, should be inspected by a geotechnical engineer to confirm that the appropriate founding material is being achieved. Where bearing pressures in excess of 1,000kPa are proven feasible and are adopted, an increased inspection regime of a higher proportion of piles and possible spoon testing of pad footings would be required. The details of such inspection should be determined following the completion of further investigations with cored boreholes. The builder must be responsible to ensuring pier holes are free of loose or softened material or standing water prior to pouring concrete.

4.5 Basement On-Grade Floor Slabs

The majority of the basement on-grade floor slabs are expected to directly overlie bedrock and no particular subgrade preparation will be required. Where a soil subgrade is exposed, the soil should be proof rolled with at least 6 passes of a minimum 5 tonne dead weight smooth drum roller, with the final pass of proof rolling inspected by a geotechnical engineer. Should any soft or heaving areas be identified, then additional treatment would be detailed by the engineer at the time of the inspection.

Slab-on-grade construction is considered appropriate. Underfloor drainage, comprising a washed single size aggregate, must however be provided. Such a layer would also act as a separation between the bedrock and floor slab. For all buildings, the underfloor drainage should connect with the wall drains, where appropriate, and direct groundwater seepage to a sump(s) for pumped disposal to a stormwater system after obtaining authority approval, unless gravity drainage is feasible. Joints in the on-grade floor slabs should incorporate dowels or keys. A joint must be incorporated at, the change from a soil to a bedrock subgrade to accommodate potential differential movements.

4.6 Pavements

For external pavements, provided the subgrade has been prepared in accordance with recommendations described in Section 4.2 above, a CBR value of 1.5% can be adopted for design, or, a short term Young's Modulus of 15MPa.

Subgrade improvement comprising the placement and compaction imported crushed sandstone or lime stabilisation of the upper 0.3m of a clay subgrade could be adopted to reduce the thickness of the granular and bound pavement layers. The thickness and quality of such a select layer, if adopted, should be taken into account in the pavement design. As a guide, assuming a minimum of 0.3m thickness of crushed sandstone or lime stabilised clay with a soaked CBR of at least 10%, over clayey fill materials, the equivalent subgrade design CBR value would increase to at least 4%, however, this must be confirmed.

Select fill, if adopted, must comprise a well graded, granular crushed sandstone (maximum particle size of 150mm), or similar material, with a soaked CBR value of at least 10%. If the available sandstone is assessed by tactile examination or laboratory testing to be a marginal material (i.e. achieving a CBR value of just over 10% at a compaction density ratio of 100% of SMDD), then we expect that it will break down and degrade during compaction with a heavy roller to a material with an "insitu" CBR value less than 10%. As such, we recommend that the CBR testing allow for the degradation of the crushed sandstone. The standardised RTA Specification T102 method, which attempts to replicate the degradation process by pre-treatment of the crushed sandstone with 3 cycles of repeated compaction, would be appropriate, or placement and compaction of the material in a test pad prior to sampling. All crushed sandstone select fill should be compacted in maximum 200mm thick loose layers to at least 100% of SMDD.

We recommend that all base course materials for flexible pavements and sub-base materials for rigid pavements comprise DGB20 in accordance with TfNSW QA Specification 3051 unbound base. The DGB20 material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 98% of Modified Maximum Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement. For rigid pavements a recycled DGB20 product would be considered appropriate.

We further recommend that all sub-base materials for flexible pavements comprise DGS40, DGS20 or DGB20 in accordance with TfNSW QA Specification 3051. Recycled materials may be used provided they conform to the specification requirements of 3051. If the recycled materials contain brick or ceramic fragments, it is highly unlikely that they will conform to the specification requirements. The subbase material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 98% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement.

The final pavement material and compaction specification must be determined by the pavement designer.

Density tests should be carried out on the granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 50 linear metres of road; three tests per lot and three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction in accordance with AS3798-2007 would be considered acceptable for the pavement layers. The geotechnical testing authority (GTA) should be directly engaged by the builder and not by the earthworks contractor or sub-contractors.

Subsoil drains should be provided below the perimeter of the proposed pavements, including any internal planters etc. with invert levels at least 200mm below subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.7 Further Geotechnical Input

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

- Completion of a supplementary investigation and review/updating of the comments and recommendations in this report as required;
- Laboratory testing of imported select fill to confirm material properties;
- Laboratory lime stabilisation trials;
- Proof rolling inspections;
- Density testing of all engineered fill, sub-base and base course materials;
- Geotechnical inspection of footing excavations/pile drilling; and
- Witnessing of anchor stressing and lift off testing.

5 SALINITY

The site is located in an area where soil and groundwater salinity may occur. Salinity can affect the longevity and appearance of structures as well as causing adverse horticultural and hydrogeological effects. The local council has guidelines relating to salinity issues which should be checked for relevance to this project.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The 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.

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Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A

MOISTURE CONTENT, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Seniors Living Development
Location: 55 Fox Hills Crescent, Fox Hills Golf Club, Prospect, NSW

Ref No: 33524PN
Report: A
Report Date: 27/11/2020
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	0.60 - 0.95	31.1	77	22	55	16.5
1	7.00 - 7.50	11.4	-	-	-	-
2	2.50 - 3.00	8.8	-	-	-	-
2	3.80 - 4.30	7.8	-	-	-	-
3	1.50 - 1.95	24	58	18	40	12.5
3	5.30 - 6.00	5.9	-	-	-	-
4	3.80 - 4.30	4.7	-	-	-	-
4	6.00 - 6.80	7.0	-	-	-	-
5	3.80 - 4.30	10.6	-	-	-	-
5	5.30 - 6.00	8.5	-	-	-	-
6	0.50 - 0.95	29.2	77	23	54	14.5*
6	2.50 - 3.00	10.0	-	-	-	-
6	5.50 - 6.00	5.8	-	-	-	-
7	4.20 - 4.50	10.1	-	-	-	-
7	5.50 - 6.00	7.7	-	-	-	-
8	5.40 - 6.00	8.1	-	-	-	-
9	3.50 - 4.00	10.0	-	-	-	-
9	5.70 - 5.80	8.4	-	-	-	-
10	0.50 - 0.95	19.8	61	20	41	14.0
10	4.00 - 4.50	10.3	-	-	-	-
10	5.50 - 6.00	4.1	-	-	-	-

Notes: * Denotes Linear Shrinkage curled.
The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 11/11/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

26/11/2020
Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Seniors Living Development
Location: 55 Fox Hills Crescent, Fox Hills Golf Club, Prospect, NSW

Ref No: 33524PN
Report: B
Report Date: 30/11/2020
Page 1 of 1

BOREHOLE NUMBER		BH 1	BH 3	BH 4	BH 10
DEPTH (m)		0.60 - 1.50	0.20 - 0.85	0.70 - 1.50	0.30 - 1.10
Surcharge (kg)		9.0	9.0	9.0	9.0
Maximum Dry Density (t/m^3)		1.71 STD	1.77 STD	1.72 STD	1.76 STD
Optimum Moisture Content (%)		20.4	17.1	19.4	16.4
Moulded Dry Density (t/m^3)		1.67	1.74	1.68	1.73
Sample Density Ratio (%)		98	98	98	98
Sample Moisture Ratio (%)		101	99	100	100
Moisture Contents					
Insitu (%)		30.7	21.5	17.2	19.9
Moulded (%)		20.5	17.0	19.4	16.4
After soaking and After Test, Top 30mm(%)		31.4	29.5	31.6	27.1
	Remaining Depth (%)	22.9	20.4	23.2	25.7
Material Retained on 19mm Sieve (%)		0	0	0	0
Swell (%)		2.5	3.0	3.0	1.0
C.B.R. value:	@2.5mm penetration	3.0	2.0	1.5	3.0

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 11/12/2020.
- BH 1 & 3 dried back prior to compaction as sample was too wet to test.
- Insufficient sample provided for BH4 only 3 point compaction curve.

NATA
NATA Accredited Laboratory Number:1327

Approved Signatory / Date
(D. Treweek) 30/11/20

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Integrated Projects	**Ref No:**	33524PN
Project:	Proposed Seniors Living Development	**Report:**	C
Location:	55 Fox Hills Crescent, Fox Hills Golf Club, PROSPECT, NSW	**Report Date:**	11/11/20
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	6.15 - 6.18	0.7	14	A
	6.45 - 6.49	1.1	22	A
	6.84 - 6.88	0.5	10	A
	7.15 - 7.17	0.6	12	A
	7.45 - 7.48	0.9	18	A
	7.65 - 7.67	0.7	14	A
	8.20 - 8.22	0.2	4	A
	8.50 - 8.54	0.4	8	A
9	5.97 - 6.00	0.5	10	A
	6.26 - 6.29	0.06	1	A
	6.60 - 6.64	0.6	12	A
	7.15 - 7.19	0.6	12	A

NOTES

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

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BOREHOLE LOG

Borehole No.
1
1 / 2

EASTING: 308579.496
NORTHING: 6257503.54

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 48.78 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			0		FILL: Sandy clay topsoil, medium plasticity, dark brown and brown, trace of silt, root fibres and roots.	w>PL			GRASS COVER APPEARS POORLY COMPACTED
	N = 6 2,3,3	48			FILL: Silty clay, high plasticity, orange brown, trace of fine to medium grained ironstone gravel, ash, roots and root fibres.				
	N = 8 2,3,5	47	1.5	CH	Silty CLAY: high plasticity, grey and red brown, trace of fine grained ironstone gravel and root fibres.	w>PL	St	140 150 150	ALLUVIAL
	N = 16 5,6,10	46	2.5		Silty CLAY: high plasticity, grey and red brown, with fine to coarse grained ironstone gravel, trace of root fibres.		VSt - Hd	440 400 410	
		45	3.7		Silty CLAY: high plasticity, grey mottled brown, trace of iron indurated bands.				RESIDUAL
	N = 24 7,12,12	44	4.7	-	Extremely weathered siltstone: silty CLAY, high plasticity, light grey, with fine grained sandstone and iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
	N > 13 17,13/ 100mm REFUSAL	43	6		Extremely weathered siltstone: silty CLAY, high plasticity, grey, trace of iron indurated bands and very low strength bands.				
		42			SILTSTONE: dark grey, with extremely weathered bands and iron indurated bands.	DW	VL		

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BOREHOLE LOG

Borehole No.
1

EASTING: 308579.496
NORTHING: 6257503.54

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 48.78 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON 10/11/20									-	SILTSTONE: dark grey, with extremely weathered bands and iron indurated bands. *(continued)*	DW - XW	VL - Hd		
						41	8			END OF BOREHOLE AT 7.50 m				GROUNDWATER MONITORING WELL INSTALLED TO 7.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 7.5m TO 2.0m. CASING 2.5m TO 0.2m. 2mm SAND FILTER PACK 7.5m TO 2.0m. BENTONITE SEAL 2.0m TO 1.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						40	9							
						39	10							
						38	11							
						37	12							
						36	13							
						35								

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BOREHOLE LOG

Borehole No. 2

1 / 2

EASTING: 308513.926
NORTHING: 6257604.282

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 54.94 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					FILL: Silty sand topsoil, fine to medium grained, dark brown, with roots and root fibres.	M			GRASS COVER
	N = 10 4,4,6	54	1	CH	Silty CLAY: high plasticity, orange brown, grey and red brown, trace of fine to coarse grained ironstone gravel, ash and roots.	w~PL w<PL	Hd	>600 490 550	RESIDUAL
	N = 30 6,13,17	53	2	-	Extremely Weathered siltstone: silty CLAY, medium to high plasticity, grey, trace of iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
		52	3		SILTSTONE: grey and brown, with iron indurated bands and extremely weathered bands.	DW	VL L		VERY LOW TO LOW RESISTANCE LOW TO MODERATE RESISTANCE
		51 50	4 5		as above, but dark grey.				
		49 48	6		REFER TO CORED BOREHOLE LOG				

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
2
2 / 2

EASTING: 308513.926
NORTHING: 6257604.282

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN	**Core Size:** NMLC	**R.L. Surface:** 54.94 m
Date: 9/11/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.S./N.E.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
					START CORING AT 5.80m						
100% RETURN		49	6		NO CORE 0.09m						Bringelly Shale
					SILTSTONE: dark grey, bedded at 0-5°, with iron indurated bands.	HW	VL			(6.00m) Fractured Zone, 0°, 210 mm.t	
						MW	M	0.70		(6.18m) Be, 0°, Un, S, Cn	
								1.1		(6.48m) Be, 0°, Un, S, Cn	
										(6.52m) XWS, 0°, 3 mm.t	
										(6.66m) Be, 0°, Un, R, Clay Ct	
		48	7					0.50		(7.02m) XWS, 0°, 5 mm.t	
								0.60		(7.20m) Be, 0°, Un, R, Clay Ct	
										(7.31m) XWS, 0°, 3 mm.t	
								0.90		(7.58m) Jh, 70°	
						HW		0.70		(7.74m) Fractured Zone, 0°, 140 mm.t	
		47	8		SILTSTONE: dark grey and grey, bedded at 0-5°.	MW	L - M			(7.90m) J, 90°, Un, S, Cn	
								0.20		(8.28m) XWS, 0°, 140 mm.t	
								0.40		(8.56m) J, 75 - 90°, Ir, S, Clay FILLED, 2 mm.t	
		46	9		END OF BOREHOLE AT 8.64 m						
		45	10								
		44	11								
		43									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33524 PN
Borehole No: BH2
Depth: 5.80m - 8.64m
KODAK Gray Scale
KODAK Color Control Patches
Job No. 33524PN START CORING AT 5.80m
5
NO CORE 90mm
6
7
8
END OF HOLE AT 8.64m

JKGeotechnics

BOREHOLE LOG

Borehole No. 3

1 / 2

EASTING: 308594.571
NORTHING: 6257643.677

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 50.62 m
Date: 9/11/20 TO 10/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			FILL: Silty clay topsoil, medium plasticity, dark brown and red brown, trace of root fibres.	w~PL			GRASS COVER
	N = 7 3,3,4		FILL: Silty clay, medium plasticity, red brown, brown and grey, trace of fine to coarse grained ironstone gravel.				APPEARS MODERATELY COMPACTED
			FILL: Silty clay, medium plasticity, dark brown, with fine gained sand, trace of ash and root fibres.	w>PL			
	N = 11 4,4,7	CH	Silty CLAY: high plasticity, grey and red brown, trace of fine to coarse grained ironstone gravel and roots.	w~PL	VSt	310 350 340	RESIDUAL
	N > 41 7,16,25/ 140mm REFUSAL	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey, trace of iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE
			SILTSTONE: dark grey, with iron indurated bands and extremely weathered bands.	DW	VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
		-	SANDSTONE: fine grained, grey and brown, with iron indurated bands, extremely weathered bands and siltstone bands.	DW	L - M		LOW TO MODERATE RESISTANCE

RL (m AHD): 50, 49, 48, 47, 46, 45, 44
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No. 3

EASTING: 308594.571
NORTHING: 6257643.677

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 50.62 m
Date: 9/11/20 TO 10/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
ON 10/11/20									-	SANDSTONE: fine grained, grey and brown, with iron indurated bands, extremely weathered bands and siltstone bands. *(continued)*	DW	L - M M - H		MODERATE TO HIGH RESISTANCE
						43				END OF BOREHOLE AT 7.50 m				GROUNDWATER MONITORING WELL INSTALLED TO 7.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 7.5m TO 2.0m. CASING 2.0m TO 0m. 2mm SAND FILTER PACK 7.5m TO 1.5m. BENTONITE SEAL 1.5m TO 1.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
							8							
						42								
							9							
						41								
							10							
						40								
							11							
						39								
							12							
						38								
							13							
						37								

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BOREHOLE LOG

Borehole No.
4
1 / 2

EASTING: 308500.243
NORTHING: 6257685.932

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 56.70 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			0		FILL: Clayey sand, fine to medium grained, dark brown, trace of silt fines and root fibres.	M			GRASS COVER
	N = 21 8,9,12	56			FILL: Silty sand, fine to medium grained, brown, trace of concrete fragments, root fibres and clay nodules.	w~PL	Hd		
			1	CH	FILL: Silty clay, medium plasticity, grey and brown, trace of fine to medium grained igneous gravel and concrete fragments.	w<PL	Hd	>600 >600 >600	RESIDUAL
					Silty CLAY: high plasticity, red brown, brown and grey, trace of fine to medium grained ironstone gravel and root fibres.				
	N = 32 7,13,19	55		-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey, with iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW TO LOW 'TC' BIT RESISTANCE
		54	2		SILTSTONE: grey, with extremely weathered bands and iron indurated bands.	DW	VL		
		53	3						
		52	4		SILTSTONE: dark grey and grey.		L - M		LOW TO MODERATE RESISTANCE
							M - H		MODERATE TO HIGH RESISTANCE
		51	5				L		LOW RESISTANCE
		50	6		SILTSTONE: dark grey.		L - M		LOW TO MODERATE RESISTANCE

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BOREHOLE LOG

Borehole No.
4

EASTING: 308500.243
NORTHING: 6257685.932

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 56.70 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	SILTSTONE: dark grey. *(continued)*	DW	M - H		MODERATE TO HIGH RESISTANCE
						49				END OF BOREHOLE AT 7.50 m				
							8							
						48								
							9							
						47								
							10							
						46								
							11							
						45								
							12							
						44								
							13							
						43								

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BOREHOLE LOG

Borehole No.
5

EASTING: 308573.032
NORTHING: 6257697.725

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 51.45 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			FILL: Sandy clay topsoil, low plasticity, dark brown, trace of root fibres and plastic fragments.	w>PL			GRASS COVER
	N = 9 4,4,5	CH	Silty CLAY: high plasticity, red brown and grey, trace of fine grained ironstone gravel, root fibres and roots.	w>PL	St - VSt	200 200 210	RESIDUAL
	N > 9 9,9/ 100mm REFUSAL	-	Extremely Weathered siltstone: silty CLAY, high plasticity, trace of iron indurated bands.	XW	Hd	510 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
			SILTSTONE: dark grey and brown, with extremely weathered bands.	DW	VL - L		VERY LOW TO LOW RESISTANCE
			SILTSTONE: dark grey.		L		LOW RESISTANCE
					L - M		

Column headers also include: SAMPLES (ES, U50, DB, DS), RL (m AHD), Depth (m), Graphic Log.

RL (m AHD): 51, 50, 49, 48, 47, 46, 45
Depth (m): 1, 2, 3, 4, 5, 6

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
2 / 2

EASTING: 308573.032
NORTHING: 6257697.725

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 51.45 m
Date: 9/11/20 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						44			-	SILTSTONE: dark grey. *(continued)*	DW	L - M		LOW RESISTANCE
						43	8			END OF BOREHOLE AT 7.50 m				
						42	9							
						41	10							
						40	11							
						39	12							
						38	13							

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BOREHOLE LOG

Borehole No.
6

EASTING: 308650.157
NORTHING: 6257720.194

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 46.69 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION		0		FILL: Sandy clay topsoil, medium plasticity dark brown, with root fibres, trace of silt fines.	w>PL			GRASS COVER
	N = 6 1,2,4		CH	Silty CLAY: high plasticity, red brown and orange brown, trace of fine grained sand, ash and root fibres.	w>PL	VSt	240 210 250	RESIDUAL
				as above, but grey and orange brown, with fine to medium grained ironstone gravel.				
	N = 15 7,4,11	1.2	-	Extremely Weathered siltstone: silty clay, high plasticity, light grey and grey, trace of fine to coarse grained ironstone gravel and fine grained sand.	XW	Hd	>600 >600 >600	BRINGELLY SHALE SOIL 'TC' BIT RESISTANCE
		2.2		SILTSTONE: dark grey and brown, with iron indurated bands and extremely weathered bands.	DW	L		VERY LOW TO LOW RESISTANCE WITH MODERATE BANDS
		3.2		SILTSTONE: dark grey, with fine grained, sandstone bands and extremely weathered bands.				
		5.2	-	Interbedded SILTSTONE: dark grey and SANDSTONE: fine grained, grey, with iron indurated and very low strength bands.		M - H		MODERATE TO HIGH RESISTANCE WITH LOW BANDS
		6.0	-	SILTSTONE: dark grey, with very low strength bands.				

RL (m AHD) scale: 46, 45, 44, 43, 42, 41, 40. Depth scale: 1, 2, 3, 4, 5, 6.

SAMPLES: ES, U50, DB, DS

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BOREHOLE LOG

EASTING: 308650.157
NORTHING: 6257720.194

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 46.69 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	SILTSTONE: dark grey, with very low strength bands. *(continued)*	DW	M - H		
						39				END OF BOREHOLE AT 7.50 m				
							8							
						38								
							9							
						37								
							10							
						36								
							11							
						35								
							12							
						34								
							13							
						33								

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BOREHOLE LOG

Borehole No.
7
1 / 2

EASTING: 308572.324
NORTHING: 6257744.343

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 51.94 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			FILL: Clayey sand topsoil, fine to medium grained, dark brown, with root fibres, trace of roots and silt fines.	M			GRASS COVER
	N = 13 3,5,8		FILL: Clayey sand, fine to medium grained, dark brown and brown, fine to coarse grained igneous and ironstone gravel.	w~PL			APPEARS WELL COMPACTED
	N = 16 5,7,9	CH	Silty CLAY: high plasticity, red brown, orange brown and grey, trace of fine to medium grained ironstone gravel and root fibres.		VSt	390 380 290	RESIDUAL
		CI	as above, but grey and red brown.				
	N = 30 12,14,16	-	Extremely Weathered siltstone: silty CLAY, high plasticity, with iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
			SILTSTONE: dark grey and brown, with extremely weathered bands and iron indurated bands.	DW	VL		LOW TO VERY LOW RESISTANCE
					L - M		LOW RESISTANCE

Samples columns: ES, U50, DB, DS. RL (m AHD): 51, 50, 49, 48, 47, 46, 45. Depth (m): 1, 2, 3, 4, 5, 6.

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BOREHOLE LOG

Borehole No. 7

EASTING: 308572.324
NORTHING: 6257744.343

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 51.94 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	SILTSTONE: dark grey and brown, with extremely weathered bands and iron indurated bands. *(continued)*	DW	L - M		
										END OF BOREHOLE AT 7.50 m				
						44	8							
						43	9							
						42	10							
						41	11							
						40	12							
						39	13							
						38								

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BOREHOLE LOG

Borehole No.
8
1 / 2

EASTING: 308648.472
NORTHING: 6257765.555

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 46.57 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			FILL: Clayey sand topsoil, fine to medium grained, trace of root fibres.	M			GRASS COVER
	N = 9 2,3,6	CH	Silty CLAY: high plasticity, red brown and brown, trace of fine to coarse grained ironstone gravel.	w>PL	St - VSt	210 190 200	RESIDUAL
	N > 6 25,6/ 10mm REFUSAL	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey, with iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE
AFTER 2 HRS			SILTSTONE: grey, with extremely weathered bands and iron indurated bands.	DW	VL - L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
			as above, but dark grey.	DW	L - M		LOW TO MODERATE RESISTANCE
			SILTSTONE: dark grey, with fine grained sandstone bands.		M - H		MODERATE RESISTANCE WITH LOW BANDS

RL (m AHD): 46, 45, 44, 43, 42, 41, 40
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33524PN PROSPECT.GPJ <<DrawingFile>> 09/12/2020 16:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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BOREHOLE LOG

Borehole No.
8
2 / 2

EASTING: 308648.472
NORTHING: 6257765.555

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 46.57 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	SILTSTONE: dark grey, with fine grained sandstone bands. *(continued)*	DW	M - H		
						39				END OF BOREHOLE AT 7.50 m				GROUNDWATER MONITORING WELL INSTALLED TO 7.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 7.5m TO 1.5m. CASING 1.5m TO 0m. 2mm SAND FILTER PACK 7.5m TO 1.0m. BENTONITE SEAL 1.0m TO 0.5m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
							8							
						38								
							9							
						37								
							10							
						36								
							11							
						35								
							12							
						34								
							13							
						33								

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BOREHOLE LOG

Borehole No. 9

1 / 2

EASTING: 308709.322
NORTHING: 6257840.175

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 43.40 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			FILL: Clayey sand topsoil, fine to medium grained, dark brown, trace of fine to medium grained ironstone gravel, clay nodules and root fibres.	M / w>PL			GRASS COVER APPEARS WELL COMPACTED
	N = 12 1,4,8		FILL: Silty clay, medium plasticity, red brown, orange brown and grey, trace of fine grained sand, fine grained ironstone gravel, root fibres and ash.	M			
			FILL: Clayey sand, fine to medium grained, dark brown, with silt fines, root fibres, trace of fine to medium grained ironstone gravel.	w>PL			
	N = 11 2,5,6		FILL: Silty clay, medium plasticity, red brown, grey and brown, with fine to coarse grained ironstone gravel, trace of root fibres, clayey sand bands.				
			FILL: Silty clay, high plasticity, orange brown, trace of fine grained ironstone gravel.				
ON COMPLETION OF CORING	N = 20 5,7,13		FILL: Clayey sandy gravel, fine to coarse grained igneous and ironstone gravel, dark brown and grey, fine to medium grained sand, trace of silt fines and ash.	M			
		CH	Silty CLAY: high plasticity, light grey and red brown, with fine to coarse grained ironstone gravel.	w~PL	(Hd)		RESIDUAL
		-	SILTSTONE: dark grey and grey, with iron indurated bands and extremely weathered bands.	DW	L - M		BRINGELLY SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
		-	Interbedded SANDSTONE: fine grained, grey and SILTSTONE: dark grey, with very low strength iron indurated bands.		L		MODERATE RESISTANCE WITH LOW BANDS
			REFER TO CORED BOREHOLE LOG				

RL (m AHD): 43, 42, 41, 40, 39, 38, 37
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33524PN PROSPECT.GPJ <<DrawingFile>> 09/12/2020 16:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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CORED BOREHOLE LOG

Borehole No.
9
2 / 2

EASTING: 308709.322
NORTHING: 6257840.175

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

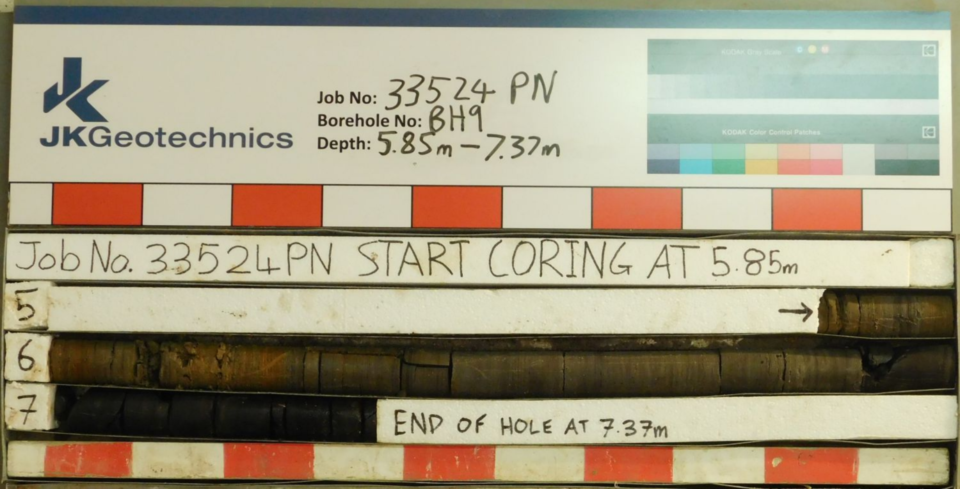
Job No.: 33524PN	**Core Size:** NMLC	**R.L. Surface:** 43.40 m
Date: 10/11/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK300	**Bearing:** N/A	**Logged/Checked By:** B.S./N.E.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
		38		START CORING AT 5.85m					
100% RETURN			6	Interbedded SANDSTONE: fine grained, grey and SILTSTONE: grey and brown, with iron indurated bands.	MW	M	0.50	(5.86m) Be, 0°, P, R, Fe Sn (6.10m) XWS, 0°, 100 mm.t	Bringelly Shale
		37			SW		0.060 0.60	(6.34m) J, 90°, P, R, Fe Sn	
			7	SILTSTONE: dark grey.			0.60	(6.92m) J, 70°, Un, S, Clay Ct (7.02m) J, 60°, P, S, Cn (7.36m) J, 90°, Un, S, Cn	
		36		END OF BOREHOLE AT 7.37 m					
		35	8						
		34	9						
		33	10						
		32	11						

SPACING (mm): 600, 200, 60, 20

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33524PN PROSPECT.GPJ <<DrawingFile>> 09/12/2020 16:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33524 PN
Borehole No: BH9
Depth: 5.85m – 7.37m
KODAK Gray Scale
KODAK Color Control Patches
Job No. 33524PN START CORING AT 5.85m
5
6
7
END OF HOLE AT 7.37m

JKGeotechnics

BOREHOLE LOG

Borehole No.
10
1 / 2

EASTING: 308648.547
NORTHING: 6257874.577

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 44.23 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION		44			FILL: Silty sand topsoil, fine to medium grained, dark brown and brown, with roots, root fibres, timber fragments, trace of fine grained igneous gavel and clay nodules.	M			MULCH COVER
	N = 16 4,8,8	43	1	CH	Silty CLAY: high plasticity, red brown and orange brown, trace of fine to coarse grained ironstone gravel, root fibres and ash.	w~PL	Hd	600 550 560	RESIDUAL
					as above, but red brown and grey.				
	N = 35 13,17,18		2	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey mottled red brown, with iron indurated bands.	XW	Hd	>600 >600 >600	BRINGELLY SHALE VERY LOW 'TC' BIT RESISTANCE
		42			as above, but with very low strength bands.				
	N=SPT 16/ 100mm REFUSAL	41	3		SILTSTONE: dark grey, with extremely weathered bands and iron indurated bands.	DW	VL - L		LOW RESISTANCE WITH MODERATE BANDS
		40	4						
		39	5		as above, but dark grey.		M		LOW TO MODERATE RESISTANCE WITH VERY ,LOW BANDS
		38	6		SILTSTONE: dark grey, with extremely weathered bands.				

SAMPLES: ES, U50, DB, DS

JKGeotechnics

BOREHOLE LOG

Borehole No.
10
2 / 2

EASTING: 308648.547
NORTHING: 6257874.577

Client: INTEGRATED PROJECTS
Project: PROPOSED SENIORS LIVING DEVELOPMENT
Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLF CLUB, PROSPECT, NSW

Job No.: 33524PN **Method:** SPIRAL AUGER **R.L. Surface:** 44.23 m
Date: 10/11/20 **Datum:** AHD
Plant Type: JK350 **Logged/Checked By:** B.S./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						37			-	SILTSTONE: dark grey, with extremely weathered bands. *(continued)*	DW	M		
						36	8			END OF BOREHOLE AT 7.50 m				
						35	9							
						34	10							
						33	11							
						32	12							
						31	13							

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	55 FOX HILLS CRESCENT, FOX HILLS GOLD CLUB, PROSPECT, NSW
Report No: 33524PN	Figure No: 1

JKGeotechnics

PLOT DATE: 17/11/2020 12:15:50 PM DWG FILE: Y:\33000's\33524PN PROSPECT\CAD\33524PN.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 20 40 60 80 100
SCALE 1:2000 @A3 METRES

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

Title: **INVESTIGATION LOCATION PLAN**

Location: 55 FOX HILLS CRESCENT, FOX HILLS GOLD CLUB, PROSPECT, NSW

Report No: 33524PN

Figure No: 2

JKGeotechnics

PLOT DATE: 8/12/2020 3:41:51 PM DWG FILE: Y:\33000'S\33524PN PROSPECT\CAD\33524PN.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

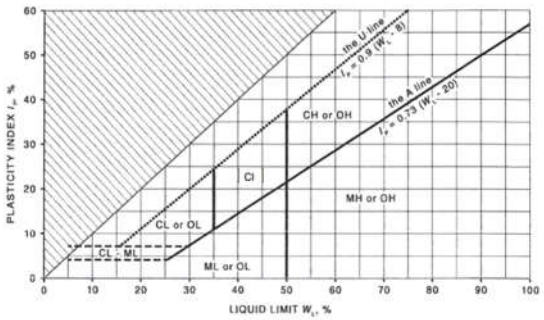
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

Density Index/ Relative Density (Cohesionless Soils)	Symbol		Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

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

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

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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