REPORT TO

UNIVERSITY OF NEW ENGLAND

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED UNIVERSITY BUILDING

AT

PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Date: 28 July 2023

Ref: 36020PNrpt

JKGeotechnics

www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801

Report prepared by:

Nicholas Smith
Senior Associate | Geotechnical Engineer

Report reviewed by:

Peter Wright
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
36020PNrpt	Final Report	28 July 2023

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

Borehole Logs 1 to 15 Inclusive

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 geotechnical investigation for the proposed University Building at Prince of Wales Park, Peel Street, Tamworth, NSW. The location of the site is shown in Figure 1. The investigation was commissioned under University of New England (UNE) Contract dated 26 July 2023. The commission was on the basis of our proposal Ref: P58373PN, dated 29 March 2023.

From the provided unreferenced architectural drawings prepared by Architectus, which were not available at the time of the fieldwork, we understand the proposed development will comprise a three or four storey teaching building towards the north-west end of the site, and an on-grade carpark towards the south-east end of the site, along with associated landscaping works. No basement is proposed for the building. The ground floor level of the proposed building will be at Reduced Level (RL) 378.7m, and to achieve design surface levels for the building, cut and fill earthworks to a maximum depth of about 4m and height of about 1m are expected to be required. Levels for the proposed carpark were not known at the time this report was prepared, but similar earthworks are expected to be required. As no structural loads have been provided, typical loads for this type of building have been assumed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on site preparation, footings, retaining walls, pavement design parameters, subgrade preparation and further works.

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: E36060PDrpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

Fifteen (15) boreholes (BH1 to BH15) were drilled to depths between 6m and 15m below existing levels using spiral augering techniques with our track mounted JK305 drilling rig. The compaction of the fill, strength of the natural cohesive soils and relative density of the natural granular soils were assessed from the results of Standard Penetration Tests (SPTs) completed in the boreholes, augmented by hand penetrometer tests on recovered cohesive soil samples. The strength of the greywacke bedrock was assessed from observation of auger penetration resistance using a tungsten carbide 'TC' drill bit, tactile examination of the recovered rock cuttings, and correlation with the results of subsequent moisture content tests. It should be noted that the assessment of rock strength in this manner is approximate and variations by about one strength order should not be unexpected.

Groundwater observations were made in the boreholes during and on completion of drilling and at the end of the fieldwork. In BH5, BH9 and BH13, class 18 machine slotted PVC standpipes were installed to allow for future groundwater level monitoring, and measurements were taken in the standpipes at the end of the fieldwork program. No longer term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Anh Phung, 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 attached borehole logs. For details of the investigation techniques adopted and a glossary of logging terms and symbols used, reference should be made to the attached Report Explanation Notes.

The borehole locations, as shown on the attached Borehole Location Plan (Figure 2) were set out by taped measurements from existing surface features. Figure 2 is based on available Nearmap aerial imagery. The approximate surface levels at the borehole locations were estimated by interpolation between contour lines on the provided Baxter Geo Consulting survey drawing (Dwg No. 0789-160229 Rev B dated 4 February 2016), which has been overlain on Figure2, and are hence approximate. The datum of the levels is Australian Height Datum (AHD).

Selected samples were returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for moisture content, Atterberg limits, linear shrinkage, Standard compaction and 4 day soaked CBR testing, the results of which are presented on STS Tables A and B. Additional samples were also returned to Envirolab Services Pty Ltd for pH, chloride content, sulphate content and resistivity testing, the results of which are presented in the attached Envirolab Services 'Certificates of Analysis' 326029.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in gently sloping terrain characterised by maximum surface gradients of about 5°, on the north-east bank of Peel River. Roderick Street, Peel Street, and Scott Road bound the site to the north-west, north-east, and south-east respectively.

At the time of the fieldwork, a velodrome occupied the site, with the site appearing to have been cut and filled to achieve design levels for the velodrome. At the western end of the site, a concrete tunnel with brick and concrete wing walls provided access below the velodrome, near the southern corner of the proposed building. Apart from the velodrome itself, the site was grass covered, with scattered shrubs and medium trees.

To the south-west of the site was a concrete cycle path, beyond which was the bank of Peel River.

3.2 Subsurface Conditions

The 1:250,000 geological map of Tamworth indicates the site to be underlain by Quaternary age alluvial soils over bedrock of the Baldwin Formation, which comprises argillite and greywacke.

The boreholes disclosed a generalised subsurface profile comprising shallow to moderately deep fill overlying alluvial soils, with greywacke bedrock at depth. Reference should be made to the attached borehole logs for detailed subsurface descriptions at specific locations. A summary of the subsoil conditions, as encountered, is presented below.

Fill

Fill was encountered from the surface in all of the boreholes to depths between 0.3m (BH9) and 4.4m (BH3), but was mostly encountered to no greater than about 3m depth. The fill generally comprised silty clay soils, but with some layers of sandy fill also encountered. Based on the results of the SPTs, the fill was assessed to be moderately or well compacted.

Alluvial Soils

Alluvial soils, predominantly comprising clays of medium plasticity, were encountered below the fill in all boreholes expect BH4, where no natural soil was encountered. The clays were mostly of very stiff and hard strength, however, in BH8, BH9, BH10, BH11, BH14, stiff and stiff to very stiff strength clay layers were encountered.

In BH1, BH4, BH9, BH10, BH12, BH13, and BH14, alluvial sands and gravels were encountered within the clayey soils. The sands and gravels were generally of medium dense or dense relative density, but with some loose soils also encountered.

The majority of the boreholes were terminated in the soils at their target depth.

Bedrock

Greywacke bedrock was encountered in BH1, BH3, BH4 and BH7 at respective depths of 9.5m, 6.4m, 3.15m, and 4.6m, and these boreholes refused after limited penetration into the greywacke. The greywacke was distinctly weathered and of generally low to medium or medium to high strength.

Groundwater

Groundwater seepage was encountered during drilling in BH1, BH8, BH9, BH10 and BH12 at respective depths of 6.0m, 5.8m, 3.4m, 6.0m, and 10.8m, which equates to between about RL 371mAHD and RL 372.5mAHD. The remaining boreholes were 'dry' during drilling.

On completion of drilling, standing water was measured in BH1, BH8, BH9 and BH12 at respective depths of 6.2m, 7.45m, 4.9m, and 9.4m (between about RL370.5m and 372.5m). BH10 collapsed to a depth of 1.1m on completion of drilling, and the remaining boreholes were 'dry' on completion of drilling.

In BH1, standing water was measured at a depth of 6.4m (RL372.1m) 3 days following the completion of drilling, and in BH9, standing water was measured at a depth of 4.9m (371.6m) 1 day following the completion of drilling. The remaining boreholes were 'dry' on completion of drilling, and the standpipe in BH13 remained 'dry' 1 day after drilling.

3.3 Laboratory Test Results

The results of the Atterberg limits tests on the recovered clayey fill and alluvial clay confirmed the clayey soils to be of medium plasticity, and the linear shrinkage tests indicated the clayey soils to be moderately to highly reactive to moisture content change. The moisture content tests completed on the clayey soils confirmed the moisture contents were generally below the soils plastic limit.

The moisture content tests on the recovered rock chip samples correlated well with our field assessment of the bedrock strength.

The four day soaked CBR tests on the recovered bulk samples of the clayey fill and alluvial clay returned CBR values of between 1% and 3% when compacted to 98% of their respective Standard Maximum Dry Density (SMDD) at close the their respective Standard Optimum Moisture Content (SOMC). The in-situ moisture content of the samples tested was between 1.8% ‘dry’ and 6% ‘wet’ of their respective SOMCs. All of the samples swelled, by between 0.5% and 3%, during soaking, which also confirms the clays are reactive to moisture content change.

The results of the Envirolab testing indicated the soils to have a near neutral to moderately alkaline pH, very low chloride and sulfate content, and low (unfavourable) to high (favourable) resistivity.

4 COMMENTS AND RECOMMENDATIONS

4.1 Existing Fill Considerations

Following excavation to design subgrade level, up to 2m of the existing fill will remain below the proposed building, and we expect similar conditions for the proposed carpark.

Whilst the existing fill was assessed to be moderately and well compacted, as no records of the fill placement or compaction are available, the fill must be considered to be uncontrolled, and as such unsuitable to support the proposed building and carpark. If the ground floor level is to be a conventional on-grade element, then the existing fill would need to be wholly removed, down to the underlying natural soils, over the footprint of the proposed building, plus a 2m buffer zone, and replaced as engineered fill to design subgrade levels.

For the proposed building, as an alternative to removing the existing fill and replacing to design subgrade level with engineered fill, consideration could also be given to leaving the existing fill in-situ, filling where required to achieve design levels with ‘formwork fill’ and designing the proposed building, including the ground floor slab, as a suspended structure supported by piled footings founded below the existing fill. We note that if this approach is adopted, then detailed consideration by the civil and structural engineers must be given to any landscaping elements connecting to the building, e.g. paths or driveways, as settlement of the fill would inherently result in differential movement between the building and landscaping elements. The magnitude of such settlement/differential movement is not possible to quantify, due to the number of unknowns with regard to compaction of the existing/formwork fill and magnitude of future loading.

Given the limited depth of fill below the proposed building, from a costs perspective, a basement storage level could also be considered, as following excavation for the basement, only natural soils would be present below the building. If this option is to be pursued, further geotechnical advice must be sought. For the purpose of this report, we have assumed a basement level would not be adopted.

For the proposed carpark, as well as for landscaping works, consideration could be given to leaving the existing fill in-situ though this would require accepting the potential for future reduced performance of pavements and landscaping should settlement of the fill occur. Alternatively, the existing fill could be removed over the entire site, and replaced as engineered fill as a part of the proposed works.

4.2 Site Preparation and Earthworks

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.2.1 Site Drainage

The clay subgrade at the site is expected to undergo substantial loss of strength when wet as evidenced from the low CBR values. Furthermore, the clay subgrade is expected to have a moderate to high shrink-swell reactive potential. Soils with these characteristics are also often dispersive, though this property has not been confirmed by testing. 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 quickly become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

4.2.2 Site Preparation

Following demolition, and removal of any vegetation on site, any topsoil or root affected soils should also be stripped. A formal topsoil layer was not noted in the boreholes, however, the upper portion of the fill was root impacted. If quantification of the thickness of any topsoil is required, excavation of test pits would be required to allow for careful inspection and measurements of any topsoil in the sides of the test pits.

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 below for guidance on the offsite disposal of soil.

In any areas where the fill or natural soils below subgrade level are water softened, the water softened material must be boxed out to a sound base.

4.2.3 Excavation

Excavation of the soils can be completed using hydraulic excavators.

Any cuts should be temporarily battered back or benched at no steeper than 1V on 1H for stability considerations and to facilitate compaction of engineered fill up against cut faces. Where cut batters are more than 3m high, a 1m wide horizontal bench must be incorporated for each 3m vertical height of the batter. Surcharge loads, including stockpiles and plant, must be located at least 3m back from the crest of such batters (assuming a maximum batter height of about 4.5m).

If excavations will be completed such that the toe of any batter is within 2H of the site boundary where H is the depth of excavation, further geotechnical advice should be sought before excavation commences. We also note the presence of several council buried services within the site. Advice must be sought from Council as to precautions and excavation approach distances for their assets.

4.2.4 Subgrade Preparation

Following stripping of fill (as required) and excavation to achieve design subgrade levels, the exposed subgrade should be tyned to a depth of 0.2m, moisture conditioned to within 2% of SOMC, and recompacted to a density ratio between 98% and 102% of SMDD. In situ density testing must be completed in accordance with the requirements of Section 4.2.5 below.

If soil softening occurs after rainfall periods, then the clay subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface must be moistened with a water cart and rolled until the shrinkage cracks are no longer evident, or the layer tyned, moisture conditioned, and recompacted. Care must be taken not to over-water the subgrade as this will result in softening.

All fill used to raise site levels must be placed as engineered fill.

4.2.5 Engineered Fill

From a geotechnical perspective, the excavated soils are 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 70mm. We note that the moisture condition of the soils is highly variable, and moisture conditioning of at least some of the soils will be required to meet to specification requirements below. Drying back of soils can be time consuming, and provision must be made in the project program and budget to allow for this as appropriate.

All clayey fill should be compacted in layers not exceeding 200mm loose thickness to a density ratio strictly between 98% and 102% of SMDD and within 2% of SOMC. Engineered fill comprising well graded granular materials, such as imported crushed rock, should be compacted layers not exceeding 200mm loose thickness to achieve a density ratio of at least 98% of SMDD.

Backfilling of service trenches and to retaining walls 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 and behind retaining walls, backfilling should be carried out in layers not exceeding 100mm loose thickness 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.

Compaction of engineered fill behind retaining walls is time consuming 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; 400mm thick layers could be adopted if an excavator mounted vibrating plate is used for compaction. 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, and this should be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of clayey engineered fill, or pavement, to reduce the potential for surface water to enter the retaining wall drainage. Provided the aggregate backfill is placed as recommended above, density testing would not be required in that material.

In-situ density tests for all engineered fill must be carried out in accordance with the requirements of Table 8.1 of AS3798-2007.

Level 1 inspection and testing of fill placement and compaction in accordance with AS3798-2007 is recommended for this project, including for trench and retaining wall backfill. Due to a potential conflict of interest, we strongly recommend that the GITA be directly engaged by the client, and not by the earthworks contractor or sub-contractors.

4.2.6 Formwork Fill

If a suspended building design is adopted, placement of fill below the footprint of the proposed building would only be required to act as a working platform for piling and as formwork during the construction phase. From an engineering perspective, the critical design case will likely be for a piling rig working platform.

From a geotechnical perspective, we also the excavated soils are considered suitable for reuse as formwork fill on condition that they are 'clean', free of organic matter and contain a maximum particle size not exceeding 70mm.

Subject to confirmation of piling rig track loads from a piling contractor, we provisionally recommend that formwork fill be compacted in layers not exceeding 200mm loose thickness to a density ratio of not less than 95% of SMDD. We further recommend an upper compaction limit of 100% of SMDD and a moisture content within 2% of SOMC for formwork fill to reduce the potential for excessive swelling of the formwork fill in the future.

For formwork fill, we recommend Level 2 testing of fill compaction be adopted in accordance with AS3798-2007. Due to a potential conflict of interest, we strongly recommend that the GTA be directly engaged by the client, and not by the earthworks contractor or sub-contractors.

4.2.7 Earthworks Testing Overview

The long term successful performance of 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 (Level 1) or GTA (Level 2) 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. It is too late at the end of the job to carry out a review as there will be no opportunity to make improvements or take corrective action.

4.3 Retaining Walls

We assume that any retaining walls will be constructed within temporarily battered excavations, and then backfilled in accordance with Section 4.2.5 above.

Free standing cantilever walls, with a maximum height of about 3m, and where minor wall movements are tolerable (e.g. when supporting soft landscaped areas) should be designed using a triangular lateral earth pressure distribution with an ‘active’ earth pressure coefficient, K_a, of 0.35 for the soil, 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, as well as for any backfill materials, assuming a horizontal retained surface.

A bulk unit weight of 21kN/m^3 should be adopted for the soil.

Any surcharge affecting the walls (e.g. traffic loading, construction loads, adjacent high level footings, sloping backfill, etc.) should be allowed for in the design using the appropriate earth pressure coefficient from above.

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 cantilever walls may be achieved by passive resistance of the soil 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. We note that significant movement is required in order to mobilise the full passive pressure of the resisting material, 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 can be calculated using a friction angle of 28˚ between the retaining wall base and the soil below, provided the base is clean, rough and 'dry' when the retaining wall footing is poured.

4.4 Footing Design

For footings to support the proposed building, two general options could be considered, being (1) an 'on-grade' building supported on high level footings founded in full depth engineered fill or natural soils, or (2) a fully suspended building (including the ground floor slab) supported on piled footings founded in the underlying Greywacke bedrock.

If Option 1 is adopted, then the advice in Section 4.2 above for removal of the existing fill, subgrade preparation, and engineered fill placement, compaction, and testing would need to be adopted in full below the proposed building, plus a 2m buffer zone beyond the building footprint.

If Option 2 is adopted, then the existing fill could remain in-situ, and new fill to achieve design subgrade levels placed as formwork fill as discussed in Section 4.2.6 above.

4.4.1 Site Classification and Movement

Whilst AS2870 does not directly apply to this project, due to the presence of fill the site would be classified as Class 'P' in accordance with AS2870-2011. However, if the existing fill is removed down to the underlying natural soils and engineered fill then placed to design subgrade levels, reclassification in accordance with Section 2.5.3 of AS2870-2011 could be considered. Assuming engineered fill would be to not less than 2.5m depth below subgrade level, and that site won clayey soils are used as engineered fill, then characteristic surface movements equivalent to a Class H2 site, in the order of 65mm, would be expected. We note this assumes the building would be constructed within 5 years of final cut and fill earthworks.

We note that AS2870-2011 is not intended to be used in the design of this types of structure, and so the standard footing designs in AS2870 are not relevant to this project. The footing design must be carried out using engineering principles, with the above movements providing an indication to the structural engineer of the range of movements to be expected.

4.4.2 Shallow Footings

For shallow footings founded in full depth engineered fill the fill (either the existing fill or new fill placed under Level 1 control to the Specification in Section 4.2.5 above) or natural clays of at least very stiff strength, an allowable bearing pressure of 100kPa can be adopted, provided the footings are founded at a depth of at least 0.8m below surrounding ground level. If natural clays of stiff strength are encountered at founding level, deepening of the footing excavations may be required.

For the design of shallow footings or raft slabs, an elastic modulus of 30MPa can be adopted for the fill and natural soil. We note that this assumes that the footings are founded at least 2m above any loose sands, such as those encountered in BH1, which based on RL of the proposed ground floor level would be the case.

If an on-grade structure supported by shallow footings is to be adopted, then the overall building structure must be designed to accommodate shrink-swell movements detailed in Section 4.4.1 above.

For linear structures, such as retaining walls, supported on footings founded in the fill, articulation at no greater than 5m centres must be adopted.

At least the initial stage of shallow footing excavation should be inspected by a geotechnical engineer to confirm the appropriate foundation material has been achieved. For footings founded in engineered fill, Dynamic Cone Penetrometer (DCP) tests are recommended as a part of the inspection regime.

We recommend that all high level footings be excavated, cleaned, inspected and poured with minimum delay to avoid either wetting or drying of the foundation. If delays in pouring concrete are anticipated, we recommend that the base of the footings be protected with a blinding layer of concrete of at least 75mm thickness. Water should be prevented from ponding in the base of footing excavations as this will tend to soften the foundation material, resulting in further excavation and cleaning being required.

4.4.3 Piles in Bedrock

Given the variability in the strength/relative density of the alluvial soils, with soils of limited strength/relative density present below the proposed building, and the moderate depth to bedrock, we consider that if piled footings are adopted, the piles to the bedrock profile would be the most appropriate option. We note however that where bedrock was encountered, in BH1, BH3, BH4 and BH7, only limited penetration into the bedrock profile was achieved. We further note that bedrock was only confirmed near the western corner of the proposed building, in BH1, with bedrock not encountered in the remaining boreholes near the proposed building, though these were terminated at shallower depth than where the bedrock was encountered in BH1.

Due to the collapsing nature of the granular soils and presence of groundwater, conventional bored piles would not be appropriate on this site, and Continuous Flight Auger (CFA) piles would be required.

For piles socketed a nominal 0.3m into the Greywacke bedrock, an allowable bearing pressure of 1,000kPa can be adopted for preliminary design. It is possible that bearing pressures to about 2,000kPa to 3,000kPa would be achievable subject to further proving. For rock sockets longer than this 0.3m, an allowable shaft adhesion of 100kPa in compression and 50kPa in tension (uplift) can be adopted.

As the bedrock was encountered in only one borehole at the proposed building, and to assess if higher allowable bearing pressures/shaft adhesions can be achieved, we strongly recommend the drilling of an additional 4 boreholes, 1 at each corner of the proposed building, with these diamond core drilled into the bedrock. We can provide a proposal to complete such drilling if commissioned to do so.

To protect the building from high uplift pressures that can result when the clayey soils swell, the suspended floor slab and ground beams must be provided with a collapsible (cardboard) void former suitable for not less than 50mm of swell movement,

For CFA piles, certification is typically provided by the piling contractor. We could however review the piling records if commissioned to do so.

4.5 Soil Aggression

Based on the results of pH, chloride content, sulfate content and resistivity testing, an exposure classification of 'Mild' is applicable to both concrete and steel piles, according to Table 6.4.2(C) and 6.5.2(C) of AS2159-2009.

4.6 On-Grade Floor Slabs

Slab-on-grade construction is considered appropriate for the proposed building provided all existing fill has been removed and engineered fill then used to fill to design subgrade level (See Section 4.2 above), and that the predicted shrink-swell movements (see Section 4.4.1 above) can be tolerated. On-grade floor slabs should be designed using a CBR value of 1.5% assuming the engineered fill platform comprises site won clayey soils (or imported soils) with a CBR of not less than 1.5%.

On-grade floor slabs should be isolated from the walls, columns and footings to allow for shrink-swell movements in the underlying clays. Joints in concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints. Articulation joints would also be required at the transition from suspended slabs over the basement carpark to the on-grade floor slab. With potential characteristic surface movements to 65mm, it is likely that the differential movements cannot be tolerated.

The structural designer must assess the required thickness and nature of the subbase to the proposed concrete floor slab.

The detailing of ground floor slabs within buildings as a slab-on-grade that can accommodate shrink-swell movements of the underlying soils, as well as the relative differential movements associated with piled structures, is difficult. If piled footings to support the building structure only are adopted, we recommend the perimeter walls of the building be provided with an edge beam that is founded at least 0.8m below surrounding ground surface levels, with the buildings surrounded by concrete pavements that are at least 2m wide and which abut the building as well as grade away from the building. The joint between the buildings and concrete pavements must be appropriately sealed to prevent water ingress.

4.7 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. Consideration could be given to adopting selective fill zoning so that imported, select, fill with a higher CBR value is used for the upper, say, 0.4m below subgrade level. Indicatively for a concrete pavement, if 0.4m thickness of material with a CBR of not less than 10% is placed above natural soils with a CBR of 1.5%, a weighted average design CBR of 3% could be adopted for the pavement. Once a select fill is sourced, a final design CBR value can be provided if required. For flexible (asphaltic concrete) pavements, the use of select fill should be considered mechanistically using Circly.

We recommend that all basecourse materials for flexible pavements and sub-base materials for rigid pavements comprise DGB20 which meets the requirements of TfNSW QA Specification 3051 unbound base. The DGB20 material should be compacted in layers not exceeding 200mm loose thickness 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 to reduce the potential for material breakdown. For rigid pavement, a cement stabilised or lean mix concrete subbase could also be considered.

We further recommend that all sub-base materials for flexible pavements comprise DGS40, DGS20 or DGB20 which meets the requirements of TfNSW QA Specification 3051. The sub-base material should be compacted in layers not exceeding 200mm loose thickness using a large smooth drum roller to at least 95% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement.

The final 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 1,000m^2, or three tests per visit, whichever requires the most tests. Level 2 testing in accordance with AS3798-2007 would be considered acceptable for the pavement layers. The geotechnical testing authority (GTA) should be directly engaged by the client 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 design subgrade level. The drainage trenches should be excavated with a continuous 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.

4.8 Earthquake Design

Based on the encountered subsurface conditions over the footprint of the proposed building, in accordance with Table 4.1 of AS1170.4-2007, the site subsoil class is 'Class C_e – Shallow Soil'.

4.9 Further Geotechnical Input

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

- Proof rolling of subgrade.
- Density testing of all engineered fill, sub-base and base course materials.
- Supplementary investigation including coring of bedrock to optimise pile design parameters.
- Geotechnical review of pile installation records.

5 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.

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.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A

MOISTURE CONTENT, LIQUID LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed University Building
Location: Prince of Wales Park, Peel Street, Tamworth, NSW

Report No.: 36020PN - A
Report Date: 11/07/2023

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	0.50 - 0.95	14.2	39	17	22	8.5
1	9.50 - 10.00	9.5	-	-	-	-
3	0.50 - 0.95	9.5	41	18	23	11.5
3	6.40 - 6.80	5.0	-	-	-	-
4	3.30 - 3.50	5.7	-	-	-	-
4	4.00 - 4.50	6.4	-	-	-	-
6	1.50 - 1.95	18.2	48	19	29	12.0
7	4.60 - 5.00	7.9	-	-	-	-
9	0.50 - 0.95	19.8	46	23	23	11.5*
14	0.50 - 0.95	12.9	43	17	26	9.0**
15	3.00 - 3.45	22.9	40	17	23	9.5

Notes:
- 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: 20/06/2023.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage curled.
- ** Denotes Linear Shrinkage cracked.

NATA
NATA Accredited Laboratory Number:1327

11/07/2023
Authorised Signature / Date
(D. Treweek)

115 Wicks Road
Macquarie Park, NSW 2113
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed University Building
Location: Prince of Wales Park, Peel Street, Tamworth, NSW

Report No.: 36020PN - B
Report Date: 29/06/2023

BOREHOLE NUMBER	BH 1	BH 6	BH 7	BH 11	BH 12	BH 14
DEPTH (m)	0.50 - 1.80	0.50 - 1.50	0.50 - 1.50	0.00 - 1.00	0.00 - 1.00	0.50 - 1.50
Surcharge (kg)	9.0	9.0	9.0	9.0	9.0	9.0
Maximum Dry Density (t/m^3)	1.68 STD	1.83 STD	1.74 STD	1.81 STD	1.79 STD	1.70 STD
Optimum Moisture Content (%)	17.9	14.6	17.3	15.3	14.9	17.4
Moulded Dry Density (t/m^3)	1.64	1.80	1.70	1.78	1.75	1.67
Sample Density Ratio (%)	98	98	98	98	98	98
Sample Moisture Ratio (%)	102	97	101	99	99	98
Moisture Contents						
Insitu (%)	20.0	14.0	18.0	21.3	13.1	16.6
Moulded (%)	18.4	14.2	17.5	15.1	14.7	17.1
After soaking and After Test, Top 30mm(%)	28.4	25.4	21.1	23.0	24.0	28.9
Remaining Depth (%)	20.5	17.5	18.2	18.1	19.8	22.3
Material Retained on 19mm Sieve (%)	2*	3*	0	3*	1*	1*
Swell (%)	1.5	1.0	2.0	0.5	1.0	3.0
C.B.R. value: @2.5mm penetration	1.5	2.5		2.5		1.0
@5.0mm penetration			2.0		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.
- BH 1, 6, 7 & 14 had insufficient material supplied to complete a four-point compacion curve.
- * Denotes not used in test sample.
- Date of receipt of sample: 20/06/2023.

NATA
NATA Accredited Laboratory Number:1327

29/06/2023
Authorised Signature / Date
(D. Treweek)

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 326029

Client Details

Client	JK Geotechnics
Attention	N Phung
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**36020PN, Tamworth**
Number of Samples	6 Soil
Date samples received	20/06/2023
Date completed instructions received	20/06/2023

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	27/06/2023
Date of Issue	27/06/2023

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		326029-1	326029-2	326029-3	326029-4	326029-5
Your Reference	UNITS	2	2	3	3	10
Depth		1.5-1.95	4.5-4.95	0.5-0.95	3-3.45	0.5-0.95
Date Sampled		05/06/2023	05/06/2023	05/06/2023	05/06/2023	05/06/2023
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	22/06/2023	22/06/2023	22/06/2023	22/06/2023	22/06/2023
Date analysed	-	22/06/2023	22/06/2023	22/06/2023	22/06/2023	22/06/2023
pH 1:5 soil:water	pH Units	8.1	7.3	8.3	8.1	8.0
Sulphate, SO4 1:5 soil:water	mg/kg	62	20	<10	280	<10
Chloride, Cl 1:5 soil:water	mg/kg	10	10	<10	<10	<10
Resistivity in soil*	ohm m	94	96	100	31	210

Misc Inorg - Soil		
Our Reference		326029-6
Your Reference	UNITS	10
Depth		4.5-4.95
Date Sampled		05/06/2023
Type of sample		Soil
Date prepared	-	22/06/2023
Date analysed	-	22/06/2023
pH 1:5 soil:water	pH Units	8.6
Sulphate, SO4 1:5 soil:water	mg/kg	10
Chloride, Cl 1:5 soil:water	mg/kg	<10
Resistivity in soil*	ohm m	320

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			22/06/2023	1	22/06/2023	22/06/2023		22/06/2023	[NT]
Date analysed	-			22/06/2023	1	22/06/2023	22/06/2023		22/06/2023	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	8.1	8.3	2	101	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	62	[NT]		93	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	10	[NT]		98	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	94	96	2	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH/EC
Samples were out of the recommended holding time for this analysis.

JKGeotechnics

BOREHOLE LOG

Borehole No. 1

1/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 378.5m

Date: 5/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty sand, fine to medium grained, dark brown, trace of igneous gravel, clay nodules and root fibres.	D			GRASS COVER
		N = 10 3,6,4				FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained sand, fine to medium grained igneous gravel, and root fibres.	w<PL		400 450 500	APPEARS MODERATELY TO WELL COMPACTED
			1							
		N = 16 5,6,10							>600 >600 >600	
			2		CI	Silty CLAY: medium plasticity, brown.	w>PL	VSt		ALLUVIAL
		N = 8 2,3,5	3						200 200 200	
			4		SP	Silty gravelly SAND: fine to coarse grained, grey and orange brown, fine to coarse grained sub-rounded and sub angular alluvial gravel, trace of clay fines, with silty sandy gravel bands.	M	L		BANDS OF LOW AND MODERATE 'TC' BIT RESISTANCE
		N = 8 4,4,4	5							
G.W. ON COMPLETION AND AFTER 1 HR		N = 7 6,3,4	6				W			
AFTER 3 DAYS			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. **1**

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 378.5m

Date: 5/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					N = 32 12,16,15	8		SP	Silty gravelly SAND: fine to coarse grained, grey and orange brown, fine to coarse grained sub-rounded and sub angular alluvial gravel, trace of clay fines, with silty sandy gravel bands.	W	D		
					N = SPT 15/150mm REFUSAL	9							
								-	GREYWACKE: fine grained, grey.	DW	L-M		BALDWIN FORMATION
													HIGH 'TC' BIT RESISTANCE
						10			END OF BOREHOLE AT 10.0m				'TC' BIT REFUSAL
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **2**

1/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m

Date: 5/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty sand, fine to medium grained, dark brown, trace of igneous gravel, clay fines and root fibres.	D			GRASS COVER
						FILL: Silty clayey gravel, fine grained, grey and dark brown, sub-angular, igneous, trace of root fibres.	M			APPEARS MODERATELY TO WELL COMPACTED
		N = 11 5,6,5				FILL: Silty clay, medium plasticity, dark brown, trace of fine grained igneous gravel and root fibres.	w≈PL			
			1							
		N = 17 4,8,9							350 370 300	
			2							
		N = 9 3,3,6	3			as above, but red brown and dark brown.			190 190 190	
			4		CI	Silty CLAY: medium plasticity, dark brown.	w≈PL	Hd		ALLUVIAL
		N = 12 5,5,7							>600 >600 >600	
			5							
		N = 21 4,9,12	6						>600 >600 >600	
			7							

BOREHOLE LOG

Borehole No.
2
2/2

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m
Date: 5/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■				CH	Silty CLAY: high plasticity, dark brown.	w≈PL	Hd		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1/2

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m
Date: 6/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION			0			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, and fine to medium grained igneous gravel, plastic fragments and root fibres.	w<PL			GRASS COVER APPEARS MODERATELY COMPACTED
		N = 12 3,6,6				FILL: Silty gravelly clay, medium plasticity, brown and grey, sub-rounded and sub-angular, igneous gravel, trace of fine to medium grained sand, plastic fragments and root fibres.			430 450 530	
			1							
		N = 12 4,5,7							530	
			2							
						as above, but trace of ash, without plastic fragments.	w≈PL			
		N = 6 3,4,2	3							
						FILL: Silty clay, medium plasticity, brown, trace of fine to coarse grained, sub-angular, igneous gravel.				
			4							
		N = 10 3,4,6			CI	Silty CLAY: medium plasticity, dark brown, trace of fine to medium grained sub-angular alluvial gravel.	w>PL	VSt- Hd	350 350 400	ALLUVIAL
			5							
						as above, but brown.				BAND OF VERY LOW 'TC' BIT RESISTANCE
		N = SPT 10/50mm REFUSAL	6			Silty CLAY: medium plasticity, yellow brown, trace of fine to medium grained sand, and fine grained, sub-angular, alluvial gravel.				
					-	GREYWACKE: fine grained, grey.	DW	M-H		BALDWIN FORMATION HIGH 'TC' BIT RESISTANCE
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
3

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m
Date: 6/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									GREYWACKE: as above.	DW	H		
									END OF BOREHOLE AT 7.1m				'TC' BIT REFUSAL
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 4

1/1

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.5m

Date: 6/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./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 COMPLET-ION		0		FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained igneous gravel, clay fines and root fibres.	D			GRASS COVER
	N = 15 5,7,6			FILL: Silty clay, medium plasticity, brown, trace of sub-angular, igneous gravel, fine to medium grained sand, and root fibres.	w≈PL			APPEARS POORLY TO MODERATELY COMPACTED
		1						
	N = 6 3,2,4						200 220 220	
		2						
	N > 3 6,3/50mm REFUSAL	3					520 440 400	
			-	GREYWACKE: fine to medium grained, grey.	DW	M-H		BALDWIN FORMATION MODERATE TO HIGH 'TC' BIT RESISTANCE
		4						
				END OF BROREHOLE AT 4.5m				'TC' BIT REFUSAL
		5						
		6						
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. **5**

1/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m

Date: 6/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
ON COMPLET-ION AND AFTER 2 DAYS						0			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, igneous gravel and root fibres.	w<PL			GRASS COVER APPEARS WELL COMPACTED
					N = 18 6,10,8				FILL: Silty clay, medium plasticity, dark brown, trace of fine to medium grained sand, igneous gravel, and root fibres.	w≈PL		380 400 350	
						1							
					N = 14 4,7,7							280 250 220	
						2							
					N = 16 6,7,9	3			FILL: Silty clay, medium plasticity, dark brown, trace of fine grained, sub-rounded and sub-angular, alluvial and igneous gravel.			350 380 330	
						4		GP	Clayey GRAVEL: fine grained, grey, sub-rounded, alluvial.	D			ALLUVIAL HIGH 'TC' BIT RESISTANCE
						5		CI	Silty CLAY: medium plasticity, dark brown, trace of coarse grained, sub-angular, alluvial gravel.	w≈PL	Hd	600 350 >600	SOIL STRENGTH RESISTANCE
									as above, but yellow brown and orange brown, trace of fine grained, sub-angular, alluvial gravel.				
					N = 25 6,11,14	6							
						7							

BOREHOLE LOG

Borehole No.
5

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m
Date: 6/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								CI	Silty CLAY: medium plasticity, yellow brown and orange brown, trace of fine grained, sub-angular, alluvial gravel.	w>PL	Hd		
						8			END OF BOREHOLE AT 7.5m				GROUNDWATER MONITORING WELL INSTALLED TO 7.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.5m TO 7.5m. CASING 4.5m TO 0.1m. 2mm SAND FILTER PACK 4.2m TO 7.5m. BENTONITE SEAL 1.9m TO 4.2m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **6**

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 379.5m

Date: 6/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./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 COMPLET-ION		0		FILL: Silty clay, medium plasticity, brown, trace of igneous gravel, fine to medium grained sand, glass fragments and root fibres.	w<PL			GRASS COVER APPEARS MODERATELY COMPACTED SCREEN: 10.61kg 0-0.1m, NO FCF INSUFFICIENT RETURN FOR SCREEN SAMPLE
	N = 12 6,6,6						410 400 400	
		1						
	N = 9 7,4,5			as above, but orange brown and brown, without glass fragments.	w≈PL		230 260 250	
		2						
			CI	Silty clay, medium plasticity, dark brown and brown.	w≈PL	VSt		ALLUVIAL
	N = 11 4,5,6	3					230 310 250	
		4						
	N = 16 5,7,0					VSt- Hd	320 410 380	
		5						
			CI	Silty CLAY: medium plasticity, brown, trace of fine to coarse grained, sub-rounded and sub-angular, alluvial gravel.				
	N = 22 7,9,13	6				Hd	460 470 540	
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. 6

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 379.5m
Date: 6/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■					Silty CLAY: medium plasticity, brown, trace of fine to coarse grained, sub-rounded and sub-angular, alluvial gravel.	w≈PL	Hd		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 7

1/1

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.5m

Date: 7/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./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: Silty clay, medium plasticity, brown, with igneous gravel, trace of fine to medium grained sand, glass fragments and root fibres.	w<PL			GRASS COVER APPEARS MODERATELY COMPACTED
	N = 9 4,4,5			FILL: Silty clay, medium plasticity, orange brown mottled brown, trace of sub-rounded and sub-angular igneous gravel.	w≈PL		320 300 400	SCREEN: 10.01kg 0-0.1m, NO FCF
		1						INSUFFICIENT RETURN FOR SCREEN SAMPLE
	N = 10 4,5,5						400 380 320	
		2						
			CI	Silty CLAY: medium plasticity, orange brown, trace of fine to coarse grained, sub-angular and sub-rounded, sedimentary gravel.	w≈PL	Hd		ALUVIAL
	N = 24 6,10,14	3					500 510 530	
				Silty gravelly CLAY: medium plasticity, yellow brown, coarse grained, sub-rounded and sub-angular alluvial gravel.	M			LOW TO MODERATE 'TC' BIT RESISTANCE BAND
		4						TOO GRAVELLY FOR HP TESTING
	N = SPT 8/50mm REFUSAL		-	GREYWACKE: fine to medium grained, grey.	DW	L-M		BALDWIN FORMATION LOW TO MODERATE 'TC' BIT RESISTANCE
		5						
		6		END OF BOREHOLE AT 5.9m				'TC' BIT REFUSAL
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
8

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 378.0m

Date: 7/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, medium plasticity, dark brown, with igneous gravel, trace of fine to medium grained sand, and root fibres.	w<PL			GRASS COVER
	N = 13 3,6,7				w≈PL		>600 >600 >600	APPEARS WELL COMPACTED SCREEN: 10.10kg 0-0.1m, NO FCF INSUFFICIENT RETURN FOR SCREEN SAMPLE
	N = 8 5,3,5	1.4	CI	Silty CLAY: medium plasticity,dark brown, trace of roots.	w≈PL	St- VSt	190 210 210	ALLUVIAL
	N = 18 5,8,10	3				VSt- Hd	390 390 410	
		4	CH	Silty CLAY: high plasticity, brown, trace of fine grained, sub-rounded and sub-angular, alluvial gravel.				
	N = 14 5,7,7						300 380 400	
▶ (5.8)								
	N = SPT 8/150mm REFUSAL	6				St- VSt	180 190 200	
		6.15		as above, but with fine to coarse grained sub-rounded and sub-angular alluvial gravel.				VERY LOW 'TC' BIT RESISTANCE BAND
		7						

Samples: ES, U50, DB, DS

JKGeotechnics

BOREHOLE LOG

Borehole No.
8

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 378.0m
Date: 7/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
ON COMPLETION ▼				■				CH	Silty CLAY: high plasticity, brown, fine to coarse grained, sub-rounded and sub-angular, alluvial gravel.	w≈PL	St- VSt		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 9

1/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 376.5m

Date: 7/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, igneous gravel and root fibres.	w≈PL			GRASS COVER
	N = 13 4,6,7		CI	Silty CLAY: medium plasticity, dark brown, trace of sub-angular and sub-rounded alluvial gravel, and roots.	w<PL	VSt- Hd	400 400 410	ALLUVIAL
	N = 19 6,6,13	1–2				Hd	>600 >600 550	BANDS OF LOW AND MODERATE 'TC' BIT RESISTANCE
	N = 17 9,9,8	3	SM	Silty SAND: fine to coarse grained, orange brown, with clay fines, trace of fine to coarse grained, sub-angular and sub-rounded alluvial gravel.	M	MD		
▶		4	CI	Silty sandy CLAY: medium plasticity, brown, fine to coarse grained sand, trace of fine to coarse grained, sub-angular and sub-rounded, alluvial gravel.	w>PL	St		
▼ ON COMPLETION AND AFTER 1 DAY	N = 12 2,3,9	5		Silty CLAY: medium plasticity, orange brown and brown, with fine to coarse grained sand, trace of coarse grained sub-angular alluvial gravel.			110 150 180	
				Silty gravelly CLAY: low plasticity, brown, fine to coarse grained sub-angular and sub-rounded alluvial gravel, trace of fine to coarse grained sand.				
	N > 13 8,13/ 150mm REFUSAL	6	GP	Sandy GRAVEL: fine to coarse grained sub-angular, alluvial, grey and brown, fine to coarse grained sand, trace of clay fines.	W	MD		
			GP	Silty clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded, alluvial, grey and brown, trace of fine to medium grained sand.				
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
9

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 376.5m
Date: 7/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								GP	Silty clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded, alluvial, grey and brown, trace of fine to medium grained sand.	W	MD		HIGH RESISTANCE BAND
									END OF BOREHOLE AT 7.5m				GROUNDWATER MONITORING WELL INSTALLED TO 5.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.5m TO 5.5m. CASING 2.5m TO 0.1m. 2mm SAND FILTER PACK 2.2m TO 5.5m. BENTONITE SEAL 0.9m TO 2.2m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **10**

1/1

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 378.0m

Date: 7/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, igneous gravel and root fibres.	w≈PL			GRASS COVER SCREEN: 10.07kg 0-0.1m, NO FCF
		N = 13 4,5,8			CI	Silty CLAY: medium plasticity, dark brown and brown, trace of roots and root fibres.	w≈PL	St	150 130	ALLUVIAL TOO FRIABLE FOR HP TESTING INSUFFICIENT RETURN FOR SCREEN SAMPLE
ON COMPLET-ION			1							
		N = 13 5,5,8						VSt	280 220 300	
			2			Silty CLAY: medium plasticity, brown mottled orange brown, trace of fine to coarse grained alluvial gravel.		Hd		
		N = 22 8,10,22	3						430 430 490	
			4		GP	Clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded alluvial, grey brown, trace of fine to coarse grained sand.	M	D		
		N = 49 14,21,28	5							
		N = 26 10,11,15	6							
			7			END OF BOREHOLE AT 6.5m				'TC' BIT REFUSAL

JKGeotechnics

BOREHOLE LOG

Borehole No. 11

1/1

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 382.5m

Date: 7/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./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: Silty clay, medium plasticity, brown, with igneous gravel, trace of root fibres.	w<PL			GRASS COVER APPEARS WELL COMPACTED SCREEN: 10.01kg 0-0.1m, NO FCF INSUFFICIENT RETURN FOR SCREEN SAMPLE
	N = 23 12,12,11				w>PL			
	N = 15 2,5,10	1–2		as above, but trace of slag.			370 180 350	
	N = 12 4,5,7	3	CI	Silty CLAY: medium plasticity, orange brown, trace of fine to coarse grained, sub-angular and sub-rounded alluvial gravel.	w≈PL	St	150 150 160	ALLUVIAL
	N = 19 5,9,10	4–5				VSt	220 220 220	
		5–6		Silty gravelly CLAY: medium plasticity, orange brown and grey, sub-rounded alluvial gravel, trace of fine to coarse grained sand.				
		6		END OF BOREHOLE AT 6.0m				
		7						

Samples: ES, U50, DB, DS

Graphic Log

JKGeotechnics

BOREHOLE LOG

Borehole No. 12

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 382.0m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, medium plasticity, brown, with igneous gravel, trace of fine to medium grained sand, ash and root fibres.	w<PL			GRASS COVER APPEARS WELL COMPACTED
		N = 15 4,8,7							50 550 560	SCREEN: 10.10kg 0-0.1m, NO FCF
			1			as above, but dark brown, without ash.	w≈PL			APPEARS MODERATELY COMPACTED
		N = 7 5,4,3							400 400 450	INSUFFICIENT RETURN FOR SCREEN SAMPLE
			2			FILL: Silty clay, medium plasticity, brown, trace of fine to medium grained sand, fine to coarse grained igneous gravel, and glass fragments.				
		N = 7 4,3,4	3						160 160 160	
					CI	Silty CLAY: medium plasticity, dark brown.	w≈PL	Hd		ALLUVIAL
			4							
		N = 13 7,7,6	5						410 410 >600 >600	
		N = 37 17,15,22	6						>600 >600 >600	
						Silty CLAY: as below				
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
12
2/2

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 382.0m
Date: 8/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					CI	Silty CLAY: medium plasticity, brown mottled orange brown, fine to coarse grained, sub-angular and sub-rounded alluvial gravel.	w≈PL	Hd		
		N = 35 10,18,21							580 570 550	
			8							
		N = SPT 12/50mm REFUSAL	9		GP	Clayey GRAVEL: fine to coarse grained, sub-rounded and sub-angular alluvial, grey and brown.	M	(D)		BANDS OF MODERATE AND HIGH 'TC' BIT RESISTANCE
ON COMPLETION										
			10							
		N = SPT 10/50mm REFUSAL								
			11			Silty clayey GRAVEL: fine to coarse grained, sub-angular and sub-rounded alluvial, brown and grey.	W			
		N = SPT 9/50mm REFUSAL	12							
			13			END OF BOREHOLE AT 13.0m				'TC' BIT REFUSAL
			14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 13

1/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, medium plasticity, yellow brown, trace of igneous gravel, and root fibres.	w≈PL			GRASS COVER APPEARS WELL COMPACTED
		N = 16 5,7,9							400 400 430	
			1							
		N = 14 3,5,9				as above, but trace of ash.			280 280 240	
			2			FILL: Silty clay, medium plasticity, orange brown, trace of igneous gravel.	w≈PL			
			3							
		N = 21 3,11,10			CI	Silty CLAY: medium plasticity, dark brown, trace of sub-angular alluvial gravel.	w≈PL	VSt	260 260 280	ALLUVIAL
			4							
		N = 21 7,10,11			GP	Silty gravelly SAND: fine to coarse grained, brown, sub-angular and sub-rounded alluvial gravel, trace of clay fines.	M	MD		
			5		CI	Silty gravelly CLAY: medium plasticity, grey and brown, fine to coarse grained, sub-angular and sub-rounded alluvial gravel.		Hd		
						Silty CLAY: medium plasticity, dark brown.	w≈PL			
		N = 24 10,11,13	6						430 430 410	
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. **13**

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 381.5m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■				CI	Silty CLAY: medium plasticity, dark brown.	w≈PL	Hd		
									END OF BOREHOLE AT 7.3m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 6.9m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.9m TO 6.9m. CASING 3.9m TO 0.1m. 2mm SAND FILTER PACK 3.5 m TO 6.9m. BENTONITE SEAL 1.6m TO 3.5m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **14**

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.0m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./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: Silty clay, low plasticity, yellow brown, with igneous gravel, trace of fine to medium grained sand, and root fibres.	w<PL			GRASS COVER APPEARS WELL COMPACTED
	N = 19 5,10,9			as above, but medium plasticity, without fine to medium grained sand.	w≈PL		550 510 560	SCREEN: 10.35kg 0-0.1m, NO FCF INSUFFICIENT RETURN FOR SCREEN SAMPLE
		1						
	N = 10 5,4,6			as above, but orange brown and yellow brown.			250 220 260	
		2	CI	Silty CLAY: medium plasticity, dark brown.	w≈PL	St- VSt		ALLUVIAL
	N = 13 3,6,7	3					200 150 150	
			GP	Sandy GRAVEL: fine to coarse grained sub-angular and sub-rounded alluvial, dark brown, fine to coarse grained sand.	M	(MD)		
			GC	Silty clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded alluvial, dark brown.				
		4	CI	Silty CLAY: medium plasticity, dark brown.	w≈PL	VSt		
	N = 18 7,8,10						340 340 350	
		5		as above, but brown, trace of sub-angular and sub-rounded alluvial gravel.				
	N = 31 12,16,15	6	GC	Silty clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded alluvial, dark brown, fine to coarse grained sand.	M	D		OCCASIONAL BANDS OF LOW 'TC' BIT RESISTANCE
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. **14**

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.0m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								GC	Silty clayey GRAVEL: fine to coarse grained sub-angular and sub-rounded alluvial, dark brown, fine to coarse grained sand.	M	D		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No.
15
1/2

Client: UNIVERSITY OF NEW ENGLAND
Project: PROPOSED UNIVERSITY BUILDING
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.5m
Date: 8/6/23 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** N.A.P./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: Silty clay, medium plasticity, brown, with igneous gravel, trace of fine to medium grained sand, and root fibres.	w<PL			GRASS COVER APPEARS WELL COMPACTED
	N = 19 6,10,19						550 550 500	SCREEN: 10.27kg 0-0.1m, NO FCF
		1		as above, but orange brown.	w>PL			INSUFFICIENT RETURN FOR SCREEN SAMPLE
	N = 11 4,5,6						250 250 260	
		2	CI	Silty CLAY: medium plasticity, dark brown.	w>PL	VSt	250 200 310	ALLUVIAL
	N = 9 3,4,5	3					210 190 220	
		4		Silty CLAY: medium plasticity, brown.				
	N = 17 6,7,10					Hd	550 550 500	
		5						
	N = 20 9,10,10	6				VSt	200 140 150	
				Silty CLAY: medium plasticity, brown and orange brown, trace of fine to medium grained sand, and alluvial gravel.				
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. **15**

2/2

Client: UNIVERSITY OF NEW ENGLAND

Project: PROPOSED UNIVERSITY BUILDING

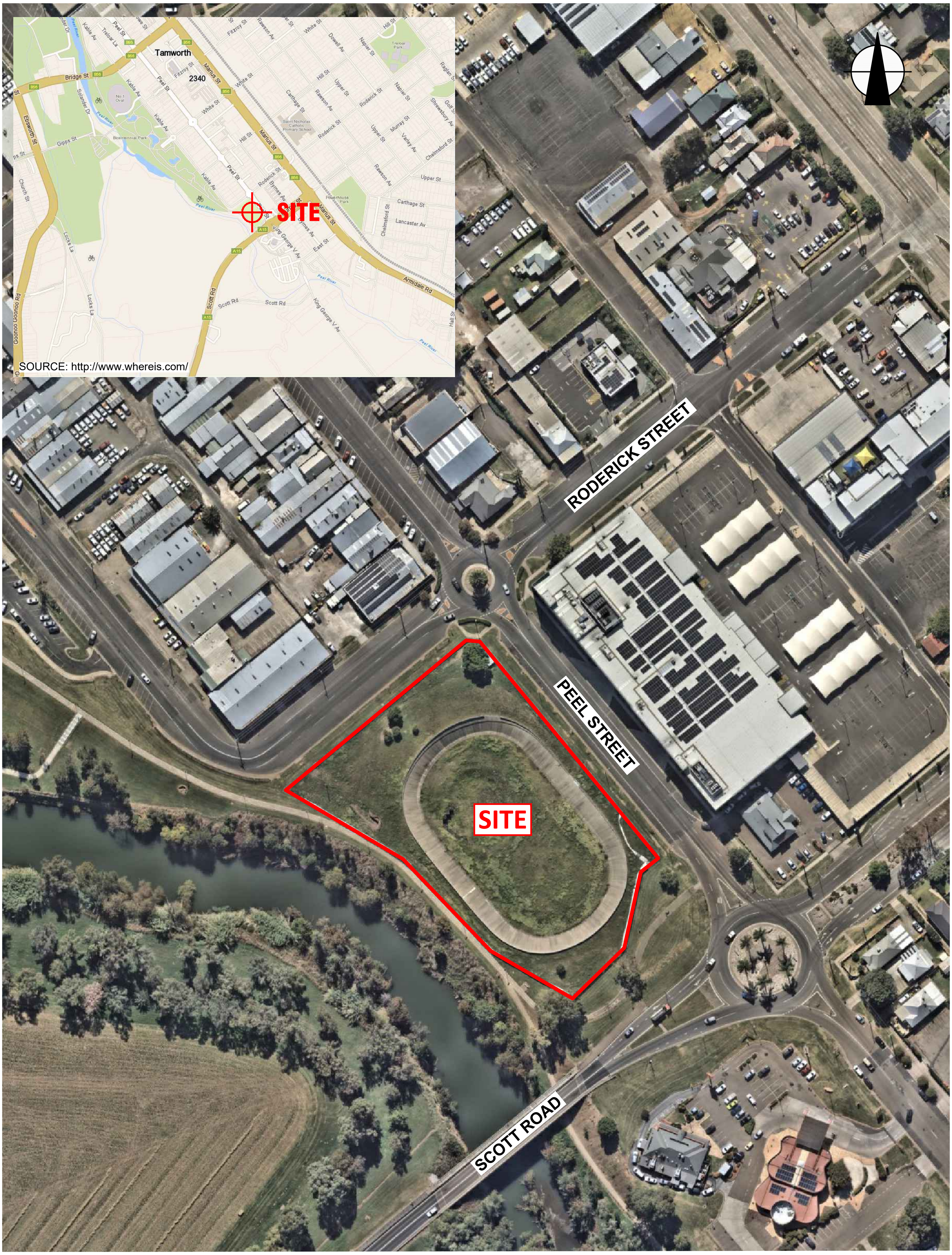
Location: PRINCE OF WALES PARK, PEEL STREET, TAMWORTH, NSW

Job No.: 36020PN **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 380.5m

Date: 8/6/23 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** N.A.P./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
								CI	Silty CLAY: medium plasticity, brown and orange brown, trace of fine to medium grained sand, and alluvial gravel.	w>PL	VSt		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							



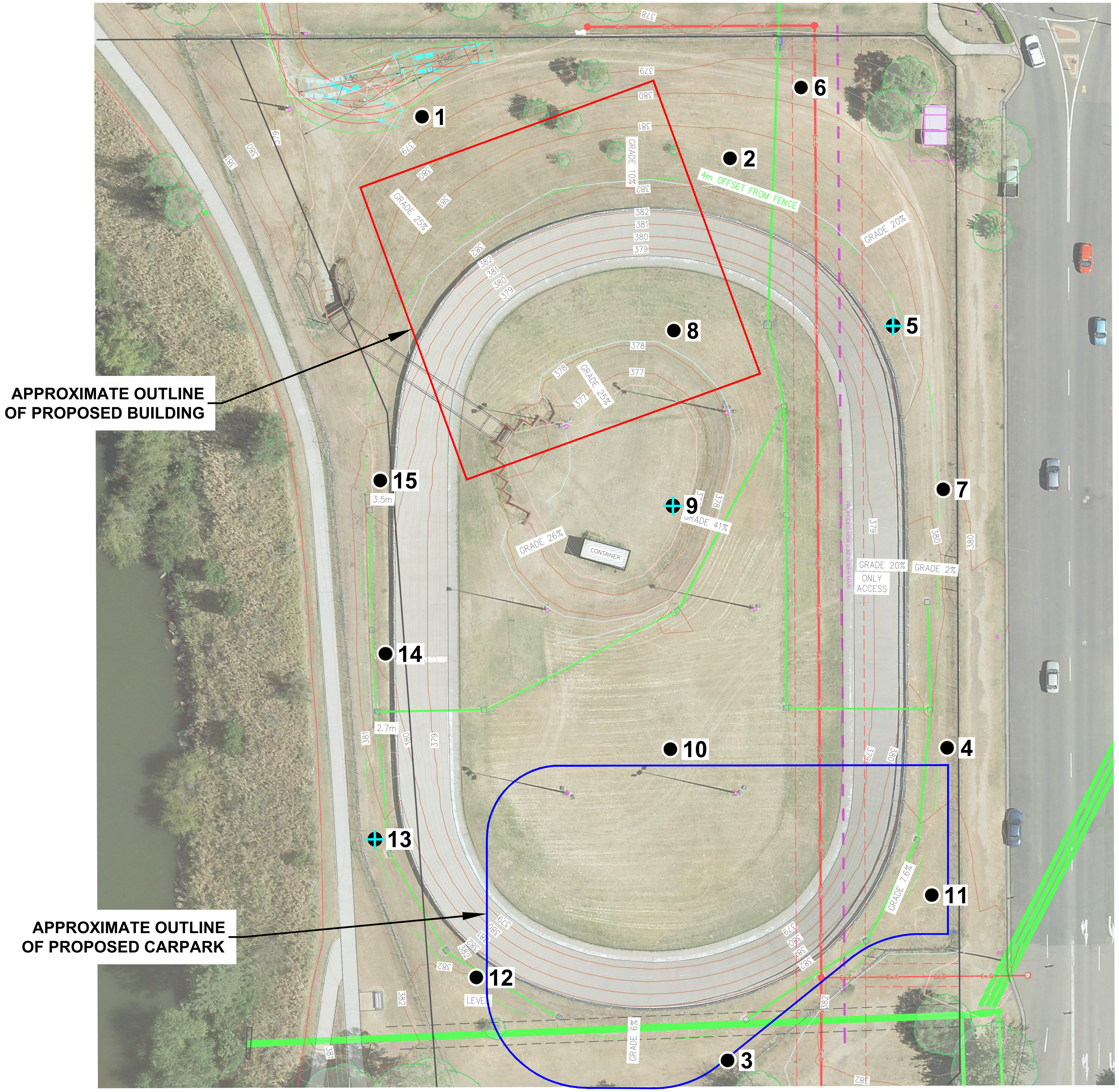
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	PRINCE OF WHALES PARK, PEEL STREET, TAMWORTH, NSW
Report No: 36020PN	Figure No: 1

JKGeotechnics

PLOT DATE: 13/06/2023 3:27:22 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000's\36020PN TAMWORTH\CAD\36020PN.DWG



LEGEND

- ● BOREHOLE
- ⊕ BOREHOLE AND GROUNDWATER MONITORING WELL

SCALE	0 6 12 18 24 30 METRES 1:600 @A3
Title:	INVESTIGATION LOCATION PLAN
Location:	PRINCE OF WHALES PARK, PEEL STREET, TAMWORTH, NSW
Report No:	36020PN
Figure No:	2

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

JKGeotechnics

PLOT DATE: 19/07/2023 1:31:15 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36020PN TAMWORTH\CAD\36020PN.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.

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

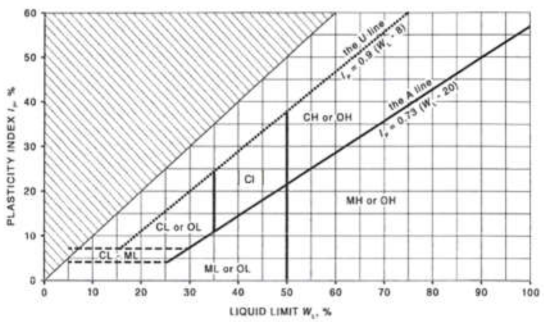
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $Is_{(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>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.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>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.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>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.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

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