REPORT TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Date: 14 June 2023
Ref: 35821YFrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations and additions at Finley Hospital, 24 Dawe Street, Finley, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by signed Consultancy Agreement, Ref: HI22656, and was carried out in accordance with our proposal, Ref: P57854BF, dated 9 December 2022.

We have been provided with the following relevant additional documents:

- Finley Hospital Due Diligence Report prepared by Northrop dated 27 September 2022.
- Survey plan for Finley Hospital with mark-ups of proposed investigation locations.

We understand from the above documents that it is proposed to construct extensions and/or new builds on the existing site, including potential refurbishment of existing buildings. While no concept drawings are currently available, we expect the alterations and additions will be constructed at essentially existing grade and therefore expect only minor excavation or filling will be required. Furthermore, we expect the new building(s) will be similar to the existing buildings, i.e. one to two storey structures, and therefore structural loads are expected to be relatively low for structures of this type.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at the borehole locations. Based on this we have provided comments and recommendations on site preparation and earthworks, excavation conditions and support, site classification, footings, earthquake design, soil aggression, mine subsidence and pavement parameters.

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: EP35821Prpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

The investigation was carried out between 8 and 10 May 2023 and comprised nine boreholes drilled with a track mounted Hanjin DB8 drilling rig operated by Mulligans Drilling. The boreholes were drilled to termination depths between 1.5m and 5.45m below existing surface levels using spiral auger techniques and a 'V' shaped bit. BH1 to BH5 were drilled for the proposed new buildings/extensions while BH6 to BH9 were drilled for the purposes of pavement design and were limited in depth to 2.0m.

The apparent compaction of the fill, strength of the natural clayey soils and relative density of the natural sandy soils was assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer tests completed in cohesive samples recovered in the SPT split tube sampler, as well as remoulded samples taken from the auger.

Groundwater observations were made during and on completion of auger drilling. Monitoring wells comprising Class 18 machine slotted PVC were installed in BH1, BH3 and BH5 and finished with a gatic cover flush with the surface. No longer term monitoring of groundwater levels was carried out.

Selected soil samples were returned to a NATA accredited laboratory, Soil Test Services (STS), for California Bearing Ratio (CBR), moisture content, Atterberg limit and linear shrinkage testing. These results are summarised in the attached STS Tables A, B and C. Selected samples were also sent to Envirolab, another NATA accredited laboratory, for pH, chloride, sulphate and resistivity testing. The test results are summarised in the attached Envirolab Services Certificate of Analysis 323302.

Our geotechnical engineer was present full-time during the fieldwork to set out the borehole locations, direct the electro-magnetic scanning, nominate testing and sampling and to prepare the attached borehole logs.

The borehole locations were positioned as practically close to the locations nominated in the brief. However, they were shifted slightly to either be clear of buried services or onto the shoulders of the road where the nominated locations were in the middle of the road, as agreed. The borehole locations, as shown on the attached Figured 2, were set out by tape measurement from existing surface features. The surface reduced levels (RLs) indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plan (prepared by Walpole Surveying, Ref: 22146 Ver. 1, dated 14 October 2022), and are therefore approximate. The survey datum is the Australian Height Datum (AHD).

For more details of the investigation procedures and their limitations and a glossary of terms and symbols used, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

Finley Hospital is located within the southern Riverina Plain in a landscape characterised by a typical flood plain with near level topography. The site has northern and southern street frontages onto Dawe Avenue and Scoullar Street respectively.

The site itself has a similar topography to the surrounding landscape and is near level terrain. There is no sign that cut and fill earthworks have recently been completed. At the time of fieldwork, the site contained a hospital complex comprising several single-storey brick and weatherboard structures, which were mostly located towards the central portion of the site. These structures appeared in good conditions based on internal and external cursory examination with no visible defects observed. An asphaltic concrete (AC) driveway extended from Dawe Avenue wrapping around the western side of the site while a second driveway ran from the south-eastern corner off Scoullar Street to the existing main building. An existing AC paved on-grade car park was present between the driveway and northern boundary while a gravel carpark was located along the southern boundary adjoining the driveway. The AC pavement over the driveways and paved car parks appeared in good conditions with no visible signs for failures. The site also contains numerous small to large trees.

The site is bounded by Diggers Park to the east which comprised a grassed area with sporadic small to large trees. An unpaved pedestrian footpath extends along the site boundary following the overhead power lines over the open grass field.

The neighbouring western property contained an aged care facility, Finley Regional Care, that has vehicular access via the hospital. The site contained multiple single storey brick and weatherboard structures that generally appeared in good condition based upon a cursory inspection from within the subject site. Ground levels across the site boundaries were the same as the subject site.

3.2 Subsurface Conditions

The 1:250,000 series geological map of Jerilderie (Geological Survey of NSW, Geological Series Sheet SI 55-14) indicates the site to be underlain by Quaternary age riverine deposits comprising unconsolidated clay, silt, sand and gravel including “flood plains and black soil plains”, associated with the Murray Irrigation Area. The major tributary canals, Mulwala Canal and Ulupna Channel, are present about 1.2km north of the site.

The boreholes have disclosed a generalised profile of fill overlying alluvial silty clay and then sandier material at depth. No bedrock was encountered. Reference should be made to the attached borehole logs for specific details at each location. A summary of the subsurface conditions encountered in the boreholes is provided below:

Pavement and Fill

With the exception of BH6, fill was encountered in all boreholes from ground level and extended to depths ranging from 0.2m to 0.8m. The fill material encountered appeared to be generally reworked natural silty clay, although in BH4 a clayey silty fill was encountered. The plasticity of the clayey fill varied between low to high plasticity and contained varying amounts of fine to medium grained igneous gravel and root fibres.

Alluvial Soil

Alluvial clay of medium to high plasticity was initially encountered below the fill in all boreholes. These clays are initially of very stiff to hard strength, however soften to soft to firm strength at depths between 2.0m and 2.5m below existing surface levels in BH2, BH3 and BH4. We note that in BH1 hard strength clays extend to the borehole termination depth of 5m. Sandier soils (sandy clays to silty sands) were encountered across the site although there was no real pattern to the location of these and it was only in BH4 and BH5 that silty sand and clayey sand of loose or medium dense relative density was encountered at depths of 2.5m and 4.5m respectively. The clays also contained varying amounts igneous (basalt) and ironstone gravels.

Groundwater

Groundwater seepage was encountered during drilling in BH1 to BH5 and measured on completion of drilling. Groundwater monitoring wells were also installed and developed in BH1, BH3 and BH5. The measured groundwater levels are summarised in the below. No long-term groundwater monitoring was carried out.

Depth(m) and Reduced Level (AHD) to Top of Measured Groundwater Levels			
Borehole Number	**Groundwater Depth (bgl, m)**	**RL of Groundwater Table (mAHD)**	**Period After Completion of Drilling**
BH1	3.80	104.90	2 days
BH2	4.72	104.28	On Completion of Drilling
BH3	3.72	105.08	2 days
BH4	3.74	105.06	On Completion of Drilling
BH5	3.80	105.00	On Completion of Drilling

3.3 Laboratory Test Results

Based on the shrink-swell, Atterberg limits and linear shrinkage test results, the alluvial clay ranges from low to high plasticity and are of a low to high reactivity . Reference should be made to the attached STS Tables A and C for further details.

The four day soaked CBR tests on clayey samples compacted to 97% to 101% of their Standard Maximum Dry Density (SMDD) returned CBR values between 1.0% and 2.5%. Reference should be made to the attached STS Table B for further details.

The results of the soil aggression testing are tabulated below. Reference should also be made to the attached Envirolab Certificate of Analysis No. 323302.

Borehole	Depth (m)	Material	pH	Sulphates SO_4 (ppm)	Chlorides CL (ppm)	Resistivity (ohm.cm)
BH1	0.2 – 0.5	Silty CLAY	7.5	20	20	18,000
BH1	4.0 – 4.45	Silty CLAY	7.4	570	1,000	1,100
BH2	1.5 – 1.75	FILL: Silty Clay	8.3	160	570	2,000
BH3	1.5 – 1.95	Silty CLAY	8.9	300	100	2,200
BH4	0.5 – 0.65	FILL: Silty Clay	8.9	200	56	2,900
BH5	3.0 – 3.2	Clayey Sand	8.8	160	280	3,600

4 COMMENTS AND RECOMMENDATIONS

Once development details are known, this report must be reviewed to confirm the comments and recommendations are still applicable to the development.

4.1 Site Preparation

Prior to any excavation commencing we recommend that reference be made to the NSW Government "Code of Practice Excavation Work" dated January 2020 or the most recent version at the time of works commencing.

Site preparation is expected to comprise demolition of the existing building(s), removal of trees and stripping of topsoil and/or root affected soils. We also assume that partial demolition of the existing access road may be required.

Following the above site preparation, in areas where no excavation is required, all deleterious or contaminated existing fill should be removed. These stripped materials should be taken offsite as they are not suitable for re-use as engineered fill. However, from a geotechnical perspective (i.e. assuming these materials are not contaminated), existing gravelly fill materials from below existing pavements may be re-used as engineered fill, provided they are separately stockpiled, inspected and approved by the geotechnical engineers. The topsoil and/or root affected soils may also be separately stockpiled and used for subsequent landscaping purposes, or appropriately disposed off site. If the depth of topsoil is critical, then we recommend test pits are excavated to confirm the topsoil thickness. We recommend test pits in lieu of boreholes, as test pits allow a more detailed visual inspection of the soil, compared to boreholes where the soil is assessed from a small diameter borehole, the drill spoil from that borehole and SPT samples.

Trees dry out the surrounding clayey soils in and around their root systems. Removal of trees usually results in an increase in the soil moisture content over time and leads to a swelling of the soils, which may have a detrimental impact on the performance of proposed buildings and paved surfaces founded/supported in the clayey soil profile within the site. Therefore, trees should only be removed where absolutely necessary and as soon as practicable, in order for the moisture content of the clayey subsoils to recover; ideally this would be years in advance of construction though we understand this is usually not practical.

We expect any cut and fill earthworks to be relatively minor and therefore we only expect that fill and alluvial soils will be encountered. The soil materials should be readily excavated using the buckets of conventional earthmoving equipment, such as hydraulic excavators.

The subgrade will comprise clayey soils. Clays are susceptible to softening when exposed to moisture. The clays are likely to provide an unsuitable subgrade if proper site drainage is not implemented during construction. It is therefore important to provide good drainage in order to promote run-off and reduce ponding. Earthworks platforms should be graded to maintain cross-falls during construction. If the clays are exposed to periods of rainfall, softening will result and site trafficability will be poor. Furthermore, the soils may no longer be suitable for re-use as engineered fill or as a suitable subgrade. If softening occurs, the subgrade should be over-excavated to below the depth of moisture softening. The material removed should be replaced with engineered fill. Such work would likely cause delays to the earthworks program. Trafficability may be improved by the use of a sacrificial surface layer of crushed demolition rubble.

4.2 Subgrade Preparation

Earthworks recommendations provided in this report should be complemented by reference to AS3798.

Recommendations for subgrade preparation below stiffened raft slabs and slabs on ground are outlined below. Slab-on-ground (other than stiffened raft slabs) should also be isolated from the footings of the building (i.e. designed as 'floating') and other structures. Prior to the placement of engineered fill, pavements or slabs on grade we recommend that the following subgrade preparations be completed:

1. All root affected or deleterious fill or topsoil must be stripped. There may be an extensive zone of root affected soil where trees have been removed. These stripped materials will not be suitable for use as engineered fill but may be suitable for landscaping purposes.
2. Where existing uncontrolled fill is present and the proposed building will be formed over areas of existing fill, then the existing fill must be excavated to the natural subgrade. We recommend excavation of the fill extend at least 1m beyond the building footprint.
3. Following the above, the entire subgrade should be proof rolled with 6 passes of an at least 8 tonne smooth drum roller used in static or non-vibratory mode of operation. The purpose of the proof rolling is to detect any soft or heaving areas.
4. The final pass of proof rolling should be undertaken in the presence of an experienced geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction. Care should be taken not to over-compact clayey subgrade areas.
5. If dry conditions prevail at the time of construction, the clay subgrade may become desiccated or have shrinkage cracks prior to pouring any concrete slabs. If this occurs then the subgrade must be watered and rolled until the cracks disappear. This should be completed immediately prior to pouring concrete.

Unstable subgrade detected during proof rolling should be locally excavated down to a sound base and replaced with engineered fill. Any fill placed to raise site levels should also be engineered fill. The subgrade should be contoured to promote the shedding of water from the platform or allowance for drainage as the subgrade is susceptible to softening on contact with water. In this regard good site drainage will be critical and preferably the earthworks are programmed to be carried out during the drier period of the year to help reduce difficulties during earthworks.

Engineered Fill

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

Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, it is likely that lighter compaction equipment will be used. Where this is the case the loose layer thickness will probably need to be reduced and probably be limited to 100mm. Layer thickness may be varied provided the required compaction specification is uniformly achieved over the full layer thickness. Maximum particle sizes must not exceed one third the loose layer thickness.

From a geotechnical perspective, the existing fill and alluvial clays at the site may be acceptable for re-use as engineered fill on condition that the soils used are clean (i.e. free of organics and oversized inclusions) and free from contaminants. However, soils with a high silt content are likely to be difficult to work as they are highly sensitive to changes in moisture content, such as the clayey silt fill in BH4. Thus, if particularly silty soils are encountered, where possible we recommend that they not be used as engineered fill. These site won clayey soils should be compacted in maximum loose layers of about 200mm to a density strictly between 98% and 102% of SMDD and at a moisture content within 2% of their Standard Optimum Moisture Content (SOMC). Where possible, all clay fill should preferably be used in the lower fill layers. Thus, the use of clay materials for engineered fill will entail more rigorous earthwork supervision and compaction control, time for possible moisture conditioning and hence, possibly a greater eventual cost for earthworks. Consideration must also be made by the building designer of the greater reactive potential of new fills comprising reactive clays as opposed to existing clayey soils, as discussed in Section 4.3 Footings below.

Compaction Control

Density tests should be regularly carried out on engineered fill to confirm the above specifications are achieved. Density tests should be carried out at the frequencies outlined in AS3798 (Table 8.1) for the volume of fill involved. Within the proposed building footprint and particularly if the engineered fill will be supporting structural loads, the fill must be placed under Level 1 supervision, as defined in AS3798-2007. In areas where engineered fill will not be supporting structural loads, a reduced Level 2 control of fill compaction may be adopted. Any areas of insufficient compaction will require reworking and retesting to confirm the required specification has been achieved. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.3 Footings

Due to the possibility of abnormal moisture conditions due to existing pavements and trees, we consider that the site classifies as Class 'P' in accordance with AS2870-2011 ‘Residential Slabs and Footings’. If the footings are designed to be founded below the fill on the inferred natural alluvial soils, consideration must still be given to the potential for the natural silty clays to shrink and swell with changes in moisture content. In our opinion, any new footings may be designed on the assumption that shrink-swell movements of the alluvial silty clays similar to Class ‘H2-D’ type movements will occur. Notwithstanding this, we recommend further advice on the potential impact of shrink-swell movements is provided once the location of any new structures is known, particularly given the presence of existing trees.

We note that in the strictest sense AS2870 does not apply to developments such as this, however it provides a useful guide for footing design on reactive clay sites. Reference may also be made to AS2870 for design, construction, performance criteria and maintenance precautions on reactive clay sites.

Our preference is for all new buildings to be uniformly founded on the upper alluvial clays. We do not consider the existing fill as a suitable founding stratum.

Given the highly reactive soils present across the site, we expect a stiffened raft slab will likely be the most appropriate footing system for the site. A strip footing system is unlikely to be appropriate for the site and be in accordance with AS2870. Furthermore, we do not recommend footings extend deeper than 1m below existing surface levels due to the presence of the deeper softer soils than may consolidate if surcharged by any new footings. The following criteria are recommended for design and construction of these footings:

Edge and internal beams (or strip/pad footings if deemed appropriate) may be proportioned for an allowable bearing pressure (AEBP) of 150kPa when founded within natural clay of at least very stiff strength. It should be noted that soft to firm soils were encountered at depths of about 2m to 2.5m and footings founded in the very stiff clay must be founded sufficient depth above these softer clays such that they are not within the zone of influence of the footing. The zone of influence of the footing is typically considered to be 1.5 x the width of the footing (or edge/internal thickenings in the case of a stiffened raft slab). All loose or softened debris should be cleaned from the base of all beams/footings piers prior to concreting. All footings should be poured immediately after excavation, cleaning and inspection by a geotechnical engineer.

The subgrade preparation recommendations provided above in Section 4.2 should be carefully followed where raft slabs are adopted. The designer should also note that if there are trees within the footprint of the proposed buildings, these will affect the performance of footings on clay soils. A potential 'abnormal moisture condition' may exist where the trees are to be removed and consideration must be given to this in the design.

4.4 Hydrogeological Considerations

We do not expect any excavations to encounter the groundwater table. Furthermore, we do not expect stormwater infiltration systems will be a viable option for disposal of stormwater, particularly given the low permeability subsurface profile. As such, any captured stormwater will need to be disposed of appropriately to Council's stormwater system.

Since the proposed development will be constructed at the ground surface with no basement proposed, drainage will only be required to control any surface water and direct it into the stormwater system.

4.5 Retention Systems

Given the uniformity of the levels across the site, we do not expect any retention systems will be required, other than possible landscaping walls. For cantilevered gravity type retaining walls supporting soil materials (if required and assuming they are set-back sufficient distance from the site boundaries such that any retained excavation on site will not result in movements induced below adjoining structures), we recommend that walls can be designed on the basis of an active earth pressure co-efficient (K_a) of 0.35 where some wall movements are tolerable and assuming a horizontal backfill surface. Where movements behind the wall must be limited, an earth pressure co-efficient (K) of 0.55 may be adopted. A bulk unit weight of 20kN/m^3 should be adopted for the soil profile. Surcharge loads (e.g. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the above earth pressure coefficient. The retaining walls should be designed as drained, otherwise appropriate hydrostatic pressures must be added to the above earth pressures.

Any backfill behind free standing cantilever retaining walls should comprise engineered fill in order to reduce post construction settlements. Where temporary batters have been adopted to allow the construction of the retaining wall, compaction of engineered fill behind retaining walls can be difficult and time consuming to carry out effectively. In addition, it is inevitable that even with good quality control and compaction that some post construction settlements will occur. Post construction settlements can cause adverse impacts on paving, landscaped retaining walls or other structures and services founded on or within the backfill. If the occurrence of some post construction settlements may be problematic, we recommend that further advice be obtained from the geotechnical engineers. However, due to the limited space that may be available behind the walls, our preference for backfill behind free standing cantilevered retaining walls is to backfill using a single sized durable gravel, such as 'blue metal' or crushed concrete gravel (free of fines). These granular materials do not require significant compactive effort and provide better long term performance in regard to settlement than soil materials. A non-woven geotextile filter fabric should be placed over the cut faces prior to backfilling and then over the top surface of the gravel in order to prevent subsoil erosion. A clay capping layer should be provided above the free draining backfill material to reduce the likelihood of surface water entering the backfill and surcharging the retaining walls.

4.6 Earthquake Design Classification

Based upon AS1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia", the following design parameters may be adopted:

- Hazard Factor (Z) = 0.10;
- Class D_e – Deep or soft soil site.

4.7 Soil Aggression

Based on the soil aggressivity testing, the soils and weathered rock would be classified as having a 'Mild' exposure classification for concrete piles in accordance with Table 6.4.2(c) of AS2159-2009 'Piling – Design and Installation'. For steel piles, the soils would be classified as 'Moderate' in accordance with Table 6.5.2(c) of AS2159-2009.

4.8 Mine Subsidence

Based upon the NSW Government ePlanning Spatial Viewer and at the date of this report, the site does not fall within an identified Mine Subsidence District and therefore does not require approval from Subsidence Advisory NSW.

4.9 Pavement Design Parameters

We presume that new access roads and external on-grade car parks will be constructed and will consequently require pavement design. Any pavement subgrade should be prepared as recommended in Section 4.2.

The CBR testing of soil samples returned very low CBR values ranging from 1.0% to 2.5%. We consider that design of the pavement thickness should be based on a soaked CBR of 1.0%, or a modulus of subgrade reaction of 10kPa/mm (750mm plate). However, given the potential implications of such a low CBR value, consideration may be given to undertaking further CBR testing at specific pavement locations (once known) which may allow optimisation of the pavement design. Where fill is used to raise site levels, or replace unsuitable subgrade, pavement design may reflect four day soaked CBR value of the imported material provided the placed thickness of material is sufficient.

For pavements constructed over the existing soils on site, the measured CBR value is typically very low and this must be taken into account during pavement design. Consideration could be given to some form of subgrade improvement to reduce the thickness of the pavement materials. The following are those options available for pavement design.

1. Design the pavement based on a CBR of 1.0%. This is likely to provide an uneconomical pavement thickness design.

OR

2. Provide an appropriate select fill layer as part of the overall pavement thickness. The select fill should be well graded ripped or crushed rock or an approved good quality granular material with a minimum soaked CBR value of 10%. This will help reduce the overall thickness of the pavement.

OR

3. Stabilise the subgrade to a depth of 200mm to 300mm by the addition of lime. When thoroughly mixed and re-compacted to a minimum of 98% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved. As a guide, the addition of approximately 4% lime by dry weight of clay should result in a soaked CBR value of around 5% or an equivalent subgrade reaction modulus of 40kPa/mm. This should, however, be confirmed by laboratory testing. If lime stabilisation is undertaken, an experienced contractor with appropriate equipment should complete it. Appropriate dust suppression will be required, particularly given the proximity of the existing Hospital. This approach will similarly help reduce the overall thickness of the pavement.

Surface and subsoil drainage should be provided on both sides of the pavements to prevent moisture ingress into the subgrade and pavement. The subsoil drains should have an invert level of at least 300mm below the adjacent subgrade level and be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of ponding in the base of the drain. In addition, the surface of the adjacent pavement subgrade should be provided with a uniform cross fall towards the subsoil drain to assist with drainage.

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

4.10 Further Geotechnical Input

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

- Review of the recommendations provided in this report once development details are known.
- Inspection of proof rolling of the subgrade;
- Density testing of any fill placed;
- Inspection of all footing excavations by a geotechnical engineer to confirm that the design bearing pressures have been achieved prior to the placement of concrete.

5 GENERAL COMMENTS

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

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

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

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

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Finley Hospital, 24 Dawe Avenue, Finley, NSW

Report No.: 35821BF - A
Report Date: 31/05/2023
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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	1.50 - 1.95	16.4	49	19	30	13.0
2	0.50 - 0.95	15.9	63	19	44	16.5*
4	1.50 - 1.70	17.8	49	15	34	13.0*
5	1.50 - 1.75	17.9	23	15	8	6.0**

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: 16/05/2023.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage curled.
- ** Denotes Linear Shrinkage cracked.

NATA
NATA Accredited Laboratory Number:1327

31/05/2023
Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Finley Hospital, 24 Dawe Avenue, Finley, NSW

Report No.: 35821BF - B
Report Date: 31/05/2023

BOREHOLE NUMBER	BH 6	BH 7	BH 8	BH 9
DEPTH (m)	0.00 - 1.00	0.70 - 1.50	0.00 - 0.80	0.50 - 1.50
Surcharge (kg)	4.5	4.5	4.5	4.5
Maximum Dry Density (t/m^3)	1.63 STD	1.61 STD	1.72 STD	1.61 STD
Optimum Moisture Content (%)	17.5	21.5	17.8	18.9
Moulded Dry Density (t/m^3)	1.60	1.57	1.69	1.59
Sample Density Ratio (%)	98	98	98	98
Sample Moisture Ratio (%)	99	101	98	97
Moisture Contents				
Insitu (%)	15.3	19.2	22.8	16.3
Moulded (%)	17.3	21.7	17.4	18.4
After soaking and After Test, Top 30mm(%)	33.8	33.7	27.5	33.5
Remaining Depth (%)	26.0	26.2	20.9	22.5
Material Retained on 19mm Sieve (%)	0	0	0	0
Swell (%)	3.0	3.0	1.5	2.0
C.B.R. value: @2.5mm penetration	2.0	1.0	2.5	2.0
@5.0mm penetration				

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate notes for soil descriptions
- BH 6 & 7 had insufficient sample mass supplied to complete a 4-point compaction curve.
- BH 8 had the material recycled to complete a 4-point compaction curve.
- Date of receipt of sample: 16/05/2023.
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.

NATA
NATA Accredited Laboratory Number:1327

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the items tested or sampled.

31/05/2023
Authorised Signature / Date
(D. Treweek)

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

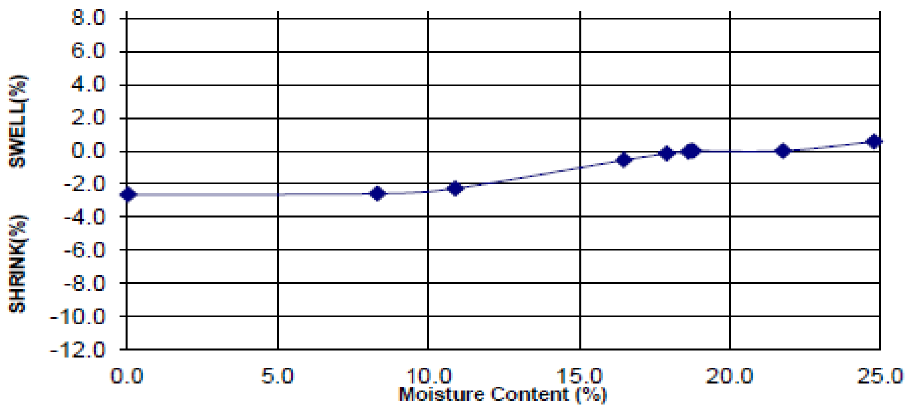
TABLE C
SHRINK - SWELL TEST REPORT
TEST METHOD: AS1289 7.1.1

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Finley Hospital, 24 Dawe Avenue, Finley, NSW

Report No.: 35821BF - C
Report Date:

Borehole No.: 4		Depth: 2.00 - 2.44m	
MOISTURE CONTENT (SWELL)		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH	
BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
21.8%	24.8%	180,220 kPa	150 kPa
LOAD	SETTLEMENT UNDER LOAD BEFORE SATURATION	SWELL ON SATURATION	SHRINKAGE
25	-2.0%	0.6%	2.5%

SHRINK SWELL GRAPH



SHRINK SWELL INDEX
1.57 %/pF

Notes: Sampled and supplied by client. Sample tested as received.
- Suction Value used in calculation = 1.8pF
- Volume Change Coefficient (α) was assumed = 2
- Visually estimated inclusions by volume = 0-5%
- Shrinkage Cracking = Moderate
- Soil Crumbling = none
- Date of receipt of sample: 16/05/2023.

NATA
NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.

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

115 Wicks Road
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SOIL TEST SERVICES

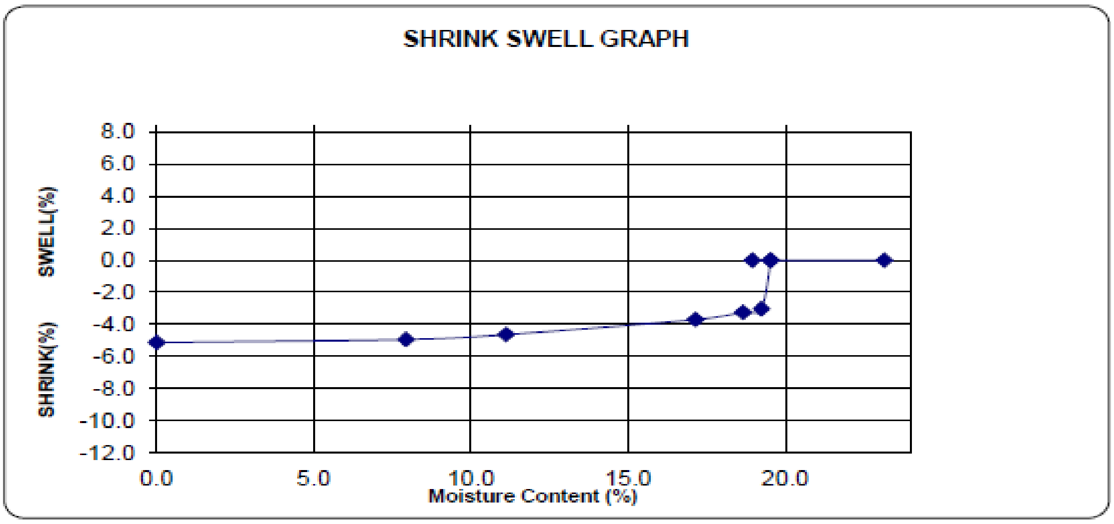
ABN 43 002 145 173

TABLE C
SHRINK - SWELL TEST REPORT
TEST METHOD: AS1289 7.1.1

Client: JK Geotechnics
Project: Proposed Alterations and Additions
Location: Finley Hospital, 24 Dawe Avenue, Finley, NSW

Report No.: 35821BF - C
Report Date:

Borehole No.: 5		Depth: 2.00 - 2.44m	
MOISTURE CONTENT (SWELL)		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH	
BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
18.9%	23.2%	240,270 kPa	115,140 kPa
LOAD	SETTLEMENT UNDER LOAD BEFORE SATURATION	SWELL ON SATURATION	SHRINKAGE
25	-1.8%	0.0%	4.8%



SHRINK SWELL INDEX
2.69 %/pF

Notes: Sampled and supplied by client. Sample tested as received.
- Suction Value used in calculation = 1.8pF
- Volume Change Coefficient (α) was assumed = 2
- Visually estimated inclusions by volume = 0-5%
- Shrinkage Cracking = Moderate
- Soil Crumbling = none
- Date of receipt of sample: 16/05/2023.

NATA
NATA Accredited Laboratory Number:1327

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

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS 323302

Client Details

Client	JK Geotechnics
Attention	Cho Sum Yip
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**35821BF, Finley**
Number of Samples	6 Soil
Date samples received	16/05/2023
Date completed instructions received	16/05/2023

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	23/05/2023
Date of Issue	19/05/2023

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		323302-1	323302-2	323302-3	323302-4	323302-5
Your Reference	UNITS	BH1	BH1	BH2	BH3	BH4
Depth		0.2-0.5	4-4.45	1.5-1.75	1.5-1.95	0.5-0.65
Date Sampled		09/05/2023	09/05/2023	09/05/2023	09/05/2023	09/05/2023
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	18/05/2023	18/05/2023	18/05/2023	18/05/2023	18/05/2023
Date analysed	-	18/05/2023	18/05/2023	18/05/2023	18/05/2023	18/05/2023
pH 1:5 soil:water	pH Units	7.5	7.4	8.3	8.9	8.9
Chloride, Cl 1:5 soil:water	mg/kg	20	1,000	570	100	56
Sulphate, SO4 1:5 soil:water	mg/kg	20	570	160	300	200
Resistivity in soil*	ohm m	180	11	20	22	29

Misc Inorg - Soil		
Our Reference		323302-6
Your Reference	UNITS	BH5
Depth		3-3.2
Date Sampled		10/05/2023
Type of sample		Soil
Date prepared	-	18/05/2023
Date analysed	-	18/05/2023
pH 1:5 soil:water	pH Units	8.8
Chloride, Cl 1:5 soil:water	mg/kg	280
Sulphate, SO4 1:5 soil:water	mg/kg	160
Resistivity in soil*	ohm m	36

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	323302-2
Date prepared	-			18/05/2023	1	18/05/2023	18/05/2023		18/05/2023	18/05/2023
Date analysed	-			18/05/2023	1	18/05/2023	18/05/2023		18/05/2023	18/05/2023
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	7.5	7.6	1	99	105
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	20	20	0	112	#
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	20	20	0	110	#
Resistivity in soil*	ohm m	1	Inorg-002	[NT]	1	180	180	0	[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

Percent recovery is not applicable due to the high concentration of the element/s in the sample/s. However an acceptable recovery was obtained for the LCS.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1/1
SDUP1: 0-0.2m

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.7m
Date: 9/5/23 **Datum:** AHD
Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

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 to medium plasticity, brown, trace of fine grained igneous gravel, and root fibres.	w<PL			GRASS COVER
								SCREEN: 10.75kg 0-0.2m, NO FCF
	N > 20 8,11,9/ 50mm REFUSAL		CI-CH	Silty CLAY: medium to high plasticity, brown and red brown, trace of fine grained sand, and root fibres.	w<PL	Hd	>600 >600 >600 >600	ALLUVIAL
		1		Silty CLAY: medium to high plasticity, brown, trace of fine grained sand, and fine to medium grained igneous gravel.				
	N = 26 5,12,14						>600 >600 >600	
		2						
				Silty CLAY: medium to high plasticity, light grey mottled orange brown, trace of fine grained sand, and fine to medium grained igneous gravel.			450 500 550	HP TESTING ON REMOULDED SAMPLE
		3						
				as above, but grey and brown.			>600 >600	
2 DAYS AFTER COMPLETION		4						
	N = 22 8,8,14						>600 >600	
								GROUNDWATER MONITORING WELL INSTALLED TO 4.9m.
		5		END OF BOREHOLE AT 5.0m				CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.9m TO 2.5m. CASING 2.5m TO 0.12m. 2mm SAND FILTER PACK 4.9m TO 2.5m. BENTONITE SEAL 2.5m TO 2.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
		6						
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1/1
SDUP3: 0-0.2m

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.8m
Date: 9/5/23 **Datum:** AHD
Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

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 to high plasticity, red brown, trace of fine grained igneous gravel and root fibres.	w>PL			GRASS COVER SCREEN: 10.0kg 0-0.2m, NO FCF
	N = 27 5,12,15		CH	Silty CLAY: high plasticity, brown, trace of fine to medium grained igneous gravel, fine grained ironstone gravel, and root fibres.	w<PL	Hd	>600 >600 >600	ALLUVIAL
	N = 14 3,6,8	1					480 530 550	
		2		Silty CLAY: medium to high plasticity, brown, trace of fine to medium grained igneous gravel and root fibres.	w<PL	(F)		
	N = 6 2,3,3	3	CL-CI	Silty CLAY: low to medium plasticity, light grey mottled orange brown and red brown, trace of sand.	w>PL	St	120 170 180	
ON COMPLETION		4		Sandy silty CLAY: low to medium plasticity, light grey mottled orange brown, fine grained sand, trace of shell and muscovite fragments.		S-F	40 30 50	HP TESTING ON REMOULDED SAMPLE
		5		END OF BOREHOLE AT 4.95m				
		6						
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No. **3**

1/1

SDUP2: 0-0.2m

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.8m

Date: 9/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, low to medium plasticity, brown, trace of fine to medium grained sand, fine grained igneous gravel, and root fibres.	w>PL			GRASS COVER
	N = 10 3,3,7		CI-CH	Silty CLAY: medium to high plasticity, red brown, trace of fine grained igneous gravel, and root fibres.	w>PL	VSt	250 320 350	SCREEN: 10.48kg 0-0.2m, NO FCF ALLUVIAL
	N = 10 3,4,6	1–2		Silty CLAY: medium to high plasticity, brown mottled red brown, trace of fine to medium grained igneous gravel, ash and root fibres.	w≈PL	Hd	430 500 550	
2 DAYS AFTER COMPLETION		2.5–3	CL-CI	Silty CLAY: low to medium plasticity, brown and orange brown, trace of fine grained sand.	w>PL	F		
		3.5–4					40 40 50	HP TESTING ON REMOULDED SAMPLE
ON COMPLETION	N = 9 0,4,5	5					60 80 110	GROUNDWATER MONITORING WELL INSTALLED TO 4.88m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 4.88m TO 2.58m. CASIN
		6–7		END OF BOREHOLE AT 5.45m				2.58m TO 0.12m. 2mm SAND FILTER PACK 4.88m TO 2.2m. BENTONITE SEAL 2.2m TO 1.8m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

BOREHOLE LOG

Borehole No. 4

1/1

SDUP4: 0-0.2m

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.8m

Date: 9/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Clayey Silt, low plasticity, red brown, with fine grained sand, trace of root fibres.	w<PL			GRASS COVER
			CI-CH	Silty CLAY: medium to high plasticity, red brown, trace of fine to medium grained igneous gravel.	w<PL	Hd		SCREEN: 10.12kg 0-0.2m, NO FCF ALLUVIAL
	N = 18 3,9,9	1						
	N = 9 3,4,5			Silty CLAY: medium to high plasticity, brown mottled light grey and orange brown, trace of fine grained igneous and ironstone gravel, and root fibres.			450 350 120	
		2	CL-CI	Silty CLAY: low to medium plasticity, light grey mottled orange brown, with fine to medium grained sand.	w>PL	VSt (S-F)	250 230 200	HP TESTING ON REMOULDED SAMPLE
		3						
ON COMPLET-ION	N = 2 2,2,0		CL	Sandy CLAY: low plasticity, brown, fine to coarse grained sand, trace of shell fragments.	w>PL	F	80 110 90	
		4						
	N = 13 3,5,8		SM	Silty SAND: fine to coarse grained, brown, trace of shell fragments.	W	MD		
		5		END OF BOREHOLE AT 4.95m				
		6						
		7						

SAMPLES: ES, U50, DB, DS

Graphic Log

JKGeotechnics

BOREHOLE LOG

Borehole No. 5

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.8m

Date: 10/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty sand, fine to coarse grained, brown, trace of fine grained sandstone gravel, and clay.	M			GRASS COVER
			CI-CH	Silty CLAY: medium to high plasticity, red brown and brown, trace of fine to medium grained sand fine to medium grained igneous gravel, ash and root fibres.	w≈PL	VSt		SCREEN: 11.45kg 0-0.2m, NO FCF
	N = 9 2,3,6						320 350 350	ALLUVIAL
		1	CI	Silty CLAY: medium plasticity, light grey, trace of ash and root fibres.	w>PL			
	N = 11 3,4,7						230 320 380	
		2					420 450 480	HP TESTING ON REMOULDED SAMPLE
			SC	Clayey SAND: fine to coarse grained, brown, fine to medium plasticity clay.	M	L		
		3						
ON COMPLET-ION	N = 8 3,4,4				W			
		4	CI-CH	Silty CLAY: medium to high plasticity, light grey mottled orange brown, trace of root fibres.	w>PL	St- VSt	120 250 310	
			SC	Clayey SAND: fine to coarse grained, brown, trace of fine to medium grained igneous gravel.	W	MD		
	N = 11 4,6,5							
		5		END OF BOREHOLE AT 5.0m				
		6						
		7						

SAMPLES: ES, U50, DB, DS

Graphic Log

JKGeotechnics

BOREHOLE LOG

Borehole No. 6

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.6m

Date: 8/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		CL-CI	Sandy silty CLAY: low to medium plasticity, red brown, fine to medium grained sand, with fine to medium grained igneous gravel, and root fibres.	w<PL	(Hd)		GRASS COVER SCREEN: 15.0kg 0-0.1m, NO FCF SCREEN: 11.45kg 0.1-0.5m, NO FCF
						1							SCREEN: 6.75kg 1.0-1.5m, NO FCF
									END OF BOREHOLE AT 1.5m				
						2							
						3							
						4							
						5							
						6							
						7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
7
1/1

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.9m

Date: 8/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

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: Sandy silty clay, low to medium plasticity, brown, fine to medium grained sand, with fine to medium grained igneous, claystone and sandstone gravel.	w<PL			GRASS COVER
					CL-CI	Sandy silty CLAY: low to medium plasticity, red brown, fine to medium grained sand, trace of fine grained igneous gravel.	w<PL	Hd	>600 >600 >600	SCREEN: 10.4kg 0-0.2m, NO FCF SCREEN: 6.01kg 0.2-0.7m, NO FCF SCREEN: 10.11kg 0.7-1.0m, NO FCF
			1			as above, but brown.				HP TESTING ON REMOULDED SAMPLE
						END OF BOREHOLE AT 1.5m				
			2							
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. **8**

1/1

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.6m

Date: 8/5/23 **Datum:** AHD

Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

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 to high plasticity, brown, trace of fine to medium grained sand, fine to medium grained igneous gravel, and root fibres.	w>PL			GRASS COVER POSSIBLY NATURAL SCREEN: 10.66kg 0-0.2m, NO FCF
			1		SC	Sandy silty CLAY: low to medium plasticity, brown, fine to medium grained sand trace of fine grained igneous gravel.	w≈PL	(St)		ALLUVIAL
			2			END OF BOREHOLE AT 2.0m				
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. 9

1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW

Job No.: 35821BF **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 108.6m
Date: 10/5/23 **Datum:** AHD
Plant Type: HANJIN DB8 **Logged/Checked by:** C.S.Y./O.F.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Sandy silt, low plasticity, red brown, fine to medium grained sand, trace of fine grained sand, clay nodules and root fibres.	w<PL			GRASS COVER SCREEN: 10.01kg 0-0.2m, NO FCF
						1		CL-CI	Silty CLAY: low to medium plasticity, brown, trace of fine grained sand, and clay nodules.	w<PL	(VSt)		ALLUVIAL
						2 3 4 5 6 7			END OF BOREHOLE AT 1.5m				

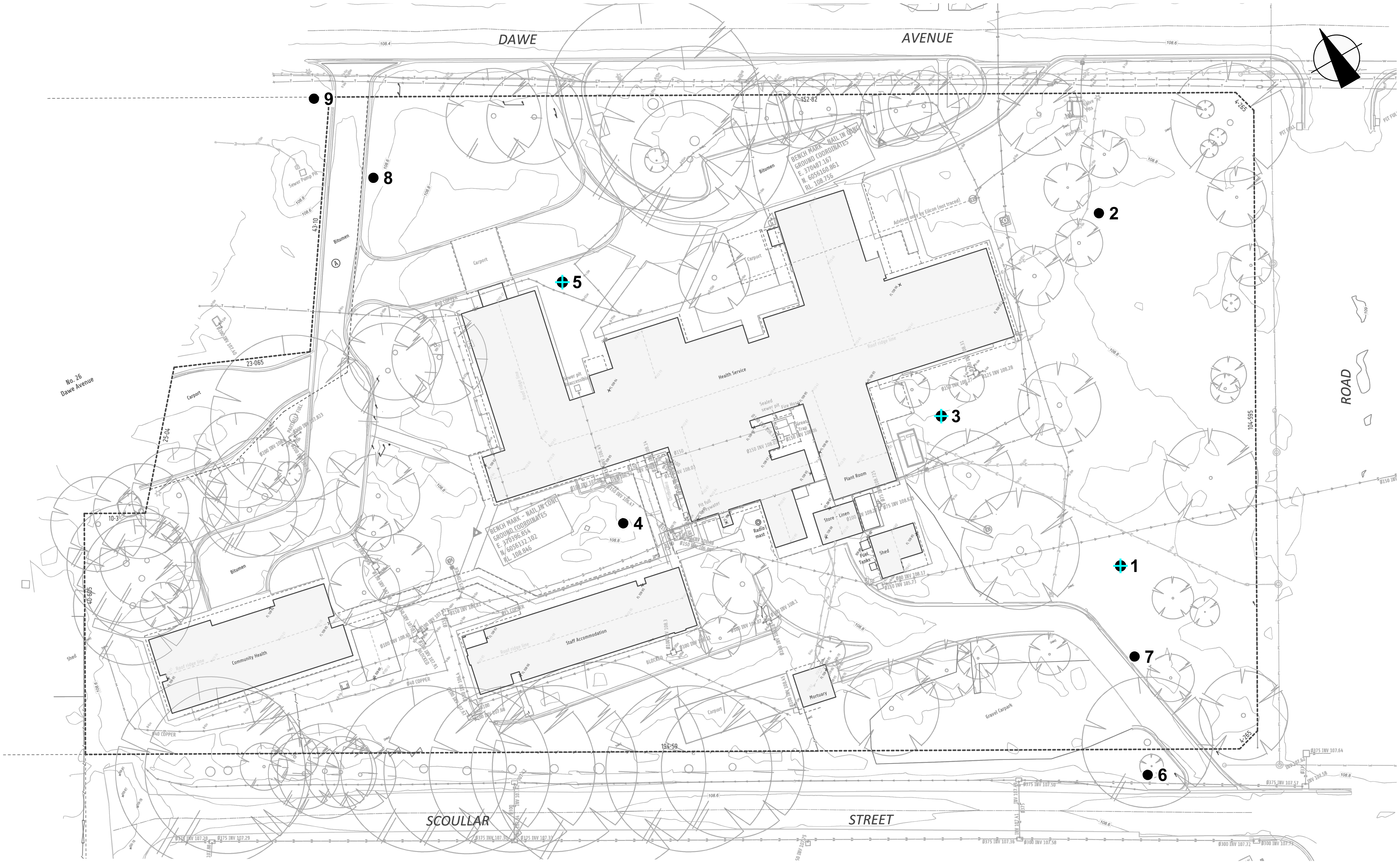
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW
Report No:	35821BF
Figure No:	1

JKGeotechnics

PLOT DATE: 16/05/2023 10:54:36 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\35000'S\35821BF FINLEY\CAD\35821BF.DWG



LEGEND

- BOREHOLE
- BOREHOLE AND GROUNDWATER MONITORING WELL

0 6 12 18 24 30

SCALE 1:600 @A3 METRES

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

Title:	TEST LOCATION PLAN
Location:	FINLEY HOSPITAL, 24 DAWE AVENUE, FINLEY, NSW
Report No:	35821BF
Figure No:	2

JKGeotechnics

PLOT DATE: 16/05/2023 10:56:20 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\35000'S\35821BF FINLEY\CAD\35821BF.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

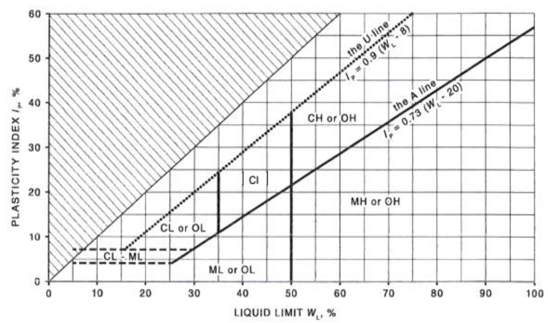
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Abbreviations Used in Defect Description

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