REPORT TO

HI

ON

ADDITIONAL GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

MOREE HOSPITAL, 35 ALICE STREET, MOREE, NSW

Date: 12 September 2023

Ref: 35092UR2rpt

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ATTACHMENTS

Douglas Partners Pty Ltd 'Material Test Report' (224178.00-1, dated 7 August 2023)

Borehole Logs 101 to 108

Figure 1: Site Location Plan

Figure 2: Test Location Plan

Figure 3: Graphical Borehole Summary (BH6, BH101, BH1, BH104, BH102, BH105, BH106, BH107)

Report Explanation Notes

1 INTRODUCTION

This report presents the results of an additional geotechnical investigation for the proposed alterations to Moree Hospital, 35 Alice Street, Moree, NSW. The location of the site is shown in Figure 1. The additional investigation was commissioned on behalf of Health Infrastructure (HI) by Ateca Vucago (Savills Project Management Pty Ltd) in Aconex correspondence (Ref. Savills-GCOR-000586) dated 14 July 2023. The commission was on the basis of our fee proposal (Ref. P58804UR2 Rev1) dated 14 July 2023.

JK Geotechnics (JKG) prepared a geotechnical report for the previous form of the proposed hospital alterations and additions (Ref. 35092URrpt) dated 17 August 2022 (JKG2022).

We have been provided with the following relevant information:

- Survey plan (Reference Number 22/0056 Rev. 4, dated 17 June 2022) prepared by Monteath & Powys.
- Architectural drawings (Drawing Numbers MHR-STH-AR-DR-SW-10-001 and 12-001, dated 8 June 2023) prepared by Silver Thomas Hanley (STH).
- An architectural drawing (Drawing Number A20-001, dated 3 May 2023 prepared by STH and "Civil Markup" annotations by Northrop dated 23 May 2023.
- Extracts of a Landscape Report (Ref. 23-049s, dated 30 May 2023) prepared by Taylor Brammer.
- A "Structural Schematic Design Report" (Ref. NL231211 Rev. 1) dated 1 June 2023) prepared by Northrop.
- Structural drawings (Ref. SK1 to SK3 Rev A, dated 1 June 2023) prepared by Northrop.
- A drawing titled "Additional Geotechnical Investigation – Comments" Rev. 3 prepared by Northrop and provided on 25 August 2023 summarising the structural aspects of the proposed footing system (including variations) with accompanying preliminary calculations dated 15 August 2023, prepared by Northrop.

Based on a review of the provided information (including the relevant information provided when preparing JKG2022) and discussions with Kevin van Aardt (Northrop), following partial demolition we understand the proposed alterations and additions will include:

- Construction of a new two storey Acute Services Building (ASB) over the south-eastern portion of the hospital grounds. The ASB will comprise a steel frame structure with either a floor slab suspended between pad or pile footings or a stiffened raft slab. The perimeter column working loads will range between 500kN and 750kN and the internal column working loads will range between 900kN and 1250kN. The central southern portion of the ASB will include an imaging department containing movement sensitive equipment and so the design of this area will require negligible floor movements to be achieved. The proposed finished floor reduced level of the ground floor level will be RL209.73m which is approximately 0.3m above existing surface level. In addition, there is a requirement for a subfloor space to allow access to install and maintain services; the intention is to hang service pipes off the underside of the floor slab. The subfloor space head height requirement is 0.9m which will require bulk excavations in the order of 0.6m to achieve design surface levels. The site lies within a flood zone and a perimeter flood wall comprising a precast flood wall abutting the base of the building and extending to the level of the Probable Maximum Flood (PMF). We understand from Northrop that a flood wall extending to the level of the ground floor level is considered sufficient for flood design purposes.

However, we have not been provided with this advice and have no further details such as any freeboard requirements. A fill embankment with a paved surface is proposed to cover the lower portion of the flood wall. Although most existing trees will remain, a small number will be removed within the building footprint.

- A new staff car park over the south-western portion of the hospital grounds and a new public car park immediately to the north of the new ASB. The design traffic loading for the car parks has been assumed to be 1×10^5 ESA's (Equivalent Standard Axles). The design surface levels of the new car parking areas will essentially be at current surface levels and so nominal re-profiling of site surface levels is expected. Some existing trees within the area of the proposed public car park will be removed.
- An ambulance entry will be provided at the eastern end of the southern side of the new ASB and a new loading bay will be provided at the western end of the new ASB.
- Landscaping of the area around the proposed alterations and additions, including a new footpath linking the northern street frontage to the northern side of the ASB and areas of new seating, external paved areas and new plantings although details of the types of plants or trees have not been provided.

The purpose of the investigation was to obtain additional geotechnical information on the subsurface conditions, and to use this as a basis for providing additional comments and recommendations on the geotechnical aspects of the revised proposed development.

The geotechnical investigation for JKG2022 was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate reports by JKE (Ref: E35093UPDrpt and E35093BTrpt-HAZ both dated August 2022, for the results of the environmental site assessments. We note that JKE returned to site in mid August 2023 to complete a detailed environmental site investigation (DSI) on behalf of HI.

2 INVESTIGATION PROCEDURE

This additional investigation was completed on 19 and 20 July 2023 and included:

- Three boreholes (BH101 to BH103) auger drilled to depths of 9.0m (BH102 and BH103) and 10.45m (BH101) using a 'Geoprobe' drill rig.
- Five boreholes (BH104 to BH108) auger drilled using a 'Geoprobe' drill rig with a 150mm diameter auger to a depth of 2.0m. The purpose of the 150mm diameter boreholes was to identify the presence and depth of tree roots.

The boreholes were drilled as close as practicable to the locations nominated by Northrop and with regard to the locations of buried services and access restrictions due to the height of the adjacent tree canopies.

The test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface RLs at the test locations were interpolated between spot levels and contours shown on the provided survey plan. The attached Figure 2 is based on aerial imagery sourced from 'Google Earth' with the outline of the proposed ASB and car parks superimposed. The survey datum is the Australian Height Datum (AHD).

The strength and relative density of the alluvial clays and sands respectively, were assessed from the Standard Penetration Test (SPT) 'N' values, augmented by the results of hand penetrometer readings on the cohesive soil samples recovered in the SPT split tube, undisturbed (U50) tube sample and cuttings recovered from the augers.

Groundwater observations were made in the boreholes during and on completion of auger drilling. No longer term ground monitoring has been carried out although we note that monitoring wells have been installed in selected environmental boreholes as part of the JKE DSI.

The fieldwork for the investigation was carried out under the direction of our geotechnical engineer (Cody Surawski), who was present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, and nominated in-situ testing and sampling. The borehole logs (which include field test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected soil samples were returned to the Douglas Partners Pty Ltd (DP) NATA registered laboratory for moisture content, Atterberg Limits, linear shrinkage, Shrink-Swell Index, Standard compaction and four day soaked CBR testing. The results are summarised on the attached DP 'Material Test Report' (Report No. 224178.00-1, dated 7 August 2023).

3 RESULTS OF INVESTIGATION

The results of our previous desk top study are presented in Section 3.1 of JKG2022. Since our previous investigation in 2022 there were no discernible changes to the site conditions and the site description provided in Section 3.2 of JKG2022 remains valid.

3.1 Subsurface Conditions

The boreholes and test pits from our current and previous investigations have disclosed a generalised profile comprising topsoil (or locally a variable thickness of fill) overlying alluvial clays with alluvial sands intermittently encountered at moderate or greater depth. Bedrock was not encountered within the depth of the current or previous investigations. Groundwater seepage was encountered in places at moderate depth in the alluvial sands in the previous investigation, however no groundwater seepage was encountered within the depth of the current investigation. Reference should be made to the attached borehole logs from our current investigation and the borehole logs and test pit logs presented in JKG2022 for specific details at each location. A summary of the pertinent subsurface characteristics from both investigations is presented below and a graphical summary of the subsurface profile along an east-west section line is presented as Figure 3.

Pavements
A variable thickness bitumen or reinforced concrete pavement (or a granular pavement) provided with a variable thickness of granular road base and overlying an alluvial clay subgrade were encountered in our previous BH-P1 to BH-P7; refer to Section 3.3 of JKG2022 for further details.

Topsoil
Low plasticity silty clay topsoil (0.2m thick) was encountered in BH2 and BH6 and silty clay topsoil of 0.1m to 0.4m thickness and assessed to be of medium plasticity was encountered in BH101 to BH108. We note that in BH102 glass and asbestos inclusions were noted in the topsoil. Based on the presence of topsoil in the remaining boreholes from our current investigation we have interpreted the presence of glass and asbestos in BH102 to indicate disturbed in-situ topsoil rather than imported topsoil fill, most likely associated with past construction works.

Fill
Except for the pavement materials described above, surficial clay fill was encountered in BH1, BH3 to BH5 and BH7. In BH1 sandy clay fill assessed to be of low plasticity extended to 0.2m depth. In BH3 to BH5 and BH7 silty clay fill assessed to be of medium plasticity extended to depths between 0.1m and 0.5m.

TP2 to TP4 encountered surficial clay fill and a 0.2m thick band of sandy gravel fill encountered at 0.4m depth in TP2. The fill extended to depths between 0.1m and 0.6m.

Due to the limited thickness of fill encountered representative SPT tests were unable to be carried out and an assessment of the relative compaction could not be made. However, we note that in the absence of formal records of placement and density test results, the fill is regarded as 'uncontrolled' as defined in AS2870-2011 'Residential slabs and footings'.

No fill was encountered in the current boreholes. However, over the western portion of the proposed ASB what appeared to be a fill platform (0.3m height) formed a helipad and a single storey brick building appeared to have been constructed on a fill platform (maximum 0.5m height).

Alluvial Soils
On first contact beneath the topsoil, paved surface, pavement construction materials or surficial fill, alluvial silty clays typically of high plasticity and very stiff to hard strength were encountered in all the boreholes from our current and previous investigations. Bands of sandy clay were encountered in BH6 (1.5m thick), BH101 (2.4m thick), BH102 (0.4m thick) and sandy silty clay in BH5 (1.5m thick) at respective depths of 4.0m, 2.2m, 3.5m and 4.0m depth. The alluvial sandy (or sandy silty) clays were assessed to be of low or medium plasticity and very stiff or hard strength. In BH1 alluvial sandy silt (1.5m thick) assessed to be of low plasticity and hard strength was encountered at 4.0m depth. The test pits from our previous investigation all encountered similar alluvial silty clays or sandy silty clays either from surface level or below the fill. All the test pits were terminated in the alluvial clays at 1.0m depth.

The alluvial clays extended to the termination of BH-P1 to P7 (1.5m depth), BH2 (5.95m depth), BH101 (10.45m depth) and BH104 to BH108 (2.0m depth).

The alluvial clays (or silts in BH1) were underlain by medium dense alluvial sands at depths of 6.0m (BH3) and 5.5m (BH1, BH4 to BH7). These boreholes were terminated in the alluvial sands at depths between 5.95m and 6.45m. In the current investigation the following bands of alluvial sands were encountered:

- In BH102 a 3.45m thick band of gravelly sand assessed to be of medium dense relative density at 3.9m depth and underlain by alluvial silty clay of very stiff strength which extended to the termination depth of 10.45m; and
- In BH103 a 3.3m thick band of sand assessed to be of loose relative density at 5.0m depth and underlain by alluvial silty clay of hard strength which extended to the termination depth of 9.0m.

In the recent JKE DSI alluvial silty sands were also encountered:

- In BH209 (immediately to the north-west of the proposed ASB) at 4.8m depth, the sands were 1.0m thick and based on limited SPT 'N' values appeared to be of loose to medium dense relative density; and
- BH224 (over the south-eastern corner of the proposed ASB) at 3.2m depth, the sands were 3.3m thick and contained a 0.4m thick band of silty clay at 4.3m depth. Based on limited SPT 'N' values the sands appeared to be of loose to medium dense relative density.

Based on the above, the sand bands encountered in the boreholes are not laterally persistent and of uniform thickness in the alluvial soils at the site. Furthermore, where the sand was of sufficient thickness for SPT tests indicated that the relative density of the sands was not consistent and was loose in some areas.

Groundwater

All boreholes and test pits from our current and previous investigation were 'dry' during and on completion of auger drilling and excavation with the exception of BH6 where seepage was recorded in the alluvial sands at 5.6m depth. We note that groundwater levels may not have stabilised over the relatively short observation period. No long-term groundwater level monitoring has been carried out and to date additional site visits by JKE have not been engaged to return to site to record any water levels in the monitoring wells.

3.2 Laboratory Test Results

Based on the Liquid Limit and Linear Shrinkage determinations:

- The samples of alluvial silty clay tested from BH2, BH4 and BH6 were of high plasticity (BH2 and BH6) and medium to high plasticity (BH4) with an assessed extreme potential for shrink/swell reactivity with changes in moisture content.
- The sample of alluvial sandy clay tested from BH101 was of low plasticity with an assessed moderate potential for shrink/swell reactivity.
- The samples of alluvial silty clay tested from BH102 and BH103 were of medium to high plasticity and an assessed high potential for shrink/swell reactivity.

The shrink swell index tests carried out on the U50 samples of alluvial silty clay recovered from BH101, BH102 and BH103 returned respective Iss values of 4.4%, 3.2% and 1.6% indicating a high potential (BH101 and BH102) and moderate potential (BH103) for shrink/swell reactivity.

The four day soaked CBR values of the alluvial silty clay samples from BH3 and BH-P1 to P7 returned values ranging between 2.5% and 6% when compacted to 100% of Standard Maximum Dry Density (SMDD) and surcharged with 4.5kg. The alluvial silty clay sample from BH103 returned a four day soaked CBR value of 6% when compacted to 98% SMDD and surcharged with 4.5kg. The natural moisture content of the samples tested in BH-P1 to P4 and BH103 were between 0.4% and 3.9% 'dry' of the Standard Optimum Moisture Contents (SOMC) and in BH-P5 to P7 and BH3 were between 0.3% and 4.1% 'wet' of the Standard Optimum Moisture Contents (SOMC). The samples swelled during soaking between 1.3% and 3% (BH-P1 to P4, and BH103) and between 0.1% and 1.1% (BH-P5 to P7 and BH3).

The results of the soil aggression testing are tabulated below:

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity (ohm.cm)
BH1	2.5-2.95	Silty Clay ALLUVIAL	6.7	12.1	49.2	14.3
BH7	4.0-4.45	Sandy Clay ALLUVIAL	6.8	18.5	409	3.7

4 COMMENTS AND RECOMMENDATIONS

We provide below additional comments and recommendations on geotechnical aspects of the revised proposed developments. Apart from the following revised/updated geotechnical advice, the comments and recommendations provided in JKG2022 should be adopted. This includes:

- demolition and excavation;
- measures to reduce vibrations;
- temporary and permanent cut and fill batter slopes;
- retention design and suitable retention systems (together with advice on geotechnical related construction aspects);
- site classification to AS2870-2011;
- existing building damage;
- earthquake classification to AS1170.4 – 2007;
- earthworks;
- pavement design parameters, and flexible pavement thickness design; and
- slope stability issues and suitable methods to improve stability.

4.1 Geotechnical Considerations and Constraints

The alluvial clay encountered at the site is colloquially referred to as black cotton soil, and geologically as black vertosol (soils containing a high content of expansive clay minerals [primarily montmorillonite]). These alluvial clays are extremely reactive soils with changes in moisture content and are susceptible to softening when wet, can also become 'sticky' when wet, and will form wide open cracks when dry.

The clay soils at the site are typically of high plasticity, have moisture contents beyond their plastic limits and have variable moisture contents when compared to their optimum moisture contents. As such, we would expect the clay subgrade to heave under proof rolling, therefore making compaction of fill and pavement materials above difficult.

Therefore, the construction of the access roads, car park areas and other earthworks associated with the building construction should be undertaken using a contractor who is experienced in working with these materials, and the use of bridging layers over the heaving subgrade may be required in some areas.

The reactive nature of the alluvial clays will also have implications for footing and floor slab design; our assessment of the alluvial clays indicates that their design would need to be based on a Class E-D site classification, in accordance with AS2870-2011.

Further advice in relation to the above geotechnical considerations are provided in the relevant sections of JKG2022 and, where applicable, additional advice is presented in the remaining sections of this current report.

4.2 Site Preparation

Based on the results of the current investigation, we confirm that the previous advice in relation to dilapidation surveys, demolition and excavation, seepage and temporary batters presented in Sections 4.2.1 to 4.2.4 of JKG2022 remains valid. However, in relation to alternative footing design options proposed by Northrop and with regard to the proposed removal of trees, additional advice on soil reactivity considerations is presented in Section 4.5 below.

4.3 Earthworks

Based on the results of the current investigation, we confirm that the previous advice in relation to earthworks (site drainage, subgrade preparation, engineered fill and trench backfill) presented in Sections 4.3.1 to 4.3.4 of JKG2022 remains valid.

4.4 Retention and Permanent Batter Slopes

Based on the results of the current investigation, we confirm that the previous advice in relation to retention and permanent batter slopes (retention design parameters, retaining walls supporting engineered fill and permanent batter slopes) presented in Sections 4.4.1 to 4.4.3 of JKG2022 remains valid. However, additional advice is provided in Section 4.5.2 below in relation to supporting the subfloor space excavations with retaining walls.

4.5 Footing Design

4.5.1 Site Classification

Based on the results of our investigations, due to the presence of trees (some of which will be removed as part of the works and some which may be planted as part of proposed landscape works) and the removal of paved areas (all which represent abnormal moisture conditions), and the presence of uncontrolled fill, the site is classified as Class 'P', in accordance with AS 2870 – 2011, "Residential Slabs and Footings". However, we note that AS2870 does not strictly apply for the proposed type of structure but should be referenced for general guidance on footing and floor slab design and site maintenance.

4.5.2 Soil Reactivity Considerations and Impacts on Design

We note that the following footing design solutions have been considered together with various ground improvement methods to attempt to manage predicted characteristic surface movements in the upper reactive clays:

- A floor slab suspended between pad footings, including replacement of reactive clay with inert granular fill or lime stabilised clayey soil beneath the footings;
- A floor slab suspended between piles; or
- A stiffened raft slab, including replacement of reactive clay with inert granular fill or lime stabilisation beneath the raft slab.

However, the footing design solutions must also provide a ground floor level at RL209.73m with the underside of the floor slab approximately 0.3m above existing surface level. In addition, a subfloor space of 0.9m height which will require bulk excavations in the order of 0.6m below the existing surface level to achieve design surface levels.

Based on the laboratory test results from our current and previous investigations there is some variability in the reactivity of the alluvial silty clays and the alluvial sandy clays. However, based on the results of our investigations and the design soil suction change depth for this climatic region (3.0m), the alluvial silty clays are extremely reactive with changes in moisture content (i.e. similar to those expected for a Class 'E-D' site).

Where large mature trees are to be removed from the footprint of the proposed ASB and car park (which includes trees of maximum 18m height, based on the survey plan) then to prevent long term shrink swell movements impacting the proposed footing system and car park pavements, the entire primary root structure must be removed in accordance with the following advice:

- For the design soil suction change depth for this climatic region this requires an excavation depth potentially of 3.4m (i.e. equivalent to H_t as defined in Figure H1 of AS2870-2011). However, an arborist must be engaged to advise on the radius and depth of the likely primary root structure of each tree being removed.
- The excavation must be backfilled with inert (non reactive) granular fill.

- The temporary excavations for the removal of tree root systems must be battered or benched back at no steeper than 1V in 1H for the clayey soils and in accordance with the advice presented in Section 4.2.4 of JKG2022.

As noted above, in response to the soil reactivity alternative ground treatment options have been considered:

- Lime stabilisation of the upper alluvial clay profile;
- Replacing the upper 0.6m of reactive clay soils with inert granular fill; or
- Replacing the upper 1.0m of reactive clay soils with inert granular fill.

We provide below advice on soil reactivity considerations relevant to each of these solutions.

Lime Stabilisation

We have assessed the option of lime stabilisation of the upper alluvial clay profile in an attempt to improve the site classification to AS2870-2011. In order to do so, the upper alluvial clay profile would need to be excavated to an appropriate depth, stockpiled, dried out to below its optimum moisture content, then thoroughly blended through a pugmill with at least 4% hydrated lime (by dry weight of clay). The excavation would need to extend laterally below the entire footprint of the proposed building, and extend at least 3.0m horizontally outside the footprint. The lime stabilised clay would then need to be placed and compacted in layers to an appropriate earthworks specification, which would likely require 'wetting up' the lime stabilised clay to close to its optimum moisture content.

The primary geotechnical issues with this approach are summarised below:

1. From our experience, the reduction in shrink-swell reactivity by lime stabilisation is typically in the range of 25% to 50%, with recent test results closer to the lower end;
2. Excavation of the upper alluvial clay profile will remove the cracked (unrestrained) zone, which we have assumed to be 1.5m deep. The crack zone essentially comprises vertical cracks which are caused by shrink-swell movements. When the cracks are open (ie. due to shrinkage), they permit horizontal swelling of the soil, thus reducing the amount of vertical heave that would otherwise occur. The nominated foundation stabilisation option described above would remove a portion (if not all) of the cracked zone; that is, all shrinkage cracks would effectively be closed. As a result, lateral swelling of the clay soil will be constrained and the volumetric change will manifest as a greater vertical movement than had the cracked zone been present. This concept is described in Section C2 of AS2870-2011.

To illustrate the above issues, we have calculated the characteristic surface movement (y_s) for several scenarios (without tree effects being considered), with the above described inert fill replacement option included for comparison purposes, as tabulated below:

Scenario	Subsurface Profile	Calculated y_s Value (mm)	Site Classification to AS2870-2011
1	Existing alluvial clay	85	E-D
2	Excavation to 0.6m depth and replacement with inert crushed rock	55	H1-D
3A	Excavation to 1.5m depth and replacement with lime stabilised (site won) clay (**25%** reduction in shrink-swell reactivity) – *complete removal of the cracked zone*	105	E-D
3B	Excavation to 1.5m depth and replacement with lime stabilised (site won) clay (**50%** reduction in shrink-swell reactivity) – *complete removal of the cracked zone*	80	E-D
4A	Excavation to 1.0m depth and replacement with lime stabilised (site won) clay (**25%** reduction in shrink-swell reactivity)	100	E-D
4B	Excavation to 1.0m depth and replacement with lime stabilised (site won) clay (**50%** reduction in shrink-swell reactivity)	80	E-D
5A	Excavation to 0.5m depth and replacement with lime stabilised (site won) clay (**25%** reduction in shrink-swell reactivity)	95	E-D
5B	Excavation to 0.5m depth and replacement with lime stabilised (site won) clay (**50%** reduction in shrink-swell reactivity)	80	E-D

As can be seen in the table above, lime stabilising the upper alluvial clay profile does not improve the characteristic surface movements and therefore the site classification to AS2870-2011. We therefore do not consider this method to be applicable for the site and has not been considered further.

Replacement With Inert Granular Fill

Based on a minimum horizontal distance of the trunk of an existing tree that will remain from the proposed ASB building (D_t) of 7.5m and a maximum tree height of 12m for the trees that will remain (based on the survey plan), then for the two proposed inert granular fill replacement depths the centre heave and edge heave for a stiffened raft slab (based on the Walsh method) adopting a design characteristic surface movement (y_s) of 85mm with no inert fill replacement and reduced y_s values of 55mm and 45mm for 0.6m or 1.0m depths of inert fill replacement are provided below:

Scenario	Centre Heave	Edge Heave
No inert fill replacement	104mm	42.5mm
0.6m inert fill replacement	82mm	27.5mm
1.0m inert fill replacement	74mm	22.5mm

Considerable differential movements would therefore be expected beneath the stiffened raft slab which would clearly exceed the negligible movements required for the imaging department area of the ASB.

Furthermore, with the removal of trees there will be localised but large deep areas of inert granular fill placed. In addition, reactive clay replacement with inert granular fill will result in further variation to the engineered fill thickness across the ASB building which could exacerbate differential movements.

Additional Comments

We note that to achieve the 0.9m subfloor space, excavation to at least 0.6m below existing surface levels would be required and where trees are to be removed and replacement of reactive clay with inert granular fill is proposed, additional excavations would be required. This would be a considerable earthworks operation and require the excavated clay to be removed from site and importation of inert granular fill which would require compaction; all of which would incur considerable cost.

The excavation, when filled within inert granular fill to the design surface levels would in effect create a large pond under the proposed ASB which could collect water and exacerbate reactive clay movements. The areas of inert granular fill would also collect water in the excavations to replace tree root systems and/or if reactive clays are replaced with inert granular fill. To prevent water ingress, it has been proposed to use clay fill (sourced from the site excavations) to backfill above existing surface level around the building up to the design surface levels of the proposed paved surface surrounds (including the access ramp and loading bay). The reactive clay fill would be subject to excessive shrink swell movements which would impact the paved surfaces and potentially cause cracking and damage. In addition, where the clay shrinks and cracks during dry periods this could potentially allow water ingress into the subfloor area and exacerbate shrink swell movements. We do not recommend this option.

We recommend that the excavation for the subfloor area be entirely supported by a tanked retaining wall (i.e. no behind wall drainage) with the retaining wall extended above surface level to form the flood wall. This would reduce the plan area of the excavation currently proposed. The retaining wall would need to be designed in accordance with the advice provided in Section 4 of JKG2022. We also consider that for the limited excavation height (at least 0.6m but less than 1m), a sub-vertical temporary cut batter through the clays is likely to be feasible and would there limit the need for any behind wall backfilling which could result in allowing water ingress if not detailed properly. Any gap between the retaining wall and cut face should be grouted up. The following would also be required:

- The subfloor surface would need to be profiled such that any water collecting in the void (from condensation, leaking pipes etc) is directed to a sump and pump system for controlled discharge into the stormwater system. A back up pump would also be required in the event of pump failure and regular inspections and maintenance of the subfloor area and the utilities within by hospital staff would be required and any issues promptly rectified.
- The subfloor surface should be provided with a cover of granular engineered fill (say demolition rubble) in order to provide a 'clean' and dry surface for personnel to work on. To prevent any potential migration of clay fines, a non-woven geofabric (such as Bidim A34) should be provided as a separation layer between the clay surface and the granular fill.
- The surface levels surrounding the building would need to be profiled to direct surface water run-off away from the building to the stormwater system.

With regard to the current proposed flood wall detail which we assume applies to the areas beyond the proposed access ramp and loading bay, we note that if site won reactive clay is used as the fill in front of the flood wall, then there are likely to be excessive and unpredictable reactive movements and the paved surface proposed on top of the fill would need to be separated from the main building. Shrinkage of a reactive material could result in separation between the fill and the flood wall with water then entering the reactive subgrade and exacerbating reactive soil movements. Our strong preference is that the fill in front of the flood wall also comprise inert material. A crushed rock with at least 12% to 15% fines (e.g. DGB20, a 40mm run of crusher, etc.) would be a suitable fill.

With regard to proposed landscaping, we do not recommend tree planting close to building footings as this would potentially worsen the predicted site classification by at least one level. Typically for such reactive sites if trees are to be planted, we would recommend a root barrier extending to at least 3.0m depth be installed to protect the buildings. However, as there are existing trees, the installation of a root barrier for new trees planted in an area where existing trees are present would have the same impact on reactive soil movements as removing the existing trees. Our strong preference is that new trees are not planted.

We also recommend that the following also be adhered to in order to reduce potential reactive soil movements with changes in moisture content:

- Buildings entirely surrounded with pavements at least 1.5m wide and slightly sloping away from the buildings to prevent the ponding of water and all joints between the building and external pavements be infilled using a flexible "Mastic" sealer.
- Avoid establishing garden beds adjacent to proposed buildings. Moisture ingress into the subgrade at these locations could cause movement and damage to nearby structural elements. Any planter beds close to buildings should be completely encased in concrete with base drainage connected to the stormwater system for controlled disposal.

4.5.3 Footing Systems and Design Parameters

Based on the assessment of reactive movements in Section 4.5.2 above and with regard to treatment of areas where trees are removed and replacement of reactive clays with inert fill (resulting in varying inert fill thicknesses), there will still be the potential for considerable differential movements impacting either pad footings used to support the suspended floor slab or beneath the stiffened raft slab. These differential reactive clay surface movements would exceed the negligible movements required for the imaging department area of the ASB.

Based on the above we do not consider that either a stiffened raft slab or a ground floor slab suspended on pad footings are suitable footing options for the ASB.

In our opinion, the most suitable footing system is a floor slab suspended between pile footings which would limit settlements and prevent reactive surface movements impacting the ASB. The piles would need to be de-bonded/permanently sleeved to 3.4m depth (H_t as defined in Section 4.5.1 above).

We note that our advice on the preferred footing system has been based on an analysis of the current proposed alternatives and with regard to the design constraints posed by the need for a subfloor space height of 0.9m, the fixed ground floor level height (due to maximum ramp angles down to the street frontage and the inclusion of a movement sensitive imaging department in the ASB. These design constraints were not known at the time of preparing JKG2022.

We do not consider a hybrid solution incorporating a mix of piled footings and high level footings (including supporting the building on high level footings and piling the slab for the imaging equipment) is appropriate as the differential movements between areas of the building supported on piles and other areas supported on high level footings will be equivalent to the predicted characteristic surface movements.

Based on advice provided by Northrop, the proposed ASB perimeter column working loads will range between 500kN and 750kN and the internal column working loads will range between 900kN and 1250kN.

For pile footings we provide below the following serviceability (allowable) and ultimate limit state design parameters and elastic parameters to assist with the design of pile footings.

Material	Ultimate Limit State Bearing Pressure	Serviceability (allowable) Limit State Bearing Pressure	Ultimate Limit State Adhesion (Bored Piles)	Serviceability (allowable) Limit State Adhesion (Bored Piles)	Elastic Modulus Short Term (Long Term)	Poisson's Ratio
Very stiff alluvial clays [1, 3 & 4]	900kPa	300kPa	54kPa	18kPa	30MPa (21MPa)	0.3
Hard alluvial clays or silts [1, 3 & 4]	1,500kPa	500kPa	60kPa	20kPa	40MPa (28MPa)	0.3
Medium dense alluvial sands [2, 3 & 4]	5,000kPa	1,700kPa	45kPa	15kPa	45MPa (35MPa)	0.3
Loose alluvial sands [2, 3 & 4]	1,800kPa	600kPa	21kPa	7kPa	25MPa (20MPa)	0.3

Notes

1. Assuming a pile depth of at least 3.4m and an embedment of at least 4 pile diameters into the applicable clay or silt foundation material.
2. Assuming piles founded below 4.0m to 5.0m depth and embedded 4 pile diameters into the appropriate relative density alluvial sands underlying the alluvial clays or silts with at least 3 pile diameters of that strata below the toes of the piles. However, pile embedment would need to be limited such that the bearing pressure on the underlying alluvial silty clays does not exceed the bearing pressure for clays of very stiff or hard strength. Piles also to be de-bonded/permanently sleeved to 3.4m depth through the swelling clays.
3. To overcome uplift of the piles from the swelling clays then bored piles or CFA piles (if selected) should be a minimum length of 6.8m (i.e. twice the design suction depth, taking account of the presence of trees) and de-bonded/permanently sleeved to 3.4m depth.
4. Shaft Adhesion values in tension and compression are only applicable to bored piles.

Unfortunately, the additional JKG and JKE boreholes did not indicate a laterally consistent or considerable thickness of medium dense sands within the alluvial clays:

- In BH102 a 3.45m thick band of medium dense gravelly sand at 3.9m depth underlain by alluvial silty clay of very stiff strength which extended to the termination depth of 10.45m;
- In BH103 a 3.3m thick band of loose sand at 5.0m depth underlain by alluvial silty clay of hard strength which extended to the termination depth of 9.0m; and

- BH224 a 3.3m thick band of loose to medium dense sands at 3.2m depth, which contained a 0.4m thick band of silty clay at 4.3m depth.

In addition, the alluvial clay strength below the sands was either very stiff or hard. As outlined in Note 2 of the above table pile embedment into the sands would need to be limited such that the bearing pressure on the underlying alluvial silty clays does not exceed the bearing pressure for clays of very stiff or hard strength.

Therefore, to adopt founding in the sandy soils would require an intensive frequency of investigation to prove the presence of the sands possibly at each pile locations, such as using CPT testing.

Due to the variability of the alluvial soil profile the piles would need to be conservatively designed assuming very stiff strength clays unless hard strength clays (or more extensive medium dense sands bands) can be proven at greater depth than at least 10.45m. in addition, due to the variability of the alluvial soil profile the load carrying capacity of the bored pile shafts would be limited due to the presence of sands within the alluvial clays.

Adopting a 0.9m diameter pile, a minimum pile length of 7.0m would be required to satisfy the embedment depth criteria for swelling clays outlined in the above table. Based on the geotechnical investigation results the foundation material would range between hard strength clay overlying very stiff strength clay (BH101), medium dense sands overlying very stiff strength clay (BH102) and loose sands overlying hard strength clay. The strength of the clays is not recorded in the environmental JKE logs (BH209 and BH224).

We note that the bearing pressures (excluding the medium dense sands) are relatively limited, requiring several piles and a pile cap (with void formers of at least 75mm thickness below) to carry the proposed column loads. For instance:

- Excluding shaft adhesion, four 0.9m diameter piles founded in hard strength clays or at least six 0.9m diameter piles founded in very stiff strength clays seven would be required to support the maximum column load of 1250kN.
- Excluding shaft adhesion, two 1.2m diameter piles founded in hard strength clays or three 1.2m diameter piles founded in very stiff strength clays seven would be required to support the maximum column load of 1250kN.

For the above allowable bearing pressures, predicted settlements would be a maximum of 5mm for the piled footings embedded in the alluvial clays, silts or sands, plus any elastic shortening of the pile shaft.

The above allowable bearing pressures must be confirmed by inspection of a representative number of footings by a geotechnical engineer.

Bored piles are expected to be suitable and should be drilled using conventional piling rigs (i.e. not drilled using an auger attachment to an excavator) to ensure that the founding depth is achieved. Some allowance for sacrificial liners should be made in the event that groundwater seepage in the sands is encountered which could cause pile side walls to collapse. In addition, if seepage is encountered there would be difficulties maintaining clean and dry bases as the clays would be susceptible to softening if water is present and would require over-drilling to remove such materials. It would therefore be imperative to drill, clean out, inspect and pour bored pile footings with minimal delay.

Alternatively, due to the potential for seepage, the collapsible nature of sands where seepage occurs, together with cleanliness of pile bases in clay, softening of clays and possible concrete segregation, auger grout injected (CFA) piles could be used but may be expensive.

Steel screw piles are also suitable but no shaft adhesion can be assumed. The installation of steel screw piles would act to 'loosen' the reactive clays from around the shaft and be beneficial in reducing swell pressures acting on the piles. However, the steel screw piles may have difficulties penetrating the hard strength alluvial clays and pre-boring may be required. If pre-boring is required then it is critical that the pre-boring does not create an over-sized hole that allows surface water ingress into the reactive clays and cause excessive swell pressures.

The piling contractor must be provided with a copy of this report so that they can provide appropriate equipment in order to install the piles. If either steel crew piles or CFA piles are preferred then their design must be based upon the bearing capacities recommended above and not on empirical correlations such as from installation torque. The piling contractor must also certify the geotechnical and structural capacity of their installed piles.

We recommend that the pile design comprise a design and construct (D&C) package within the Contract with only suitably experienced and insured piling contractors invited to tender. Due to the variability of the foundation materials and the thickness and lateral extent of the sand layers in the alluvial profile the D&C package could include a requirement for further geotechnical investigation to assist in optimising the piling contractors design, particularly in the areas of the highest column loads requiring several piles and a pile cap. The additional investigation would be best completed using a CPT rig as about 6 locations could be tested in a single day to say 10m to 15m depth.

4.6 Existing Building Damage

Based on the results of the current investigation, we confirm that our comments on existing building damage presented in Section 4.6 of JKG2022 are unchanged.

4.7 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) = 0.09;
- Class D_e – "Deep soil site".

4.8 Floor Slabs

Based on the results of the current investigation, we confirm that the previous advice in relation to floor slabs and external paved areas presented in Section 4.8 of JKG2022 remains valid. However, in relation to recommended suspended floor slab design discussed in Section 4.5 above, we recommend that the loading bay and access ramps also be suspended. This will prevent differential movements between the on-grade external pavement areas and the suspended building which would be equivalent to the reactive surface movements. This could cause damage and cracking which could allow water to enter the subfloor space.

4.9 Pavement Design and Construction

Based on the results of the current investigation, we confirm that the design CBR of 2% recommended in Section 4.9 of JKG2022 remains valid. Furthermore, we also confirm that the previous advice in relation to pavement design considerations, pavement thickness design, pavement materials and existing pavements presented in Sections 4.9.1 to 4.9.4 of JKG2022 remains valid.

Where trees are removed from the proposed car park area and the root system removed and replaced with inert fill as outlined in Section 4.5.2 above then this will present variable localised subgrade conditions and differential movements between the reactive clays and inert fill can be expected. This could result in cracking of the pavement surface and would require crack sealing to prevent excessive amounts of water entering the reactive subgrade. It is essential that the cracks are appropriately sealed as soon as possible in order to prevent significant reduction in the pavement life.

4.10 Soil Aggression

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability and in AS3600-2018 “Concrete Structures” we note that the laboratory chemical test results presented in JKG2022 have indicated that the following Exposure Classifications are applicable:

- ‘Non-aggressive’ for concrete piles (based on Table 6.4.2 (C), in AS2159-2009),
- ‘Moderate’ for steel screw piles (based on Table 6.5.2 (C), in AS2159-2009), and
- A1 for concrete footings or slab thickenings (based on Table 4.8.1 in AS3600-2018).

4.11 Site Stability

Based on the results of the current investigation, we confirm that the advice on site stability presented in Section 4.11 of JKG2022 remains valid.

4.12 Preliminary Groundwater Assessment

The current and previous investigations have extended boreholes to a maximum depth of 10.45m and only one borehole encountered seepage in a band of sand within the alluvial clays at 5.6m depth. The expected maximum excavation depth is expected to be in the order of 1.0m to accommodate the subfloor area below the suspended ground floor slab.

Based on the above we do not expect to encounter groundwater to be encountered in the excavations and so not specific requirements for measures such as dewatering are expected to be required. Localised seepage may be encountered in some pile drill holes which can be satisfactorily managed by techniques appropriate to the piling system adopted; temporary liners in bored piles to prevent collapse, use of CFA piling techniques to support the drill hole or installation of screw piles.

4.13 Further Geotechnical Input

Reference should be made to Section 4.12 of JKG for the summary of further geotechnical input relevant to the contents of that report. 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:

- Further geotechnical investigation by the piling contractor as part of a D&C package, if required.
- Inspection of a representative number of bored pile footings or witnessing the installation of a select number of screw piles or CFA piles, if selected.

5 GENERAL COMMENTS

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

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

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

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

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

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

Material Test Report

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Sunshine Coast Laboratory
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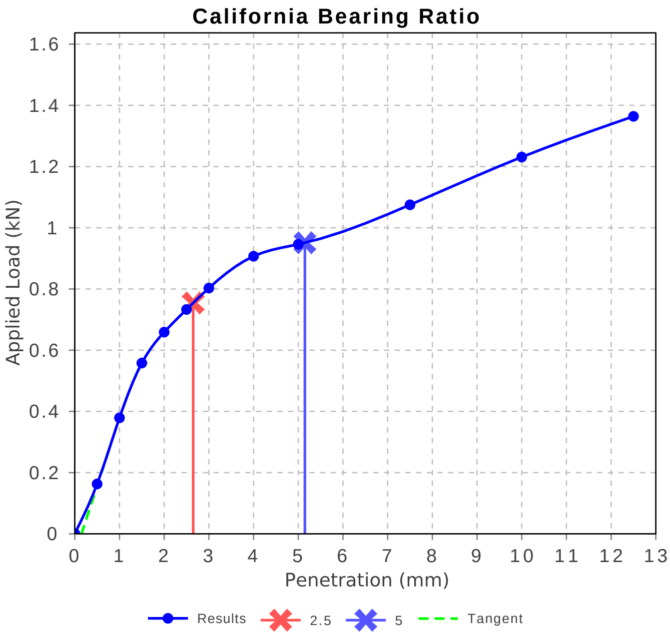
Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Shae Harry
Laboratory Manager
Laboratory Accreditation Number: 828

Report Number:	**224178.00-1**
Issue Number:	1
Date Issued:	07/08/2023
Client:	JK Geotechnics
	8 Innovation Parkway, Birtinya QLD 4575
Contact:	Paul Roberts
Project Number:	224178.00
Project Name:	Proposed Development
Project Location:	35 Alice Street, Moree NSW
Client Reference:	35092UR2
Work Request:	25022
Sample Number:	SS-25022A
Date Sampled:	19/07/2023
Dates Tested:	25/07/2023 - 31/07/2023
Sampling Method:	Sampled by Client
	The results apply to the sample as received
Sample Location:	**BH103, Depth: 0.4 - 1.3**

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	**6**		
Method of Compactive Effort	Standard		
Method used to Determine MDD	AS 1289 5.1.1 & 2.1.1		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)	1.71		
Optimum Moisture Content (%)	19.0		
Laboratory Density Ratio (%)	98.0		
Laboratory Moisture Ratio (%)	100.5		
Dry Density after Soaking (t/m^3)	1.64		
Field Moisture Content (%)	18.6		
Moisture Content at Placement (%)	19.1		
Moisture Content Top 30mm (%)	27.5		
Moisture Content Rest of Sample (%)	20.2		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	24.0		
Swell (%)	2.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0.0		



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Project Number:	224178.00
Project Name:	Proposed Development
Project Location:	35 Alice Street, Moree NSW
Client Reference:	35092UR2
Work Request:	25022
Sample Number:	SS-25022E
Date Sampled:	19/07/2023
Dates Tested:	25/07/2023 - 04/08/2023
Sampling Method:	Sampled by Client *The results apply to the sample as received*
Sample Location:	**BH101 , Depth: 2.5 - 2.95**

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Air Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	30		
Plastic Limit (%)	17		
Plasticity Index (%)	**13**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**8.5**		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	10.8		

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Project Name:	Proposed Development
Project Location:	35 Alice Street, Moree NSW
Client Reference:	35092UR2
Work Request:	25022
Sample Number:	SS-25022F
Date Sampled:	19/07/2023
Dates Tested:	25/07/2023 - 04/08/2023
Sampling Method:	Sampled by Client *The results apply to the sample as received*
Sample Location:	**BH102 , Depth: 1.0 - 1.45**

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Air Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	56		
Plastic Limit (%)	21		
Plasticity Index (%)	**35**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**18.0**		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	17.2		

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	8 Innovation Parkway, Birtinya QLD 4575
Contact:	Paul Roberts
Project Number:	224178.00
Project Name:	Proposed Development
Project Location:	35 Alice Street, Moree NSW
Client Reference:	35092UR2
Work Request:	25022
Sample Number:	SS-25022G
Date Sampled:	19/07/2023
Dates Tested:	25/07/2023 - 04/08/2023
Sampling Method:	Sampled by Client
	The results apply to the sample as received
Sample Location:	**BH103 , Depth: 2.3 - 2.5**

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Air Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	46		
Plastic Limit (%)	17		
Plasticity Index (%)	**29**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	**16.0**		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)		Min	Max
Moisture Content (%)	17.0		

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Contact: Paul Roberts
Project Number: 224178.00
Project Name: Proposed Development
Project Location: 35 Alice Street, Moree NSW
Client Reference: 35092UR2
Work Request: 25022
Date Sampled: 19/07/2023
Dates Tested: 25/07/2023 - 31/07/2023
Sampling Method: Sampled by Client
The results apply to the sample as received
Location: Material Testing

Shrink Swell Index AS 1289 7.1.1 & 2.1.1					
Sample Number	SS-25022B	SS-25022C	SS-25022D		
Date Sampled	19/07/2023	19/07/2023	19/07/2023		
Date Tested	31/07/2023	31/07/2023	31/07/2023		
Material Source	Insitu	Insitu	Insitu		
Sample Location	BH101 (1.0 - 1.3)	BH102 (0.4 - 0.76)	BH103 (2.5 - 2.87)		
Inert Material Estimate (%)	1	2	1		
Pocket Penetrometer before (kPa)	>600	>600	>600		
Pocket Penetrometer after (kPa)	550	430	450		
Shrinkage Moisture Content (%)	19.7	17.0	19.3		
Shrinkage (%)	**5.8**	**3.2**	**2.1**		
Swell Moisture Content Before (%)	20.2	17.9	17.8		
Swell Moisture Content After (%)	23.4	23.5	21.7		
Swell (%)	**4.4**	**5.2**	**1.7**		
Shrink Swell Index Iss (%)	**4.4**	**3.2**	**1.6**		
Visual Description	**	**	**		
Cracking	UC	SC	UC		
Crumbling	No	No	No		
Remarks	**	**	**		

Shrink Swell Index (Iss) reported as the percentage vertical strain per pF change in suction.
Cracking Terminology: UC Uncracked, SC Slightly Cracked, MC Moderately Cracked, HC Highly Cracked, FR Fragmented.
NATA Accreditation does not cover the performance of pocket penetrometer readings.

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JKGeotechnics

BOREHOLE LOG

Borehole No.
101
1/2

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND AFTER 2 HRS	DS		0		TOPSOIL: Silty clay, high plasticity, dark grey, with root fibres.	w≈PL		200 240 200	GRASS COVER
	U50			CH	Silty CLAY: high plasticity, dark grey.	w≈PL	VSt	330 310 310 340 360 340	ALLUVIAL
			1				Hd	>600 >600 >600	
			2	CL	Sandy CLAY: low plasticity, brown, fine grained sand, with silt, trace of medium grained sand.	w<PL	VSt		
	DS	N = 11 5,6,5						240 340 360	
			3						
	DS	N = 10 4,5,5	4		as above, but trace of root fibres.			330 320 300	
				CI	Silty CLAY: medium plasticity, light grey and light brown mottled orange brown and dark grey, with fine to medium grained sand, and root fibres.		Hd		
	DS	N = 16 3,7,9	5					420 570 >600	
			6						
			7		as above, but light brown mottled light grey.				

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BOREHOLE LOG

Borehole No.
101

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■	N = 26 4,13,13			CI	Silty CLAY: medium plasticity, light grey and light brown mottled light grey, with fine to medium grained sand, and root fibres.	w<PL	Hd	480 530 510	
						8							
				■	N = 4 5,2,2				Silty CLAY: medium plasticity, light grey and light brown, with silt, trace of fine grained sand.	w≈PL	VSt	310 210 320	
						9							
				■	N = 19 5,8,11	10						360 370 340	
									END OF BOREHOLE AT 10.45m				
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No.
102
1/2

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND AFTER 4 HRS		0		FILL: Silty clay, medium plasticity, dark grey, trace of glass and root fibres.	w≈PL			GRASS COVER TRACE OF POSSIBLE ASBESTOS
			CH	Silty CLAY: high plasticity, dark grey.	w<PL	VSt- Hd	350 360 470 450 430	ALLUVIAL
	N = 16 5,8,8	1				Hd	>600 >600 >600 >600 >600 >600	
		2		as above, but dark brown mottled dark grey.	w<PL			
	N = 13 6,6,7	3					>600 >600 >600	
			CI	Sandy CLAY: medium plasticity, dark brown, fine to medium grained sand, trace of fine to medium grained rounded gravel.	w≈PL	(VSt)		
	N = 16 5,9,7	4	SP	Gravelly SAND: fine to coarse grained, brown mottled light grey, fine to medium grained rounded gravel, with coarse gravel, trace of silt fines, with clayey sand bands approx. 50mm.t.	M	MD		
		5						
	N = 13 4,6,7	6						
		7						

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BOREHOLE LOG

Borehole No.
102

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		N = 11 7,7,4			SP	Gravelly SAND: fine to coarse grained, brown mottled light grey, fine to medium grained rounded gravel, with coarse gravel, trace of silt fines, with clayey sand bands approx. 50mm.t.	M	MD		
			8		CH	Silty CLAY: high plasticity, light grey.	w≈PL	VSt	340 370 370	
		N = 17 4,8,9							370 350 340	
			9			END OF BOREHOLE AT 9.0m				
			10							
			11							
			12							
			13							
			14							

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BOREHOLE LOG

Borehole No.
103
1/2

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.8m
Date: 20/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

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			TOPSOIL: Silty clay, medium plasticity, dark grey brown, with root fibres.	w≈PL	VSt- Hd		GRASS COVER
					CI-CH	Silty CLAY: medium to high plasticity, dark grey brown, trace of root fibres.	w≈PL	VSt- Hd	320 440 450	ALLUVIAL
	DB					as above but with tree root 5mm.t.			320 410 410	
	DS	N = 12 5,6,6	1					Hd	>600 >600 >600	
	DB, ES		2							
			3						>600 >600 >600	
					CH	Silty CLAY: high plasticity, brown, trace of fine grained sand.		VSt		
	DS	N = 7 2,3,4	4						320 360 390	
			5		SP	SAND: fine to medium grained, brown, trace of silt.	M	L		
	DS	N = 7 4,3,4								
			6			as above, but with medium plasticity clay, brown mottled grey.				
						as above, but brown mottled light grey, with fine				
			7							

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BOREHOLE LOG

Borehole No.
103
2/2

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.8m
Date: 20/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■	N = 9 5,4,5			SP	to coarse grained rounded gravel, without clay.	M	L		
						8			SAND: fine to medium grained, brown mottled light grey, with fine to coarse grained rounded gravel, trace of silt.				
				■	N = 46 5,23,23			CI	Silty CLAY: medium plasticity, light grey mottled light brown, with fine gained sand.	w<PL	Hd	570 550 >600	
						9			END OF BOREHOLE AT 9.0m				
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No.
104
1/1

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

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			TOPSOIL: Silty clay, medium plasticity, dark grey, with root fibres.	w≈PL			GRASS COVER
								CI	Silty CLAY: medium plasticity, dark grey, trace of root fibres.	w≈PL	Hd		ALLUVIAL
				■	N = 13 5,6,7							>600 >600 >600	
						1							
									as above, but dark grey mottled brown, without root fibres.				
				■	N = 14 5,7,7							>600 >600 >600	
						2			END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
105
1/1

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.6m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION						0			TOPSOIL: Silty clay, medium plasticity, dark grey, with root fibres.	w≈PL			GRASS COVER
								CI	Silty CLAY: medium plasticity, dark grey, trace of root fibres.	w≈PL	Hd		ALLUVIAL ORGANIC ODOUR
					N = 10 5,5,5				as above, but with tree root 20mm.t			>600 >600 >600	
						1							
					N = 13 5,6,7				as above, but without root fibres.			>600 >600 >600	
						2			END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
106
1/1

Client: HI
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.7m
Date: 19/7/23 **Datum:** AHD
Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

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			TOPSOIL: Silty clay, medium plasticity, dark grey, with root fibres.	w≈PL			GRASS COVER
					CI	Silty CLAY: medium plasticity, dark grey, trace of root fibres.	w≈PL	Hd		ALLUVIAL
	DS	N = 12 5,6,6							>600 >600 >600	
			1			as above, but with tree root 10mm.t				
							w<PL	Hd		
	DS	N = 20 7,9,11							>600 >600 >600	
						as above, but without root fibres.				
			2			END OF BOREHOLE AT 2.0m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **107**

1/1

Client: HI

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.7m

Date: 19/7/23 **Datum:** AHD

Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

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			TOPSOIL: Silty clay, medium plasticity, dark brown, with root fibres.	w≈PL			GRASS COVER
								CI	Silty CLAY: medium plasticity, dark grey, trace of root fibres.	w≈PL	Hd		ALLUVIAL
				■	N = 14 4,7,7							>600 >600 >600	ORGANIC ODOUR
						1			as above, but with tree root 5mm.t				
				■	N = 12 5,6,6				as above, but without root fibres.			>600 >600 >600	
						2			END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
108
1/1

Client: HI

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 35 ALICE STREET, MOREE, NSW

Job No.: 35092UR2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 208.8m

Date: 20/7/23 **Datum:** AHD

Plant Type: GEOPROBE **Logged/Checked by:** C.S./P.R.

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			TOPSOIL: Silty clay, medium plasticity, dark grey brown, with root fibres.	w≈PL			GRASS COVER
								CI	Silty CLAY: medium plasticity, dark grey, trace of root fibres.	w≈PL	Hd		ALLUVIAL
					N = 13 4,6,7							>600 >600 >600	ORGANIC ODOUR
						1			as above, but with tree root 5mm.t				
					N = 20 7,10,10							>600 >600 >600	
						2			as above, but without root fibres.				
									END OF BOREHOLE AT 2.0m				
						3							
						4							
						5							
						6							
						7							

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AERIAL IMAGE SOURCE: EARTH.GOOGLE.COM

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

Title:	**SITE LOCATION PLAN**
Location:	MOREE HOSPITAL, 35 ALICE STREET, MOREE, NSW
Report No: 35092UR2	Figure No: 1

JKGeotechnics

PLOT DATE: 26/07/2023 9:21:34 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35092UR MOREE\CAD\35092UR2.DWG

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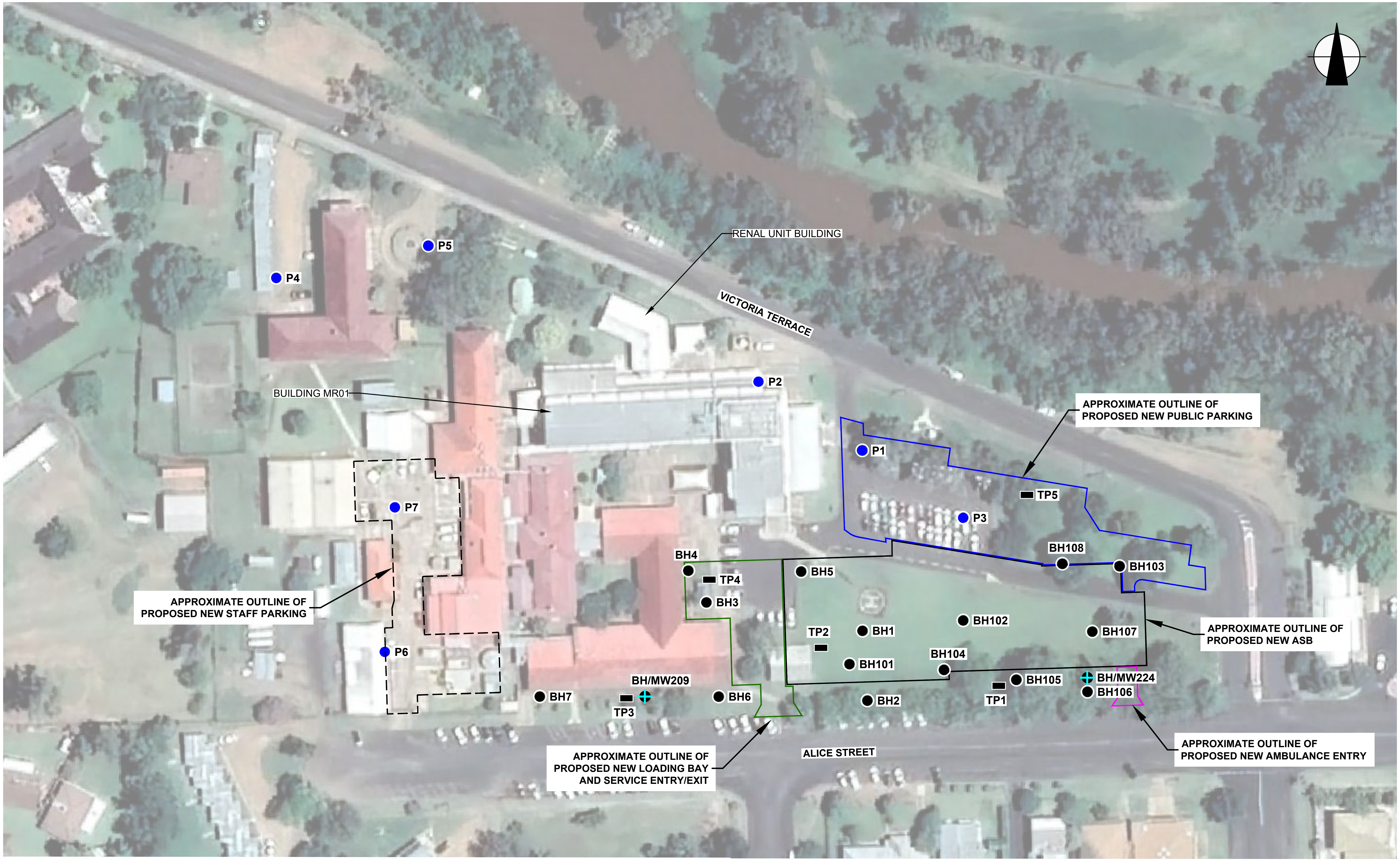

LEGEND

- ● BOREHOLE (BH1 TO BH7 JKG 2022 INVETIGATION, BH101 TO BH108 CURRENT JKG INVESTIGATION AND BH/MW209 AND 225, JKE DSI, 2023)
- ● PAVEMENT BOREHOLE (JKG 2022 INVESTIGATION)
- ▬ TEST PIT (REFER TO JKE 2022 REPORT)

AERIAL IMAGE SOURCE: EARTH.GOOGLE.COM

0 10 20 30 40 50

SCALE 1:1000 @A3 METRES

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

Title:	**TEST LOCATION PLAN**
Location:	MOREE HOSPITAL, 35 ALICE STREET, MOREE, NSW
Report No:	35092UR2
Figure No:	2

JKGeotechnics

PLOT DATE: 6/09/2023 2:15:28 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35092UR MOREE\CAD\35092UR2.DWG

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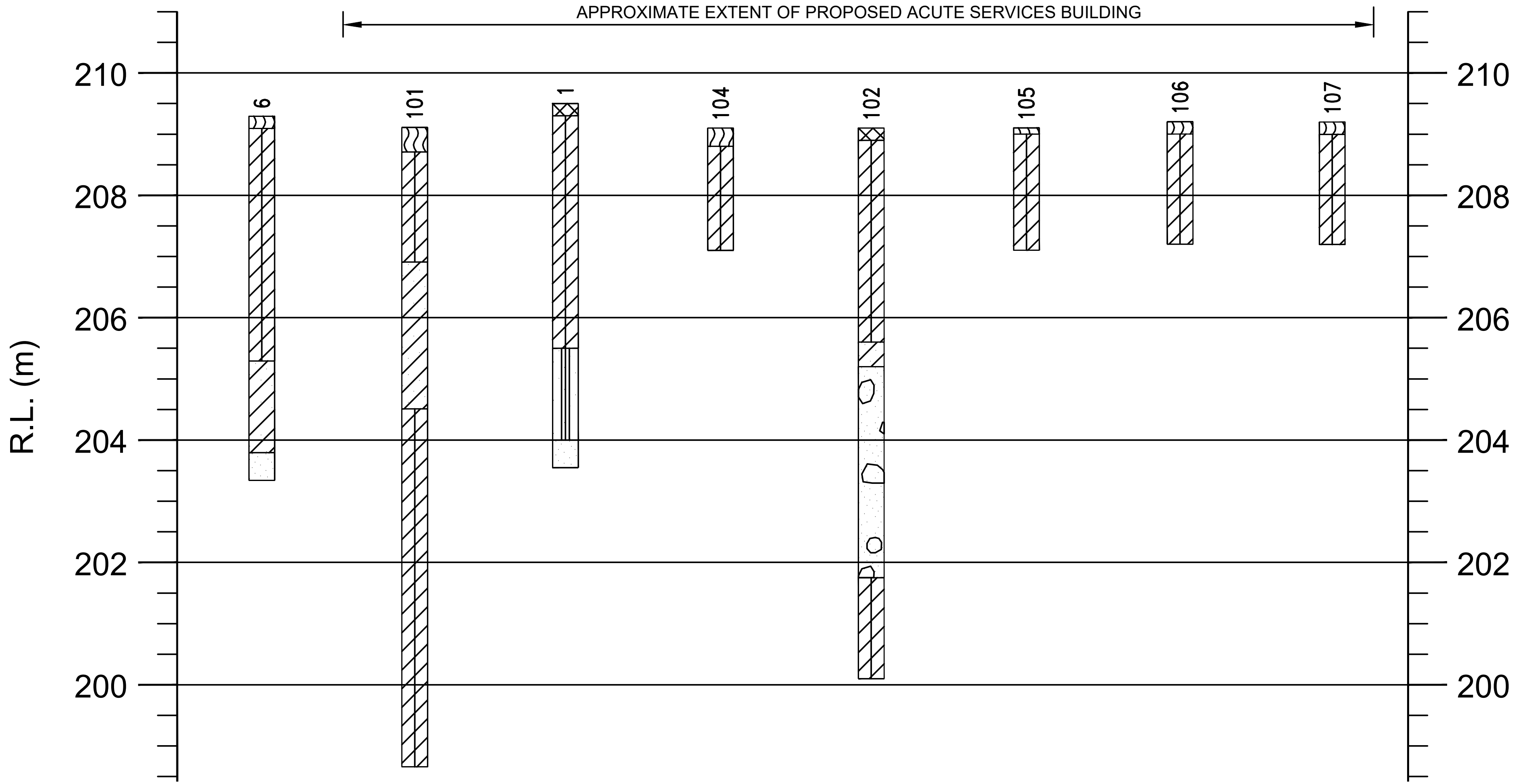

Fill
Topsoil
N SPT "N" VALUE
Silty Clay
Sandy Clay
Nc SOLID CONE BLOW COUNTS PER 150mm
Sand
Groundwater seepage level

NOTE:
REFER TO BOREHOLE AND TEST PIT LOGS.

0 1 2 3 4 5
V:SCALE 1:100 @A4 METRES

HORIZONTAL: NOT TO SCALE

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

Title:	**GRAPHICAL BOREHOLE SUMMARY**
Location:	MOREE HOSPITAL, 35 ALICE STREET, MOREE, NSW
Report No: 35092UR2	Figure No: 3

JKGeotechnics

PLOT DATE: 7/09/2023 11:21:00 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35092UR MOREE\CAD\35092UR2.DWG

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REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

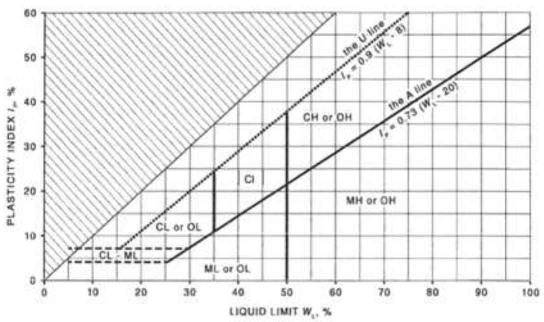
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES 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)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	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.
(Coarse Grained Soils)	D M W	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)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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