REPORT TO

HI

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

GUNNEDAH HOSPITAL, MARQUIS STREET, GUNNEDAH, NSW

Date: 15 August 2022

Ref: 35091URrpt

JKGeotechnics

www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801

Report prepared by:

Paul Roberts
Principal Associate | Engineering Geologist

Report reviewed by:

Peter Wright
Principal | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 503,
Cooroy, QLD, 4563
Head Office:
PO BOX 976
NORTH RYDE BC NSW 1670

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
35091URrpt	Final Report	15/08/22

This report (which includes all attachments and annexures) has been prepared by JK Geotechnics (JKG) for its Client, and is intended for the use only by that Client.

This Report has been prepared pursuant to a contract between JKG and its Client and is therefore subject to:

a) JKG's proposal in respect of the work covered by the Report;
b) The limitations defined in the Client's brief to JKG;
c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.

If the Client, or any person, provides a copy of this Report to any third party, such third party must not rely on this Report, except with the express written consent of JKG which, if given, will be deemed to be upon the same terms, conditions, restrictions and limitations as apply by virtue of (a), (b), and (c) above.

Any third party who seeks to rely on this Report without the express written consent of JKG does so entirely at their own risk and to the fullest extent permitted by law, JKG accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

At the Company's discretion, JKG may send a paper copy of this report for confirmation. In the event of any discrepancy between paper and electronic versions, the paper version is to take precedence. The USER shall ascertain the accuracy and the suitability of this information for the purpose intended; reasonable effort is made at the time of assembling this information to ensure its integrity. The recipient is not authorised to modify the content of the information supplied without the prior written consent of JKG.

Table of Contents

1 INTRODUCTION 1

2 INVESTIGATION PROCEDURE 2

3 RESULTS OF INVESTIGATION 3

3.1 Desk Top Study 3

3.2 Site Description 4

3.3 Subsurface Conditions 4

3.4 Existing Footings 6

3.5 Laboratory Test Results 6

4 COMMENTS AND RECOMMENDATIONS 7

4.1 Site Preparation 7

4.1.1 Dilapidation Surveys 7

4.1.2 Demolition and Excavation 7

4.1.3 Seepage 8

4.1.4 Temporary Batters 8

4.2 Earthworks 9

4.2.1 Site Drainage 9

4.2.2 Subgrade Preparation 9

4.2.3 Engineered Fill 10

4.2.4 Trench Backfill 11

4.3 Retention and Permanent Batter Slopes 12

4.3.1 Retention Design Parameters 12

4.3.2 Retaining Walls Supporting Engineered Fill 12

4.3.3 Permanent Batter Slopes 13

4.4 Footing Design 13

4.4.1 Site Classification and Soil Reactivity Considerations 13

4.4.2 Footing Systems and Design Parameters 14

4.5 Earthquake Design Classification 16

4.6 Floor Slabs 16

4.7 Pavement Design and Construction 17

4.7.1 Pavement Design Considerations 17

4.7.2 Pavement Thickness Design 17

4.7.3 Pavement Materials 18
4.8 Soil Aggression 18
4.9 Site Stability 18
4.10 Further Geotechnical Input 19
5 GENERAL COMMENTS 19

ATTACHMENTS

East West Enviroag Pty Ltd 'California Bearing Ratio Report Sheets (Report No. EW225312-1 and 2)
East West Enviroag Pty Ltd 'Soil Classification Test Report' (Report No. EW225312-3 and 4).
East West Enviroag Pty Ltd 'Analysis Report Soil' (EW221209 Report No. 1)
Table 1: Flexible Pavement Thickness Recommendations
Borehole Logs 1 to 8
Test Pit Logs 1 to 6
Figure 1: Site Location Plan
Figure 2: Test Location Plan
Figure 3: TP101 Cross Sectional Sketch
Figure 4: TP102 Cross Sectional Sketch
Figure 5: TP103 Cross Sectional Sketch
Figure 6: Graphical Borehole and Test Pit Summary (BH6, BH5, BH3, TP3 and BH2)
Figure 7: Graphical Borehole and Test Pit Summary (TP5, TP6, BH7, BH4, TP2, BH1 and TP1)
Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations to Gunnedah Hospital, Marquis Street, Gunnedah, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Health Infrastructure (Contract No. HI22038GU and Variation No. 1) via emails respectively dated 2 May 2022 and 19 July 2022. The commission was on the basis of our fee proposals (Ref. P56125UR) dated 9 September 2021 and (Ref. P56125UR Variation No 1) dated 1 June 2022.

We have been provided with the following relevant information:

- Survey plan (Reference Number 22-0056 Rev. 4, dated 31 May 2022) prepared by Monteath & Powys.
- Architectural drawings (Drawing Numbers AR-00-AA0000, AA0010, AA1000, AA1001, AA1019, AA1020, AA1220, AA9001, AA9002, AR-EW-AA1100, AA1101, AA1110, AR-MW-AA1200, AA1201, AA1209, AA1210, AA1212, AA3001, AR-RW-AA1100, AA1101, AA1109, AA1110, AA1112, AA1209, AA1210 and AA1212, Issue A, dated 10 June 2022) prepared by Design Worldwide Partnership (DWP).
- Structural drawings (Drawing Numbers SK01.1 to SK01.3, SK02.1 and SK02.2 Rev. 1, dated 9 June 2022) prepared by Northrop.
- Civil engineering drawings (Drawing Numbers DAC04.01 and 02 Rev. 02, dated 14 July 2022) prepared by Northrop.

Based on a review of the provided information, initial information presented in the Request For Tender (RFQ) documentation and additional information provided by Hadyn Douglas (Ranbury) and Trudy Myers (Northrop), we understand that the proposed alterations and additions will be carried out in three stages; Early Works, Main Works and Refurbishment Works. Following partial demolition required for each of the stages, the proposed alterations and additions will include:

- A new single level inpatient unit building situated over the central portion of the hospital grounds, an extension to the existing kitchen building and a new emergency access situated respectively to the south-west and to the east of the new inpatient unit building. The maximum working column loads are expected to be approximately 600KN based on a typical 6.5m x 6.5m grid column spacing and line loads in the order of 65 to 70kN/m. The floor slabs will either be suspended between bored piers or a raft slab will be formed with integrated pad footings at column locations. Where adopted, the suspended floor slab will either be supported by sacrificial formwork ('Bondek') or formed over a subgrade comprising engineered fill and natural ground. In areas the design surface levels would need to be raised by a maximum of 0.7m (by placing fill) or lowered by a maximum of about 0.2m by excavation.
- The existing ward building to the north-east of the new inpatient unit building will be reconfigured and a new concrete floor slab provided in the sub-floor space; localised excavations to a maximum of about 0.2m will be required to achieve design surface levels. Existing footings will support additional loads and underpinning may be required, new pad footings will tie into existing footings or be positioned to avoid clashes with existing footings.
- Additional car parking areas and access roads will be provided over the north-western, north-eastern, southern and south-eastern portions of the hospital grounds. In the main, the new parking areas will involve extending existing parking areas. It appears that in the main new parking areas and access roads will be formed at similar levels to existing surface levels with only nominal excavations (less than 0.5m)

locally required to achieve design surface levels. The design traffic loading for the car parks and access roads has been assumed to be 1 x 10^5 ESA's (Equivalent Standard Axles).

- Landscaping of sections of the hospital grounds including regrading of the link between the new main entry to the inpatient unit building north-eastwards to the rear (south-eastern side) of the Rural Health Centre. The access ramp will require raising of site surface levels by a maximum of about 1.4m.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on the geotechnical aspects of the proposed development, such as 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, footing design, earthquake site classification to AS1170.4 – 2007, earthworks, pavement design parameters, pavement materials, a flexible pavement thickness design, any slope stability issues and suitable methods to improve stability, if required.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate reports by JKE, Ref: E35091UPDrpt, dated 1 August 2022 and E35091BTrpt-HAZ, dated August 2022, for the results of the environmental site assessments.

2 INVESTIGATION PROCEDURE

The investigation included:

- A desk top study of available geotechnical and geological mapping, soil landscape information, other published information (where available), our database of nearby investigations and historical aerial imagery in order to identify any risks in relation to the expected geology, soils and instability.
- A subsurface investigation including:
  - Eight boreholes (BH1 to BH8) drilled using a drill rig to depths between 6.0m and 10.45m.
  - Six test pits (TP1 to TP6) excavated using a bucket attachment to a 5 tonne excavator to depths between 0.3m and 1.0m. The test pits were primarily excavated for environmental investigation purposes.
  - Three test pits (TP101 to TP103) excavated using hand held tools to depths between 1.1m and 1.2m in order to expose existing building footings.

Prior to the commencement of the fieldwork, the test locations were scanned using electro-magnetic equipment for the presence of buried services by a specialist sub-contractor.

The test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features and were situated as close as practicable to the test locations nominated by Northrop. 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 'Nearmap' with the outline of the proposed ground floor level of the new inpatient unit building and the proposed pavement areas superimposed. The survey datum is the Australian Height Datum (AHD).

The compaction of the fill, relative density and strength of the natural sands and clay soils 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 and sides of test pits. The strength of the bedrock was assessed from observation of drilling resistance when using a tungsten carbide ('TC') bit and examination of the recovered rock cuttings. The assessment of rock strength in this way is approximate, and variations of about one order of strength should not be unexpected.

Groundwater observations were made in the boreholes and test pits during and on completion of auger drilling and excavation. No longer term ground monitoring has been carried out.

The fieldwork for the investigation was carried out under the direction of RGS Geotechnical Engineers, who were present full-time on site, and set out the test locations, directed the buried services scan, logged the encountered subsurface profile, nominated in-situ testing and sampling and prepared the test pit cross sectional sketches (presented as Figures 3, 4 and 5). The borehole and test pit logs and the test pit cross sectional sketches (which include field tests and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

Selected soil samples were returned to the East West Enviroag Pty Ltd (EWEPLP) NATA registered laboratory for moisture content, Atterberg Limits, linear shrinkage, Standard compaction and four day soaked CBR testing and soil pH, chloride and sulfate content and resistivity testing. The results are summarised on the attached EWEPLP 'Soil Classification Test Reports' (Report No. EW225312-1 and 2, dated 6 July 2022 and 9 August), 'California Bearing Ratio Report Sheets' (Report No. 2022EW225312-3 and 4 dated 6 July 2022) and 'Analysis Report Soil' (Report No. 1, dated 28 June 2022).

3 RESULTS OF INVESTIGATION

3.1 Desk Top Study

Reference to the Manilla 1:250 000 scale geological map and the report (Ref. LS032436 EP, dated 19 May 2022) prepared by Lotsearch Pty Ltd (LPL) for the JKE assessment indicates that the site is underlain by Quaternary age alluvial deposits comprising sands, silts and clays close to the interface with Quaternary to Pleistocene age undifferentiated colluvial and residual soils overlying the Werrie Basalt (essentially comprising basalt or rhyolite).

Based on the available soil mapping summarised in the LPL report the soil landscape comprises 'broken topography' characterised by 'undulating low hilly terrain' intersected by step sided gullies. The soils primarily comprise 'hard alkaline red soils' in the undulating terrain and the steep ridges with various soils, including (sands, dispersible solodic soils and sandy 'red earth soils') often covered with rounded gravels on gentler slopes.

Based on our experience the geology of the area is expected to comprise the Quaternary age alluvial deposits comprising a mixed soil profile of alluvial sand, silt and gravel close to the interface with the Werrie Basalt and sedimentary Gladstone Formation. Residual clays derived from the weathering of the igneous and/or sedimentary bedrock would be expected above the bedrock. Nearby investigations have encountered bedrock at depths in the order of about 6.0m to 9.0m.

The drillers log information from the registered bores within a 500m radius of the site typically identified silty clay soil with some gravels and, where bores extended to sufficient depth, encountered 'weathered' sandstone and mudstone below 18.0m depth. Standing water levels in the bores ranged between 2.8m and 6.5m depth.

3.2 Site Description

The site is situated within the grounds of Gunnedah Hospital which is located within gently undulating relatively flat topography which overall gently slopes down to the north at a maximum of approximately 2°. The hospital site has south-eastern, south-western and north-western frontages onto Anzac Parade, Reservoir Street and Marquis Street, respectively.

At the time of the fieldwork the hospital site contained a number of single storey and in places one to two storey brick and weatherboard clad buildings with grass covered surrounds, brick paved and concrete paved areas were scattered throughout the site together with concrete paved driveways and asphaltic concrete (AC) paved car park areas.

Locally planter beds were supported by timber retaining walls (maximum height about 1.0m). A concrete block retaining wall (maximum height about 1.2m) supported an elevated area to the south and south-east of the Rural Health Centre building located over the north-western portion of the site. Based on the nearby BH5 it is likely that the retaining wall supported an area of fill.

Trees (approximately 5.0m maximum height) were observed along the southern site boundary and in landscaped areas throughout the site. Small shrubs were observed adjacent to some of the hospital buildings.

Surface levels were similar across the site boundaries and extended north-east beyond the site into the neighbouring grounds of Alkira Nursing Home to the north-east. The nursing home was occupied by one and two storey brick buildings with grass covered surrounds and scattered trees.

3.3 Subsurface Conditions

The boreholes and test pits have disclosed a generalised profile comprising a variable thickness of fill overlying typically alluvial clays but occasionally overlying residual clays with weathered basalt bedrock encountered at moderate depth. Groundwater was intermittently encountered at moderate depth in the alluvial soils. Reference should be made to the attached borehole and test pit logs and test pit cross sectional sketches for specific details at each location. A summary of the pertinent subsurface characteristics is presented below and a graphical summary of the subsurface profile along two sections orientated north-west to south-east and extending from the Marquis Street frontage to Anzac Parade are presented as Figures 6 and 7.

Paved Surface

A concrete paved surface was encountered in BH7 and was 0.15m thick.

Fill

Silty or gravelly clay topsoil fill assessed to be of medium plasticity was encountered from surface level in TP1 and TP3 to TP6 and was 0.1m to 0.3m thick.

Silty or sandy clay assessed to be of variable plasticity was encountered from surface level in BH1 to BH3 and BH8 and extended to depths between 0.6m and 0.9m. Gravelly sandy clay and sandy clay fill assessed to be of low plasticity was encountered from surface level in TP101 to TP103 and was 0.2m thick. From surface level or below the topsoil fill gravelly (occasionally sandy) clay fill with occasional sand or clayey gravel bands (about 0.1m thick) was encountered in TP1 to TP6. A thin band of asphalt was encountered in TP5 immediately below the topsoil fill and may represent a buried paved surface.

Sandy fill with varying gravel content and sandy gravel fill was encountered in BH4 to BH7 and extended to depths between 0.1m and 1.6m. Below the paved surface in BH7 the initial 0.1m thickness of sandy gravel has been interpreted to represent base course materials.

The fill extended to the alluvial clays in all boreholes and test pits with the exception of TP1 to TP3 and TP5 which were terminated in the fill at depths between 0.3m and 1.0m. TP3 was terminated in the fill at 0.3m depth due to a fire hydrant water pipe being intercepted and damaged (and subsequently repaired).

Due to the limited thickness of fill encountered only a limited number of SPT tests were carried out. However, based on the SPT 'N' values the fill was assessed to be poorly compacted. In addition, 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'.

Alluvial Clays

Alluvial silty clays and sandy clays with variable sub-rounded gravel content and typically of low to medium plasticity were encountered below the fill in all the boreholes, TP4 and TP6 and TP101 to TP103. On first contact, the alluvial clays were firm to stiff strength in TP101, stiff strength in in BH5, BH6, BH7, TP4, TP6, TP102 and TP103, and very stiff strength in BH1 to BH4 and BH8 and improved to very stiff or hard strength with depth. With the exception of BH8, all the boreholes and TP4, TP6 and TP101 to TP103 were terminated in the alluvial clays.

Weathered Basalt Bedrock

Extremely weathered basalt material assessed to be very dense clayey gravel was encountered at 4.0m depth below the alluvial clays in BH8. The 'TC' bit refusal at 4.15m depth may be interpreted as representing basalt bedrock of at least medium to high strength.

Groundwater

All test pits were 'dry' during and on completion of excavation. Discernible seepage was not encountered whilst auger drilling the boreholes and BH8 was 'dry' on completion of auger drilling. However, standing water levels were recorded in BH1 to BH7 at depths between 2.5m and 3.5m on completion of auger drilling. No long-term groundwater level monitoring was carried out.

3.4 Existing Footings

The details of the existing footings exposed in TP101 to TP103 are summarised below:

- TP101 exposed a brick wall that extended to the top of a concrete strip footing at 0.6m depth. The concrete footing stepped out 0.15m from the wall face and was 0.4m thick. The footing was founded at 1.0m depth in stiff to very stiff alluvial clay; refer to Figure 3 for more details.
- TP102 exposed a brick wall that extended to the top of a concrete strip footing at 0.75m depth. The concrete footing stepped out 0.15m from the wall face and was 0.35m thick. The footing was founded at 1.1m depth in very stiff alluvial clay; refer to Figure 4 for more details.
- TP103 exposed a brick wall that extended to the top of a concrete strip footing at 0.6m depth. The concrete footing stepped out 0.22m from the wall face and was 0.4m thick. The footing was founded at 1.0m depth in very stiff alluvial clay; refer to Figure 5 for more details.

3.5 Laboratory Test Results

Based on the Liquid Limit and Linear Shrinkage determination, the samples of alluvial silty clay tested from BH3 and BH4 and the alluvial sandy clay tested from BH7 and TP102 were of low to medium plasticity with an assessed moderate to high potential for shrink/swell reactivity with changes in moisture content.

The four day soaked CBR values of the alluvial sandy clay samples from BH1 and BH5 returned values of 4% when compacted to 100% of Standard Maximum Dry Density (SMDD) and surcharged with 4.5kg. The natural moisture content of the samples tested in BH1 and BH5 were respectively 0.5% and 1.2% 'wet' of the Standard Optimum Moisture Contents (SOMC).

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)
BH2	2.5-2.95	Silty Clay ALLUVIAL	8.4	13.4	50.9	24.39
BH6	4.0-4.45	Sandy Clay ALLUVIAL	8.1	19.6	57.9	35.71

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

4.1.1 Dilapidation Surveys

Prior to site preparation works Council may require that dilapidation survey reports be completed on their assets lining the street frontages, i.e. the paved footpaths, roadways, kerbs and gutters. We recommend that the property owners be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the client in defending themselves from unfair damage claims.

4.1.2 Demolition and Excavation

Site preparation will require demolition of selected existing buildings, structures and paved surfaces, stripping of topsoil and/or root affected soils and removal of the existing uncontrolled fill. Any obviously deleterious or contaminated existing fill should be removed in accordance with the advice presented in the JKE report. The stripped contaminated materials should be taken off-site as they are not suitable for re-use as engineered fill.

The topsoil and/or root affected soils may also be separately stockpiled and used for subsequent landscaping purposes. The existing clayey fill will not be suitable for re-use as engineered fill without moisture conditioning or possibly lime stabilisation, and if either of these actions are not undertaken should be removed from site. However, existing gravelly or sandy fill materials may be reused as select fill provided it is separately stockpiled; thorough blending with the clayey soils would aid workability.

Tree root systems dry out the surrounding clayey soils and their removal will result in localised moisture recovery leading to swelling which may have a detrimental impact on the performance of nearby buildings and paved surfaces founded/supported in the clayey soil profile within the site. Therefore, trees should only be removed where absolutely necessary and as soon as practicable, in order for the moisture content of the clayey subsoils to recover; ideally this would be years in advance of construction though we understand this is not practical here.

Localised excavations to a maximum depth of about 0.5m will be required to achieve the design surface levels and elsewhere engineered fill will need to be placed to raise site surface levels (maximum height about 1.4m).

Demolition, localised excavations and the earthworks will need to be carefully completed in order to maintain the stability of the adjacent sections of existing hospital buildings and structures that will remain and in this regard, excavations should not extend below a line drawn downward from any footing at 1V in 2H unless temporary shoring is installed, although this is unlikely to be required. This work will need to be completed using suitably experienced (and insured) contractors.

If the existing concrete block retaining wall immediately to the north and north-west of the proposed inpatient unit building and new access road is to remain then we recommend that it is inspected by the structural engineer. Based on inspection by the structural engineer, the need and extent of any necessary wall strengthening measures can be determined and detailed. Test pits to expose the retaining wall footings and possibly further existing ward building footings (in addition to TP101 to TP103) may be required in order to confirm footing details and the foundation materials.

Where locally required to achieve design surface levels, excavations will encounter the fill and possibly alluvial clays which we expect to be readily achievable using tracked excavators.

We do not recommend that rock breakers be used for demolition close to existing buildings or structures as they could be adversely affected by ground vibrations. We recommend that the removal of concrete paved surfaces, floor slabs and footings be completed using a diamond saw followed by removal of the concrete pieces using ripping tyne and bucket attachment to the tracked excavator. Where access is restricted hand-held equipment, including demolition saws, may be required.

If rock breakers are proposed to be used during demolition, then further advice should be sought regarding the need for quantitative vibration monitoring.

4.1.3 Seepage

Groundwater seepage was encountered in the boreholes with standing water levels at depths of at least 2.5m. Some ephemeral seepage inflows may be encountered in the excavations, particularly after periods of heavy rain.

In general, we expect that inflows, if any, to be very small and managed by conventional sump and pump techniques or gravity drainage. Inspection and monitoring of groundwater seepage during excavation is recommended, so that any unexpected conditions, which may be revealed can be incorporated into the drainage design.

4.1.4 Temporary Batters

Temporary excavation batter slopes through the clayey soil profile no steeper than 1V in 1H are considered to be appropriate, provided surcharge loads such as from plant and stockpiles of material are kept well clear of the crests of the temporary batter slopes, say at least 1.5m back. Flatter batter slopes of 1V in 2H will be appropriate for sandy/gravelly soils. These temporary batters are expected to be achievable within the site geometry. Steeper (sub-vertical) temporary soil batters may be considered in the clay fill or natural clays, say for trench excavations, but would only be feasible for cut faces of less than about 1.0m height and on condition no structures were located within a horizontal distance equivalent to at least twice the vertical height of the cut and the retaining walls were constructed or trenches backfilled as soon as practicable. With regard to service trenches, if there are concerns regarding the stability of existing buildings and structures in close proximity to the sub-vertical trench excavations, a temporary trench shoring system would be required and would also be necessary if sub-vertical batters are preferred and proposed trench depths are in excess of 1.0m.

Some instability of temporary sand batters may occur at, or below, the level of any groundwater seepage, especially after rain periods, and sand bagging may be required to stabilise the lower portion of these batters. Sub-vertical sand batters for the trench sides would need to be appropriately shored and dewatered if seepage was encountered; further detailed geotechnical advice would be required in this instance, but is believed to be unlikely.

4.2 Earthworks

The following earthworks recommendations should be complemented by reference to AS3798-2007 “Guidelines on Earthworks for Commercial and Residential Developments”.

Following site preparation as outlined in Section 4.1 above, the subgrade of the proposed buildings will comprise a mix of engineered fill (where site surface levels need to be raised) and/or moderately to highly reactive, low to medium plasticity alluvial clays (in localised areas of cut). We assume that the new car park areas and access roads will typically be formed at similar surface levels to the adjacent pavements and/or existing area of the hospital grounds, with localised areas of cut (maximum 0.5m depth). Based on the investigation results and following site preparation as outlined in Section 4.1 above, the car park and access road pavements will be constructed over a subgrade comprising reactive alluvial clays. Further discussion of reactive soil movements is presented in Section 4.4.1 below.

4.2.1 Site Drainage

The clayey soils at the site are expected to undergo some loss in strength when wet. Furthermore, based on our investigation results the clay fill and alluvial clay soil subgrades are expected to have a moderate to high shrink-swell reactive potential. Therefore, it will be important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clayey subgrade may become un-trafficable when wet, and consideration should be given to providing a crushed rock or crushed concrete working platform to minimise delays following rainfall. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

Good surface and subsurface drainage must also be provided post construction to improve the long-term performance of the buildings and external paved areas.

4.2.2 Subgrade Preparation

Following excavation to design levels and stripping of surface materials as described in Section 4.1.2 above, the exposed soil subgrade should be inspected by an experienced geotechnical engineer. In general, the soils have moisture contents in excess of their plastic limits and it is possible that heavy rollers may get ‘bogged’. Following inspection by the geotechnical engineer proof rolling should then be carried out where the subgrade is deemed suitable using an 8 tonne smooth drum roller operated in the static (non-vibration) mode under the direction of the geotechnical engineer in order to detect any unstable or soft areas. Where access is restricted, say at the margins of the site areas close to existing buildings and structures, we expect that proof rolling will be completed using a small (say 3 tonne) smooth drum roller.

If soft or heaving areas are detected during the geotechnical inspection or the proof rolling, they should be locally removed to a stable base and replaced with engineered fill, as outlined in Section 4.2.3 below, or further geotechnical advice should be sought. Further guidance on the treatment of heaving areas must be provided by the geotechnical engineer during or following the proof rolling inspection.

Alternatively, it may be preferred to treat any thicker or more laterally extensive poor subgrade areas (if encountered) using a 'bridging layer' which would involve over-excavating material by about 0.5m, then placing, tracking and rolling in a thin layer of hard/durable well graded angular 75mm to 150mm (with <10% 'fines') select fill and with subsequent layers placed until no more granular fill can be tracked into the subgrade. The area may then be static proof rolled (as described above) at least 24 hours after placing the select fill 'bridging' layer under the direction of a geotechnical engineer. Following satisfactory completion of the rolling of the 'bridging layer', engineered fill may then be placed in thin layers without vibration due to the potential for pumping of 'fines', to achieve design surface levels.

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

Where the exposed subgrade below the road pavements comprises natural clay soils, they must be ripped to a depth of 0.15m and recompacted to at least 100% of Standard Maximum Dry Density (SMDD), with a moisture content within 2% of the Standard Optimum Moisture Content (SOMC).

4.2.3 Engineered Fill

For raising of site surface levels and treatment of any poor subgrade areas, engineered fill should be used.

Engineered fill should be a well graded select granular material such as crushed sandstone free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 75mm. Site won clays from the excavations may need to be disposed of due to their high moisture contents (unless moisture conditioning and possibly stabilisation with lime) and their reactive potential beneath floor slabs. Any existing sandy fill sourced from the excavations will be suitable for engineered fill, provided they are thoroughly mixed, with any coarse gravel size material removed. However, it is expected that the majority of fill required to raise site surface levels will need to be imported non-reactive granular fill to limit potential reactive movements, as discussed in Section 4.4.1 below. Engineered fill should be compacted using the above-mentioned roller in layers of maximum 200mm loose thickness to a density between 98% and 102% of SMDD and within 2% of their SOMC. Where granular materials are used the specification could be relaxed to a density at least 98% of SMDD.

Backfill to conventional retaining walls should also comprise engineered fill. Well graded granular materials such as crushed sandstone and demolition rubble would be suitable for this purpose. This granular fill should be free of deleterious substances and should have a maximum particle size not exceeding 40mm. Such fill should be compacted in horizontal layers as above using a hand-held plate compactor (e.g. whacker packer). Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

As an alternative to engineered fill for retaining wall backfill, single sized granular material (or 'no fines' gravel) may be used as backfill to retaining walls and this would also act as the drainage behind the wall and would only require nominal compaction (with no compaction testing). However, such material would not be suitable where building footings founded on Level 1 engineered fill are located close to the retaining wall backfill. The drainage material should be wrapped in a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion. Further, unless covered by the car park pavement, the free draining retaining wall backfill should be provided with a cap of clay or crushed bedrock of at least 0.3m thickness at surface level to reduce the likelihood of surface water entering the backfill and surcharging the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798 (Table 8.1) for the volume of fill involved. Where the fill is required to support structural loads, Level 1 testing as defined in AS3798 is recommended, if not, then Level 2 testing will be appropriate. Any areas of insufficient compaction will require reworking. The Geotechnical Testing Authority should be directly engaged by the client or their representative and not by the earthworks contractor.

4.2.4 Trench Backfill

Backfilling of the trenches should be carried out using engineered fill, as described above, in order to reduce post-construction settlements, particularly where trenches will extend below pavements.

Engineered fill should be compacted as outlined above using a trench roller or a pad foot roller attachment fitted to an excavator, to a density ratio of at least 98% of SMDD and a moisture content within 2% of SOMC.

To further reduce post-construction settlements, we recommend that the trench works are completed before the pavements are constructed and the compacted trench backfill would form localised sections of the prepared subgrade upon which the sub-base is placed.

Where the trench passes through landscaped areas, it may be possible to relax the compaction specification if post construction settlements are acceptable to the designers. In landscaped areas, each layer should be compacted to a density ratio of at least 95% of SMDD and a moisture content within 3% of SOMC.

Density tests should be carried out on the engineered backfill to confirm the above specifications are achieved as outlined in Section 4.2.3 above.

4.3 Retention and Permanent Batter Slopes

Permanent retaining walls will be required to support cut faces and engineered permanent batters are not suitable. Advice is provided below with regard to retention design, retaining walls supporting engineered fill and permanent batter slopes.

4.3.1 Retention Design Parameters

The following earth pressure coefficients and subsoil parameters may be adopted for the design of conventional retaining walls:

- For design of conventional walls that will be propped, backfilled and permanently supported by the structure, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.6 for the retained profile, assuming a horizontal backfill surface.
- For design of the conventional walls, where some movements of retaining walls may be tolerated, they may be designed using a triangular lateral earth pressure distribution and a coefficient of 'active' earth pressure, (ka), of 0.35 for the soil profile, assuming a horizontal backfill surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the retained profile.
- Any surcharge affecting the walls (e.g. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.
- Lateral restraint of retaining walls founded in the soil profile below adjacent surface levels or bulk excavation level may be provided by the passive pressure of the soil below these levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, using a triangular pressure distribution and provided a Factor of Safety of at least 2 is used in order to reduce the high deflections that are associated with achieving a full passive case. Localised excavations in front of the walls e.g. for buried services etc must also be taken into account in the design.

4.3.2 Retaining Walls Supporting Engineered Fill

The proposed access ramp between the new main entry to the proposed inpatient unit building and the Rural Health Centre will require raising of site surface levels by a maximum of about 1.4m, unless it is designed as a suspended structure. Where retaining walls are proposed to support the fill, the construction sequencing may involve:

- Fill initially placed and then cut back to allow retaining wall construction; this would only be feasible where there was sufficient space to place the fill outside the area of the works within the site.
- The retaining wall may be constructed first and then backfilled.

In either case, good compaction close to the retaining wall may not be feasible and some post-construction settlement of the fill surface may occur. In addition, care would be required not to transfer compaction stresses to the retaining wall, hence the use of free draining backfill, where appropriate.

4.3.3 Permanent Batter Slopes

If required, permanent fill batter slopes and soil cuts should be formed at no steeper than 1V in 2H. However, for ease of maintenance (such as mowing) flatter slopes of 1V in 4H would be appropriate. For the permanent fill batter slopes, this assumes that the fill is placed as engineered fill in accordance with the advice provided above. Surface erosion protection, for example, quick establishing grass or proprietary erosion protection systems, must be provided to the permanent batter slopes.

4.4 Footing Design

4.4.1 Site Classification and Soil Reactivity Considerations

Based on the results of our investigation, due to the presence of trees (some of which may be planted as part of proposed landscape works) and the removal of paved areas (both of 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 site but should be referenced for general guidance on footing and floor slab design and site maintenance.

Based on the results of the investigation and the design soil suction change depth for this climatic region (3.0m), the alluvial silty and sandy clays are highly reactive with changes in moisture content (i.e. similar to those expected for a Class ‘H1-D’ site). In this regard, if engineered fill is placed to raise site surface levels and is proposed to support structural loads then we note the following:

- If site won or imported reactive clays are used as engineered fill then the site classification will likely be more severe depending on material properties and fill thickness. In this case, further advice should be sought from JK Geotechnics.
- If imported non-reactive granular soils are used as engineered fill and assuming a thickness of at least 0.4m, then the predicted characteristic surface movements with changes in moisture content will reduce to those expected for a Class ‘M-D’ site.

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. If trees are to be planted, we recommend a root barrier extending to at least 3.0m depth be installed to protect the buildings.

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.

4.4.2 Footing Systems and Design Parameters

Based on advice provided by Northrop we note the following:

- For the new inpatient unit building the maximum working column loads are expected to be approximately 600KN based on a typical 6.5m x 6.5m grid column spacing and line loads in the order of 65 to 70kN/m. The floor slabs will either be suspended between bored piers or a raft slab will be formed with integrated pad footings at column locations. Where adopted, the suspended floor slab will either be supported by sacrificial formwork ('Bondek') or formed over a subgrade comprising engineered fill and natural ground. Where required, design surface levels would need to be raised by a maximum of 0.7m (by placing fill) or excavated to a maximum of about 0.2m.
- The existing ward building to the north-east of the new inpatient unit building will be reconfigured and a new concrete floor slab provided in the sub-floor space; localised excavations to a maximum depth of about 0.2m will be required to achieve design surface levels. Existing footings will support additional loads and underpinning may be required, new pad footings will tie into existing footing or be positioned to avoid clashes with existing footings.
- Where engineered fill is required to support structural loads then it must be placed under Level 1 control as outlined in Section 4.2 above. However, we note that appropriately qualified and experienced earthworks contractors and geotechnical testing authorities would be required in order to provide Level 1 engineered fill platforms to support structural loads. If there are concerns regarding the ability to source appropriate qualified contractors then to reduce the risk of poor long-term performance and potential building damage, our recommendation would be to adopt structures and floor slabs suspended from piled footings with void formers between the subgrade and suspended structures and slabs.
- We do not recommend that site won reactive clay soils be used as engineered fill due to potential problems with moisture conditioning and the increase in predicted reactive soil movements unless the clays are stabilised with the addition of lime. Our strong recommendation is that imported non-reactive (granular) engineered fill be used where structural loads are to be supported; further advice is presented in Section 4.2 above.

We consider that pad or strip footings, a raft slab or pile footings are suitable footing systems provided they are designed and constructed in accordance with the advice presented in this report.

High level footings (pad footings, strip footings and raft slabs) founded on fill placed under Level 1 testing or natural clays of at least stiff strength may be designed for an ultimate limit state bearing pressure of 300kPa or a serviceability (allowable) limit state bearing pressure of 100kPa. For raft slab design the following elastic parameters may be adopted:

- Imported non-reactive granular fill: Elastic Modulus 40MPa (short term) and 28MPa (long term), Poisson's Ratio of 0.3.

- Stiff alluvial clays: Elastic Modulus 20MPa (short term) and 14MPa (long term), Poisson's Ratio of 0.3.

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

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
Stiff alluvial clays	450kPa[1]	150kPa[1]	45kPa[2]	15kPa[2]	20MPa (14MPa)	0.3
Very stiff alluvial clays	900kPa[1]	300kPa[1]	54kPa[2]	18kPa[2]	30MPa (21MPa)	0.3
Hard alluvial clays	1,500kPa[1]	500kPa[1]	60kPa[2]	20kPa[2]	40MPa (28MPa)	0.3
Bedrock	2,100kPa	700kPa	210kPa (compression) 105kPa (tension)	70kPa (compression) 35kPa (tension)	60MPa (42MPa)	0.3

<u>**Notes**</u>

1. Assuming a pile depth of at least 3m and an embedment of at 4 pile diameters into the applicable clay foundation material. To overcome uplift of the piles from the swelling clays the piles should either be a minimum length of 6m (i.e. twice the design suction depth) or de-bonded/permanently sleeved to 3m depth
2. Shaft Adhesion values in tension and compression are only applicable to bored piles.

For high level footings founded in the Level 1 engineered fill or alluvial clays, predicted elastic settlements for the above allowable bearing pressures would be 10mm to 15mm although the amount of settlement is a function of footing size. Once the footing sizes have been determined then further advice will need to be sought from JK Geotechnics to confirm the predicted settlements. However, we note that the predicted reactive soil movements would be the governing design constraint.

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

Maximum settlements for footings founded in bedrock would be equivalent to 1% of the pile diameter.

The above allowable bearing pressures must be confirmed by inspection of a representative number of footings by a geotechnical engineer. All high level footings in the engineered fill or alluvial clays should be excavated, inspected and poured with minimal delay. All footings should be free from all loose or softened materials prior to pouring. If water ponds in the base of the footings they should be pumped dry and then over-excavated to remove any water softened materials.

If pile footings are adopted then bored piles may 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 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 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 together with cleanliness, softening 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 and they must not be designed as founding on rock. If either of these options 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.

4.5 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 C_e – “Shallow soil site”

4.6 Floor Slabs

On-grade floor slabs are considered suitable provided the earthworks are carried out in accordance with the advice provided in Section 4.2 above. The subgrade below the floor slabs could undergo characteristic surface movements associated with a Class ‘H1-D’ site for a ‘natural’, profile, with movements increasing to at least ‘H2-D’ where reactive site won clay fill is used to raise site surface levels or reduce to ‘M-D’ if imported non-reactive granular fill (minimum 0.4m thickness) is used to raise site surface levels. These movements between the slab and structure must be accounted for in the design. If these movements cannot be tolerated, the floor slabs should be designed as suspended from piled footings with void formers.

Where preferred, on-grade slabs should be provided with joints capable of resisting shear forces but not bending moments by providing dowels or keys. In addition, close to the interface between any suspended and on-grade sections of floor slabs, we recommend that additional joints are provided.

Where floor slabs are designed as suspended then void formers must be provided below the slab and beams between the piles so that the swell pressures from the reactive clays are not transferred to the slab. The void formers must be at least 50mm thick.

For floor slabs suspended over areas of fill subgrade, the subgrade preparation would comprise the removal of any existing paved surfaces, topsoil and/or any soil containing organics, completion of any excavations to achieve design surface levels and the nominal tracking of ‘formwork fill’ to the required subgrade level and the placement of a void former as described above. Alternatively, a sacrificial formwork (such as ‘Bondek’ or similar) could be used, which we understand is being considered in some instances for this site.

Pile and raft slab design parameters and advice is presented in Section 4.4 above.

4.7 Pavement Design and Construction

The advice provided below assumes that the subgrade is prepared and any engineered fill placed in accordance with the recommendations given in Section 4.2 above.

4.7.1 Pavement Design Considerations

Based on the variable subgrade conditions, the results of the laboratory CBR tests and with regard to the elevated in-situ moisture contents of the alluvial clays, we recommend that the design of the proposed flexible pavements be based on a reduced CBR value of 3%.

Improvement of poor subgrade areas represented by poorly compacted existing fill may comprise placement of a select subgrade layer of at least 0.3m thickness of material (either select granular or lime stabilised clay with a soaked CBR value of at least 10%). This layer should then be utilised in the mechanistic design of the pavements, such as with the 'Circly' software program. If rigid pavements are proposed, the provision of a 150mm thick lean mix concrete sub-base would give an effective design CBR value of 40%.

The flexible pavements will need to be provided with cross falls to maintain drainage.

Surface and sub-soil drains should be provided along the perimeter of pavements, with subsoil drain inverts not less than 0.2m below subgrade level. The drainage trenches should be excavated with a longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The pavement subgrade should be graded to promote water flow towards the sub-soil drains.

4.7.2 Pavement Thickness Design

The flexible pavement thickness design outlined below has been based on a mechanistic analysis using the program 'Circly' Version 6 which is in accordance with 'Guide to Pavement Technology' Part 2: Pavement Structural Design (AUSTROADS Sydney 2017).

For proposed access roads and car parking areas we recommend their design be based on a CBR value of 3% for the alluvial clay subgrade. The flexible pavement thickness design provided in the attached Table 1 is based on an assumed design traffic loading of $1x10^5$ ESA's and must be confirmed once the actual traffic loading is provided.

The recommended pavement thickness design and material quality specifications are provided in the attached Table 1.

4.7.3 Pavement Materials

In addition to the information provided on the attached Table 1 we note the following compaction requirements and additional information regarding selection of pavement construction materials:

- The DGB20 should be compacted in a single layer using a large smooth drum roller to at least 98% of Modified Maximum Dry Density (MMDD).
- The sub-base should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 95% of MMDD. Alternatively, LMC could be used which would not require compaction but fatigue damage would need to be taken into account by the designer.
- Recycled materials may be used as sub-base provided they conform to the TfNSW QA specification 3051 (2018) requirements. However, recycled materials can be self-cementing, which can then cause reflective cracking through the pavement surface, which would then require crack sealing. While this may be an aesthetic issue, it would not necessarily cause significant reduction in the pavement life provided the cracks are appropriately sealed.

For the pavement construction materials, adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement so as to reduce the potential for material breakdown during compaction.

Density tests should be carried out on the granular pavements material to confirm the above specifications are achieved. The frequency of density testing should be as per the requirements outlined in AS3798 and Level 2 testing is recommended. The Geotechnical Testing Authority should be directly engaged by the client or their representative and not by the earthworks contractor.

4.8 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 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).

4.9 Site Stability

Based on the relative flat and gently sloping nature of the topography within and neighbouring the site it is considered 'Barely Credible' that natural slope instability would occur and on this basis levels of risk to property and life under existing conditions and following the development are assessed to be at 'Acceptable' levels.

Provided the design and construction of the proposed alterations and additions are carried out in accordance with the advice presented in this report then we consider that for the assessed probabilities associated with the assessed likelihood of instability and assuming typical temporal, vulnerability, evacuation and spatial factors for this type of site, risks to life and property associated with potential instability of temporary or permanent batters slopes and existing and proposed retaining walls will also be at an 'Acceptable' level.

The terminology and criteria adopted in the above assessment is in accordance with the methodologies and criteria adopted in the Australian Geomechanics Society (2007c) 'Practice Note Guidelines for Landslide Risk Management' (Reference 1).

4.10 Further Geotechnical Input

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

- Inspection of additional footings exposing existing footings.
- Witnessing proof rolling.
- Density testing of engineered fill and pavement materials.
- Inspection of a representative number of footings.

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.

Reference 1: *Australian Geomechanics Society (2007c) 'Practice Note Guidelines for Landslide Risk Management', Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.*

East West Enviroag Pty Limited
82 Plain Street Tamworth NSW 2340
ABN 82 125 442 382 124 ph 02 6762 1733 fax 02 6765 9109

eastwest
geo ag enviro

CALIFORNIA BEARING RATIO REPORT SHEET

CLIENT:	Regional Geotechnical Solutions		
CLIENT ADDRESS:	Unit 14, 25-27 Hurley Drive, COFFS HARBOUR NSW 2450	**REPORT NO:**	EW225312-1
PROJECT:	Hospital Redevelopment	**PROJECT NO:**	EW225312
SITE LOCATION:	Gunnedah - RGS 35091UR		
DATE OF TESTING:	4/07/2022	**DATE OF REPORT:**	6/07/2022
TECHNICIAN:	S.M.	**DATE SAMPLED:**	27/06/2022

SAMPLE DATA

Test Location	BH 1
Depth	0.6 - 1.2m
Material Description	Sandy CLAY Trace Gravel

Sample Number	1
Sampling Clause	Client, results apply to samples as received.

LABORATORY COMPACTION DATA

Australian Standards
- ☑ AS1289.5.1.1 - Standard Compction
- ☐ AS1289.5.2.1 - Modified Compaction
- ☑ AS1289.2.1.1 - Moisture Content - Oven

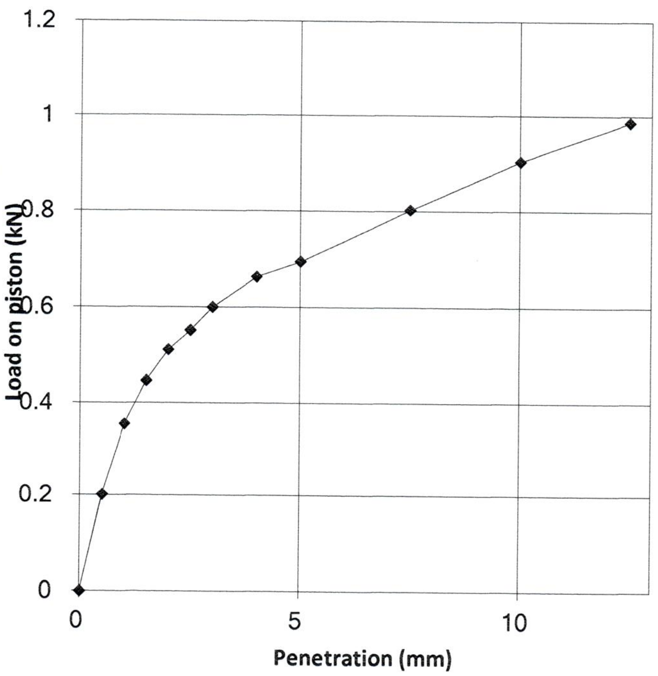
RTA Methods
- ☐ TfNSW T111 - Standard Compaction
- ☐ TfNSW T112 - Modified Compaction
- ☐ TfNSW T120 - Moisture Content - Oven

Maximum Dry Density	t/m³	1.69
Optimum Moisture Content	%	19.3
Curing time	hrs	48
Liquid limit determination		Technician
Field Moisture Content	%	19.8

CALIFORNIA BEARING RATIO TEST RESULTS
- ☑ AS1289.6.1.1
- ☐ TfNSW T117

Dry Density (At Compaction)	t/m³	1.69
Density Ratio (At Compaction)	%	100
Moisture Content (At Compaction)	%	19.3
Moisture Ratio (At Compaction)	%	100
Days Soaked		4
Surcharge Weight	kg	4.5
Swell (After soaking)	%	0.1
Dry Density (After soaking)	t/m³	1.69
Moisture Content (Top 30mm)	%	22.3
Moisture Content (Remaining Sample	%	20.7
CBR Value @ 2.5mm Penetration	%	**4**
Percentage of sample retained on 19mm Sieve		0

Oversize retained on the 19mm sieve was not included in CBR test samples



Sampled By: Client

Tested By: EW - Tamworth

Comments:

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

This report remains the property of East West Enviroag Pty Ltd until paid in full
Accredited for compliance with ISO/IEC 17025 - Testing
This document shall not be produced, except in full.
NATA Accredited Laboratory Number. 12360

Signed:
Approved Signatory

Signatory Name: S Mitchell
Document ID: REP-119
Issue No: 4
Date of Issue: 11-Mar-20

East West Enviroag Pty Limited
82 Plain Street Tamworth NSW 2340
ABN 82 125 442 382 124 ph 02 6762 1733 fax 02 6765 9109

eastwest
geo ag enviro

CALIFORNIA BEARING RATIO REPORT SHEET

CLIENT:	Regional Geotechnical Solutions		
CLIENT ADDRESS:	Unit 14, 25-27 Hurley Drive, COFFS HARBOUR NSW 2450	**REPORT NO:**	EW225312-2
PROJECT:	Hospital Redevelopment	**PROJECT NO:**	EW225312
SITE LOCATION:	Gunnedah - RGS 35091UR		
DATE OF TESTING:	4/07/2022	**DATE OF REPORT:**	6/07/2022
TECHNICIAN:	S.M.	**DATE SAMPLED:**	27/06/2022

SAMPLE DATA

Test Location	BH 5
Depth	0.8 - 1.4m
Material Description	CLAY

Sample Number	2
Sampling Clause	Client, results apply to sample as received.

LABORATORY COMPACTION DATA

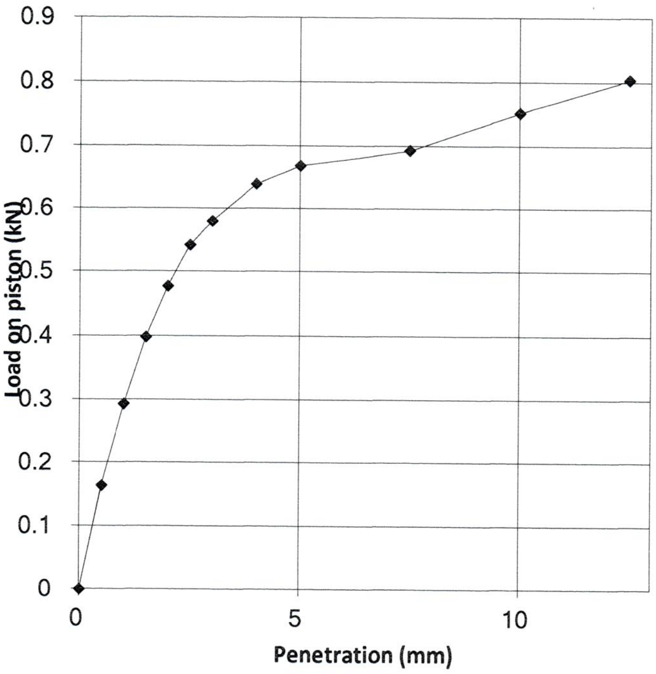
Australian Standards
- ☑ AS1289.5.1.1 - Standard Compction
- ☐ AS1289.5.2.1 - Modified Compaction
- ☑ AS1289.2.1.1 - Moisture Content - Oven

RTA Methods
- ☐ TfNSW T111 - Standard Compaction
- ☐ TfNSW T112 - Modified Compaction
- ☐ TfNSW T120 - Moisture Content - Oven

Maximum Dry Density	t/m³	1.66
Optimum Moisture Content	%	20.3
Curing time	hrs	48
Liquid limit determination		Technician
Field Moisture Content	%	21.5

CALIFORNIA BEARING RATIO TEST RESULTS
- ☑ AS1289.6.1.1
- ☐ TfNSW T117

Dry Density (At Compaction)	t/m³	1.66
Density Ratio (At Compaction)	%	100
Moisture Content (At Compaction)	%	20.3
Moisture Ratio (At Compaction)	%	100
Days Soaked		4
Surcharge Weight	kg	4.5
Swell (After soaking)	%	-0.1
Dry Density (After soaking)	t/m³	1.66
Moisture Content (Top 30mm)	%	27.9
Moisture Content (Remaining Sample	%	21.5
CBR Value @ 2.5mm Penetration	%	**4**
Percentage of sample retained on 19mm Sieve		0
Oversize retained on the 19mm sieve was not included in CBR test samples		



Sampled By: Client

Tested By: EW - Tamworth

Comments:

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

This report remains the property of East West Enviroag Pty Ltd until paid in full
Accredited for compliance with ISO/IEC 17025 - Testing
This document shall not be produced, except in full.
NATA Accredited Laboratory Number. 12360

Signed:
Approved Signatory
Signatory Name: S Mitchell
Document ID: REP-119
Issue No: 4
Date of Issue: 11-Mar-20

East West Enviroag Pty Limited
82 Plain Street Tamworth NSW 2340
eastwest ABN 82 125 442 382 124 ph 02 6762 1733 fax 02 6765 9109
geo ag enviro

SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT:	Regional Geotechnical Solutions		
CLIENT ADDRESS:	Unit 14, 25-27 Hurley Drive, COFFS HARBOUR NSW 2450	**REPORT NO:**	EW225312-3
PROJECT:	Hospital Redevelopment	**PROJECT NO:**	EW225312
SITE LOCATION:	Gunnedah - RGS 35091UR		
DATE OF TESTING:	28/06/2022	**DATE OF REPORT:**	6/07/2022
TECHNICIAN:	W.S.	**DATE SAMPLED:**	27/06/2022

Sampled By Client, results apply to sample as received.

TEST METHOD	TEST DESCRIPTION				
	SAMPLE No:		3	5	8
	SAMPLE LOCATION:		BH 1	BH 7	TP 102
	DEPTH:		1.0 - 1.45m	1.0 - 1.45m	0.6 - 0.8m
	MATERIAL DESCRIPTION:		Clay	Clay	Clay
	WASHED/UNWASHED		-	-	-
	AS1289.3.8.1 Water Type & Temperature:				
~~AS1289.3.6.1~~	% Passing Sieve Analysis	75.0mm			
		63.0mm			
		53.0mm			
		37.5mm			
		26.5mm			
		19.0mm			
		13.2mm			
SAMPLING METHOD		9.5mm			
		6.7mm			
		4.75mm			
~~AS1289.1.2.1.6.4~~		2.36mm			
~~AS1289.1.2.1.6.5.1~~		1.18mm			
~~AS1289.1.2.1.6.5.3~~		600µm			
~~AS1289.1.2.1.6.5.4~~		425µm			
		300µm			
		150µm			
		75µm			
AS1289.3.8.1	EMERSON CLASS NO:				
AS1289.2.1.1	MOISTURE CONTENT: %		17.0	27.3	23.8
		CODE			
~~AS1289.3.1.1~~	LIQUID LIMIT %				
AS1289.3.1.2	LIQUID LIMIT %	2 , 4	34	39	43
AS1289.3.2.1	PLASTIC LIMIT %	2 , 4	12	14	11
AS1289.3.3.1	PLASTICITY INDEX %	2 , 4	22	25	32
AS1289.3.4.1	LINEAR SHRINKAGE %	2 , 4	9	10	11

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

This report remains the property of East West Enviroag Pty Ltd until paid in full

Accredited for compliance with ISO/IEC 17025 - Testing

This document shall not be produced, except in full.

NATA Accredited Laboratory Number. 12360

Signed: ....
Approved Signatory

Signatory Name: S Mitchell
Document ID: REP-102
Issue No: 2
Date of Issue: 24-Apr-13

East West Enviroag Pty Limited
82 Plain Street Tamworth NSW 2340
eastwest ABN 82 125 442 382 124 ph 02 6762 1733 fax 02 6765 9109
geo ag enviro

SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT:	Regional Geotechnical Solutions		
CLIENT ADDRESS:	Unit 14, 25-27 Hurley Drive, COFFS HARBOUR NSW 2450	**REPORT NO:**	EW225312-4
PROJECT:	Hospital Redevelopment	**PROJECT NO:**	EW225312
SITE LOCATION:	Gunnedah - RGS 35091UR		
DATE OF TESTING:	1/08/2022	**DATE OF REPORT:**	9/08/2022
TECHNICIAN:	S.M.	**DATE SAMPLED:**	27/06/2022

Sampled By Client, results apply to sample as received.

	SAMPLE No:		4		
	SAMPLE LOCATION:		BH 5		
	DEPTH:		1.0 - 1.45m		
	MATERIAL DESCRIPTION:		Clay		
	WASHED/UNWASHED		-		
AS1289.3.8.1 Water Type & Temperature:					
TEST METHOD	TEST DESCRIPTION				
~~AS1289.3.6.1~~	% Passing Sieve Analysis	75.0mm			
		63.0mm			
		53.0mm			
		37.5mm			
		26.5mm			
		19.0mm			
		13.2mm			
SAMPLING METHOD		9.5mm			
		6.7mm			
		4.75mm			
~~AS1289.1.2.1.6.4~~		2.36mm			
~~AS1289.1.2.1.6.5.1~~		1.18mm			
~~AS1289.1.2.1.6.5.3~~		600µm			
~~AS1289.1.2.1.6.5.4~~		425µm			
		300µm			
		150µm			
		75µm			
AS1289.3.8.1	EMERSON CLASS NO:				
AS1289.2.1.1	MOISTURE CONTENT:	%	21.0		

		CODE			
~~AS1289.3.1.1~~	LIQUID LIMIT %				
AS1289.3.1.2	LIQUID LIMIT %	2 , 4	41		
AS1289.3.2.1	PLASTIC LIMIT %	2 , 4	14		
AS1289.3.3.1	PLASTICITY INDEX %	2 , 4	27		
AS1289.3.4.1	LINEAR SHRINKAGE %	2 , 4	11		

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025 - Testing

NATA Accredited Laboratory Number. 12360

Signed: ....
Approved Signatory

Signatory Name: S Mitchell
Document ID: REP-102
Issue No: 2
Date of Issue: 24-Apr-13

TABLE 1 PAVEMENT THICKNESS RECOMMENDATIONS

Traffic Frequency and Pavement Type	Asphaltic Concrete Wearing Course Thickness (mm)	Base Thickness (mm)	Subbase Thickness (mm)	Total Pavement Thickness (mm)
1 x 10^5 ESAs	30	180	200	410

NOTES

1. The Asphaltic Concrete wearing course comprises AC14 with A10E Polymer Modified Binder compacted in a single layer and in accordance with the requirements of TfNSW QA Specification R116.
2. A 7mm primer seal is to be used between the base course and the AC.
3. All Base material to be crushed rock to TfNSW QA specification 3051 (2018) DGB20.
4. All Subbase to be crushed rock to TfNSW QA specification 3051 (2018) DGS40 or DGS20.
5. The base and subbase are to be placed and compacted in accordance with the requirements of TfNSW QA Specification R71
6. The above pavement thicknesses are based on a design CBR value of 3%, and assume that good surface and subsurface drainage is provided.
7. Surface level for the AC is to be at or above design surface.
8. All layer thicknesses are minimum thicknesses. Appropriate tolerances to be allowed for by contractor.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.55m
Date: 1/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Silty clay, low plasticity, dark brown, with fine to medium grained sand and fine grained gravel, top 100mm root affected.	w>PL			SCREEN: 10.7kg 0-0.1m NO FCF; SCREEN: 10.65kg 0.1-0.6m NO FCF
					CL-CI	Silty CLAY: low to medium plasticity, brown, with fine to medium grained sand.	w>PL	VSt		ALLUVIAL
		N = 5 3,3,2	1						240	
			2			as above, but becoming red brown.				
		N = 17 5,7,10						Hd	410	
ON COMPLETION			3							
		N = 28 10,13,15	4		CL-CI	Sandy CLAY: low to medium plasticity. brown, with fine to medium grained sub-rounded gravel, fine to medium grained sand.	w<PL		500	
			5							
		N = 28 11,12,16							>600	
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. **1**

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.55m
Date: 1/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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 = 33 10,14,19			CL-CI	Sandy CLAY: low to medium plasticity. brown, with fine to medium grained sub-rounded gravel, fine to medium grained sand.	w<PL	Hd		
						8							
					N = 41 13,18,23								
						9							
						10							
					N = 41 15,19,22								
									END OF BOREHOLE AT 10.45m				
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No.
2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.1m
Date: 1/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Sandy clay, low plasticity, dark brown, fine to medium grained sand, with fine to medium grained gravel, top 100mm root affected	w>PL			SCREEN: 10.0kg 0-0.1m NO FCF SCREEN: 4.18kg 0.1-0.8m NO FCF
		N = 10 3,4,6	1		CI	Silty CLAY: medium plasticity, brown, with fine to medium grained sand.	w>PL	VSt	260	ALLUVIAL
			2							
ON COMPLETION		N = 7 3,3,4	3						300	
		N > 25 8,10, 15/60mm REFUSAL	4			as above, but with fine to medium grained gravel and layers of coarse grained gravel.	w<PL	(Hd)		NO SPT SAMPLE RECOVERY
			5							
		N = 38 17,17,21	6							NO SPT SAMPLE RECOVERY
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. 2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.1m

Date: 1/6/22 **Datum:** AHD

Plant Type: DRILL RIG **Logged/Checked by:** R.G.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 = 36 22,18,18			CI	Sandy CLAY: medium plasticity, brown, with fine to coarse grained sub-rounded gravel.	w<PL	Hd		NO SPT SAMPLE RECOVERY
						8							
					N > 32 17,22, 10/20mm REFUSAL								NO SPT SAMPLE RECOVERY
						9			END OF BOREHOLE AT 8.85m				
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **3**

1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 278.9m
Date: 1/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Sandy clay, medium plasticity, dark brown, fine to medium grained sand, with fine to medium grained gravel, top 100mm root affected.	w>PL			SCREEN: 10.07kg 0-0.1m NO FCF SCREEN: 4.75kg 0.1-0.8m NO FCF
					CI	Silty CLAY: medium plasticity, brown, with fine to medium grained sand.	w>PL	VSt		ALLUVIAL
		N = 9 2,4,5	1						210	
			2							
		N = 14 5,6,8							350	
ON COMPLETION			3							
		N = 31 9,13,18	4					Hd	450	NO SPT SAMPLE RECOVERY
			5							
		N = 27 8,13,14				as above, but with fine to medium grained sub-rounded gravel.			>600	
			6			END OF BOREHOLE AT 6.0m				
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.3m
Date: 2/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Silty sand, fine to medium grained, brown and grey, with fine to coarse grained gravel, top 100mm root affected.	M / D			SCREEN: 10.69kg 0-0.1m NO FCF SCREEN: 10kg 0.1-1.0m NO FCF
		N = 4 2,2,2	1			as above, but with clay nodules.				APPEARS POORLY COMPACTED SCREEN: 4.07kg 1.0-1.6m NO FCF
		N = 13 4,6,7	2		CI	Silty CLAY: medium plasticity, brown, with fine to medium grained sand.	w>PL	VSt	250	ALLUVIAL
ON COMPLETION			3							
		N = 23 7,10,13	4			as above, but with fine to medium grained sub-rounded gravel.	w<PL	Hd	550	
		N = 31 11,13,18	5 6 7		CL-CI	Sandy CLAY: low to medium plasticity, brown and orange brown, with fine to coarse grained sub-rounded gravel.				

JKGeotechnics

BOREHOLE LOG

Borehole No. 4

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 280.3m

Date: 2/6/22 **Datum:** AHD

Plant Type: DRILL RIG **Logged/Checked by:** R.G.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 = 41 9,18,23				Sandy CLAY: low to medium plasticity, brown and orange brown, with fine to coarse grained sub-rounded gravel.	w<PL	Hd	>600	
									END OF BOREHOLE AT 7.45m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 5

1/2

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 278.6m

Date: 2/6/22 **Datum:** AHD

Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Silty sand, fine to medium grained, brown, with fine to coarse grained gravel.	M			SCREEN: 9.87kg 0-0.1m NO FCF SCREEN: 2.02kg 0.1-0.8m NO FCF
		N = 5 1,2,3	1		CI	Sandy CLAY: medium plasticity, brown, fine to medium grained sand, with fine to medium grained sand lenses.	w>PL	St	110	ALLUVIAL
			2							
		N = 13 5,6,7	3					VSt	220	
ON COMPLETION										
		N = 29 7,13,16	4			as above, but with fine to coarse grained sub-rounded gravel.	w<PL	Hd	520	
			5							
		N = 27 10,12,15	6						>600	
			7							

JKGeotechnics

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 278.6m

Date: 2/6/22 **Datum:** AHD

Plant Type: DRILL RIG **Logged/Checked by:** R.G.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 = 29 9,13,16				Sandy CLAY: medium plasticity, brown, fine to medium grained sand, with fine to medium grained sand lenses and fine to coarse grained sub rounded gravel.	w<PL	Hd	580	
									END OF BOREHOLE AT 7.45m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. 6

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 278.1m

Date: 2/6/22 **Datum:** AHD

Plant Type: DRILL RIG **Logged/Checked by:** R.G.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
			0			FILL: Silty sand, fine to medium grained, dark brown, with fine to coarse grained gravel and roots.	M			SCREEN: 11.02kg 0-0.1m NO FCF
					CI	Silty CLAY: medium plasticity, brown, with fine to medium grained sand and fine to medium grained sand lenses.	w>PL	St		ALLUVIAL
		N = 7 2,3,4	1						150	
			2							
ON COMPLET-ION		N = 15 5,6,9						VSt	220	
			3							
		N = 27 9,12,15	4		CI	Sandy CLAY: medium plasticity, brown and orange brown, fine to medium grained sand, with fine to coarse grained sub-rounded gravel.		Hd	450	
			5							
		N = 33 12,15,18							550	
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 278.1m
Date: 2/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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 = 35 10,17,18				Sandy CLAY: medium plasticity, brown and orange brown, fine to medium grained sand, with fine to coarse grained sub-rounded gravel.	w>PL	Hd	550	
									END OF BOREHOLE AT 7.45m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **7**

1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 279.7m
Date: 3/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.S./P.R.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 150mm.t				100mm TOP COVER
			-	FILL: Sandy gravel, fine to medium grained, grey, fine to coarse grained sand.	M			SCREEN: 2.77kg 0.15-0.3m NO FCF
				FILL: Clayey sand, fine to coarse grained, brown, with fine to medium grained gravel.				SCREEN: 9.50kg 0.3-0.7m NO FCF
			CI	Sandy CLAY: medium plasticity, brown, fine to medium grained sand, with fine to medium grained sand lenses	w>PL	St		ALLUVIAL
	N = 4 2,2,2	1					110	
		2						
	N = 14 4,6,8					VSt	280	
		3						
ON COMPLETION				as above, but with fine to coarse grained sub-rounded gravel.				
	N = 27 7,11,16	4				Hd	500	
		5						
	N = 33 11,14,19						550	
		6						
		7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
7
2/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 279.7m
Date: 3/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.S./P.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					N = 35 9,17,18			CI	Sandy CLAY: medium plasticity, brown, fine to medium grained sand, with fine to medium grained sand lenses and fine to coarse grained sub-rounded gravel.	w>PL	Hd		NO SPT SAMPLE RECOVERY
									END OF BOREHOLE AT 7.45m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JKGeotechnics

BOREHOLE LOG

Borehole No. **8**

1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 277.8m
Date: 3/6/22 **Datum:** AHD
Plant Type: DRILL RIG **Logged/Checked by:** R.G.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			FILL: Sandy clay, low plasticity, dark brown, with fine to coarse grained gravel, top 100mm root affected.	w<PL			SCREEN: 10.85kg 0-0.1m NO FCF SCREEN: 8.63kg 0.1-0.9m NO FCF
		N = 14 9,7,7	1		CI	Sandy CLAY: medium plasticity, brown, fine to medium grained sand.	w<PL	VSt	300	ALLUVIAL
			2							
		N = 32 10,14,18	3			Sandy CLAY: medium plasticity, brown and orange brown, fine to medium grained sand, with fine to coarse grained sub-rounded gravel.		Hd	>600	
		N = SPT 30/140mm REFUSAL	4		-	Extremely Weathered basalt: clayey GRAVEL, fine to coarse grained, grey and brown, iron staining. END OF BOREHOLE AT 4.15m	XW	(VD)		WERRIE BASALT 'TC' BIT REFUSAL
			5							
			6							
			7							

JKGeotechnics

TEST PIT LOG

Test Pit No.
TP1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 280.25m
Date: 1/6/22 **Datum:** AHD
Plant Type: - **Logged/Checked by:** R.G.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			FILL: Silty clay topsoil, medium plasticity, dark brown, trace of fine to medium grained gravel.	w<PL		120	SCREEN: 10.10kg 0-0.1m NO FCF
						FILL: Gravelly clay, medium plasticity, dark brown, with fine to coarse grained, sub-angular and sub-rounded gravel, concrete clasts and 0.4m				SCREEN: 10.20kg 0.1-0.2m NO FCF
										SCREEN: 10.91kg 0.2-0.6m NO FCF
			1			END OF TEST PIT AT 1.0m				SCREEN: 10.71kg 0.6-1.0m NO FCF
			2							
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

TEST PIT LOG

Test Pit No. **TP2** 1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 280.7m
Date: 1/6/22 **Datum:** AHD
Plant Type: - **Logged/Checked by:** R.G.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			FILL: Gravelly clay, dark brown, with fine to coarse grained, sub-angular and sub-rounded gravel.	w≈PL			SCREEN: 11.71kg 0-0.1m NO FCF
									END OF TEST PIT AT 0.3m				SCREEN:10.05kg 0.1-0.3m FCF1
													WATER MAIN HIT
						1							
						2							
						3							
						4							
						5							
						6							
						7							

JKGeotechnics

TEST PIT LOG

Test Pit No.
TP3

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 279.35m
Date: 1/6/22 **Datum:** AHD
Plant Type: - **Logged/Checked by:** R.G.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			FILL: Gravelly clay topsoil, medium plasticity, red brown, with fine to medium grained sub-angular gravel.	w<PL			SCREEN: 11.70kg 0-0.1m NO FCF
						FILL: Sand, fine to medium grained, dark grey.	D-M		160	SCREEN 11.11kg 0.1-0.2m NO FCF
						FILL: Gravelly clay, low to medium plasticity, dark brown, with fine to coarse grained, sub-angular and sub-rounded gravel, fine to coarse grained sand.	w<PL		120	SCREEN:10.70kg 0.2-1.0m NO FCF
			1			END OF TEST PIT AT 1.0m				
			2							
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

TEST PIT LOG

Test Pit No.
TP4
1/1

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 277.45m

Date: 1/6/22 **Datum:** AHD

Plant Type: - **Logged/Checked by:** R.G.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			FILL: Sandy clay topsoil, medium plasticity, dark brown, with fine grained sand.	w<PL			SCREEN: 10.41kg 0-0.1m NO FCF
						FILL: Gravelly clay, medium plasticity, dark brown, with fine to coarse grained sub-angular and sub-rounded gravel, trace of fine to coarse grained sand.				SCREEN: 10.10kg 0.1-0.7m NO FCF
					CI	Sandy CLAY: medium plasticity, light red brown, with fine to coarse grained sand, trace of fine to medium grained, sub-rounded gravel.	w<PL	(St)		ALLUVIAL
			1			END OF TEST PIT AT 1.0m				
			2							
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

TEST PIT LOG

Test Pit No. **TP5**
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 278.3m
Date: 1/6/22 **Datum:** AHD
Plant Type: - **Logged/Checked by:** R.G.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		-	FILL: Gravelly clay topsoil, medium plasticity, dark brown, with fine to coarse grained, sub-angular and sub-rounded gravel.	w<PL			SCREEN: 10.19kg 0-0.2m NO FCF
								-	ASPHALT: 20mm.t				
									FILL: Clayey gravel, fine to medium grained, dark brown, sub-rounded, trace of medium plasticity clay.	M			SCREEN: 11.03kg 0.25-0.5m NO FCF
									FILL: Gravelly clay, medium plasticity, dark brown, with fine to coarse grained, sub-angular gravel, trace of fine to coarse grained sand.	w<PL			SCREEN:10.01kg 0.5-0.9m NO FCF
						1			END OF TEST PIT AT 0.9m				
						2							
						3							
						4							
						5							
						6							
						7							

JKGeotechnics

TEST PIT LOG

Test Pit No.
TP6
1/1

Client: HEALTH INFRASTRUCTURE

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: MARQUIS STREET, GUNNEDAH, NSW

Job No.: 35091UR **Method:** 5T EXCAVATOR **R.L. Surface:** ≈ 279.1m

Date: 1/6/22 **Datum:** AHD

Plant Type: - **Logged/Checked by:** R.G.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			FILL: Gravelly clay topsoil, medium plasticity, brown, with fine to coarse grained, sub-rounded gravel.	w<PL			SCREEN: 10.76kg 0-0.1m NO FCF
									FILL: Gravelly clay, medium plasticity, dark grey, with fine to medium grained slag gravel, fine to coarse grained, sub-angular gravel.	w<PL		180	SCREEN: 10.57kg 0.1-0.3m NO FCF
								CI	Sandy CLAY: medium plasticity, light red brown, with fine to coarse grained sand, trace of fine to medium grained, sub-rounded gravel.	w<PL	St	120	SCREEN: 10.45kg 0.3-0.8m NO FCF
						1			END OF TEST PIT AT 1.0m				ALLUVIAL
						2							
						3							
						4							
						5							
						6							
						7							

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	GUNNEDAH HOSPITAL, MARQUIS STREET, GUNNEDAH, NSW
Report No: 35091UR	Figure No: 1

JKGeotechnics

PLOT DATE: 22/06/2022 9:31:05 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35091\CAD\35091UR.DWG

LEGEND

●	BOREHOLE
▬	TEST PIT
(blue line)	APPROXIMATE OUTLINE OF PROPOSED GROUND FLOOR LEVEL
(red line)	APPROXIMATE OUTLINE OF PROPOSED PAVEMENT AREAS

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 10 20 30 40 50

SCALE 1:1000 @A3 METRES

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

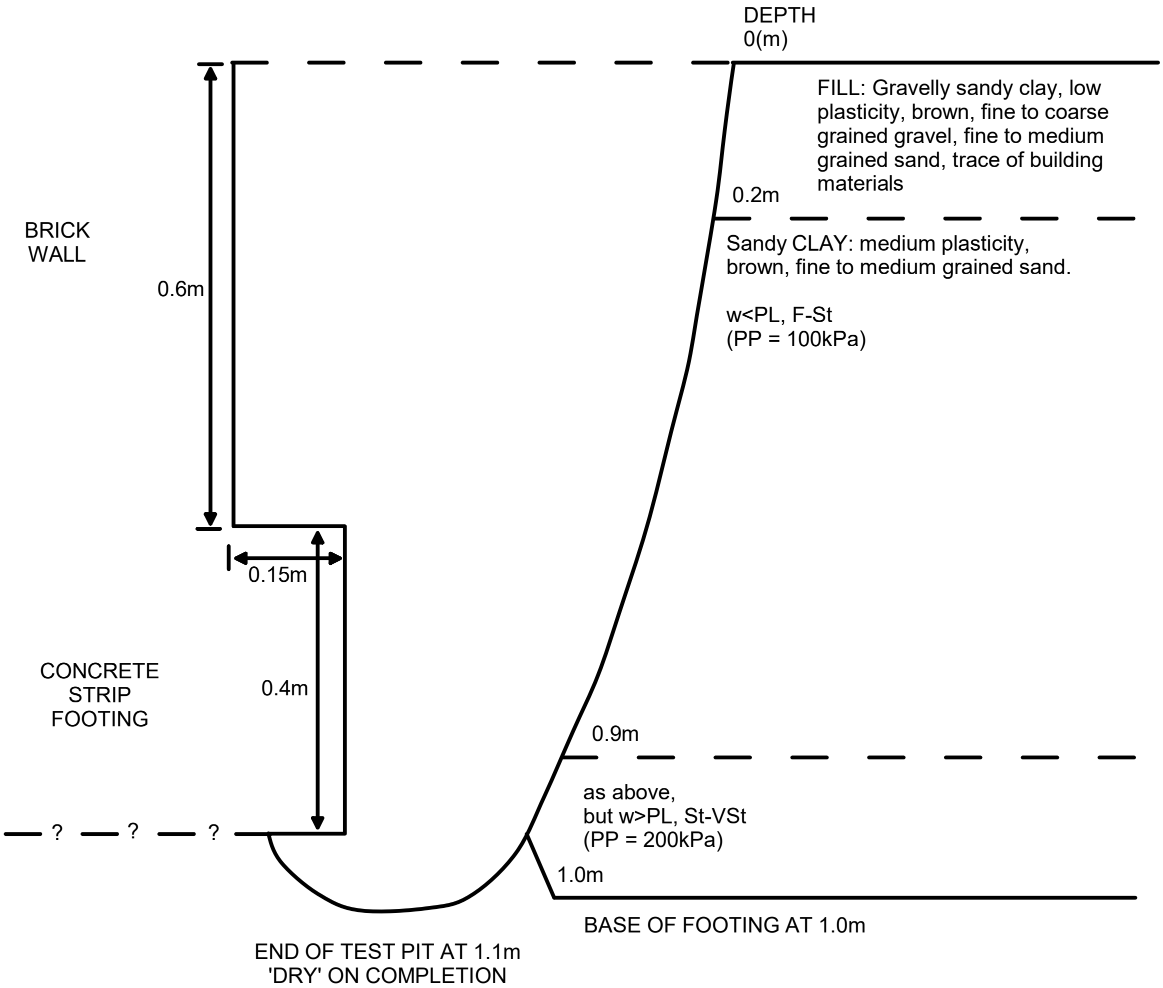
Title: **TEST LOCATION PLAN**

Location: GUNNEDAH HOSPITAL, MARQUIS STREET, GUNNEDAH, NSW

Report No: 35091UR

Figure No: 2

JKGeotechnics

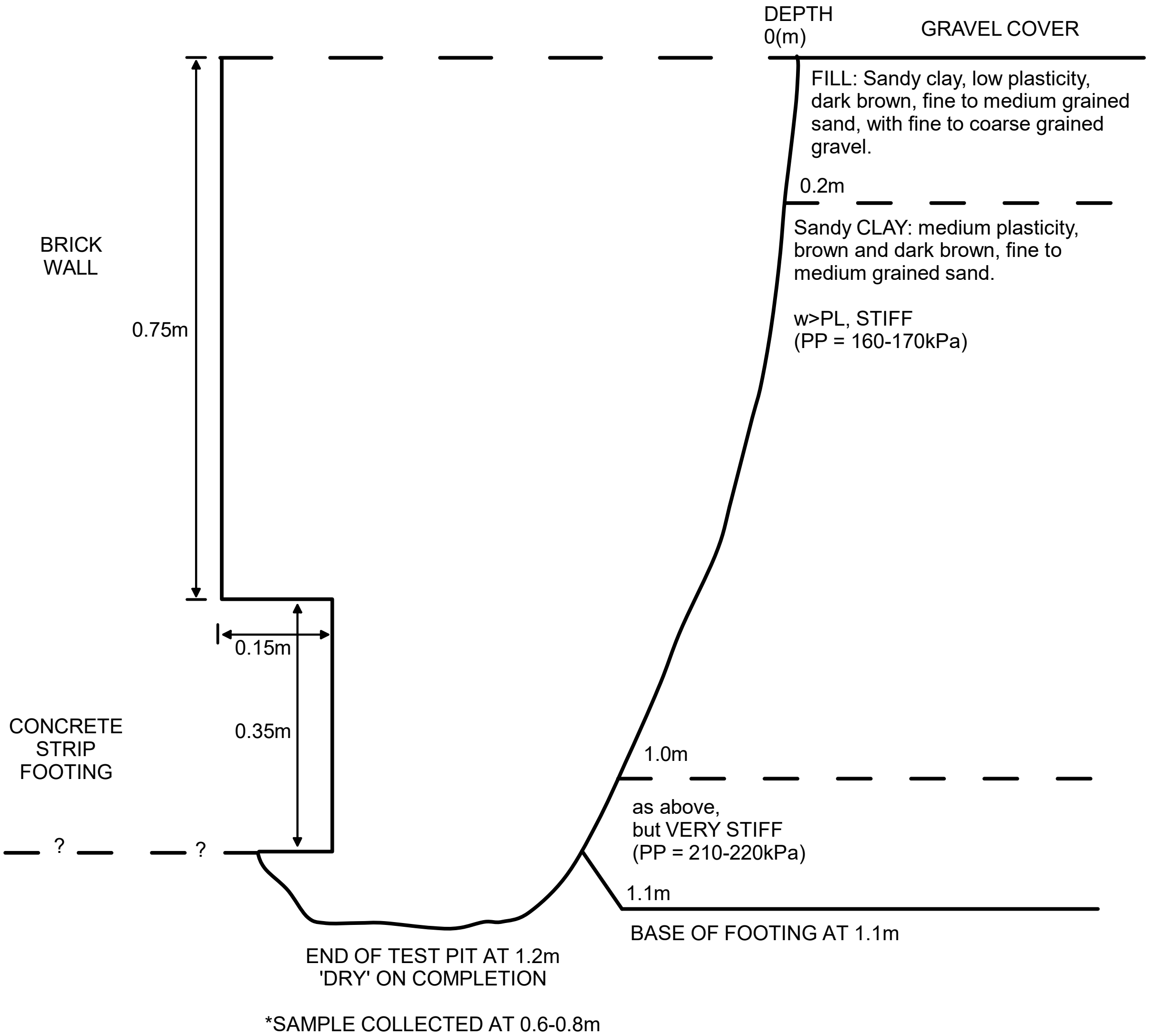


TEST PIT 101
CROSS SECTIONAL SKETCH
LOOKING SOUTH-EAST

0 0.5
Scale 1:10 @A4 Metres

JKGeotechnics

Report No. 35091UR Figure No. 3

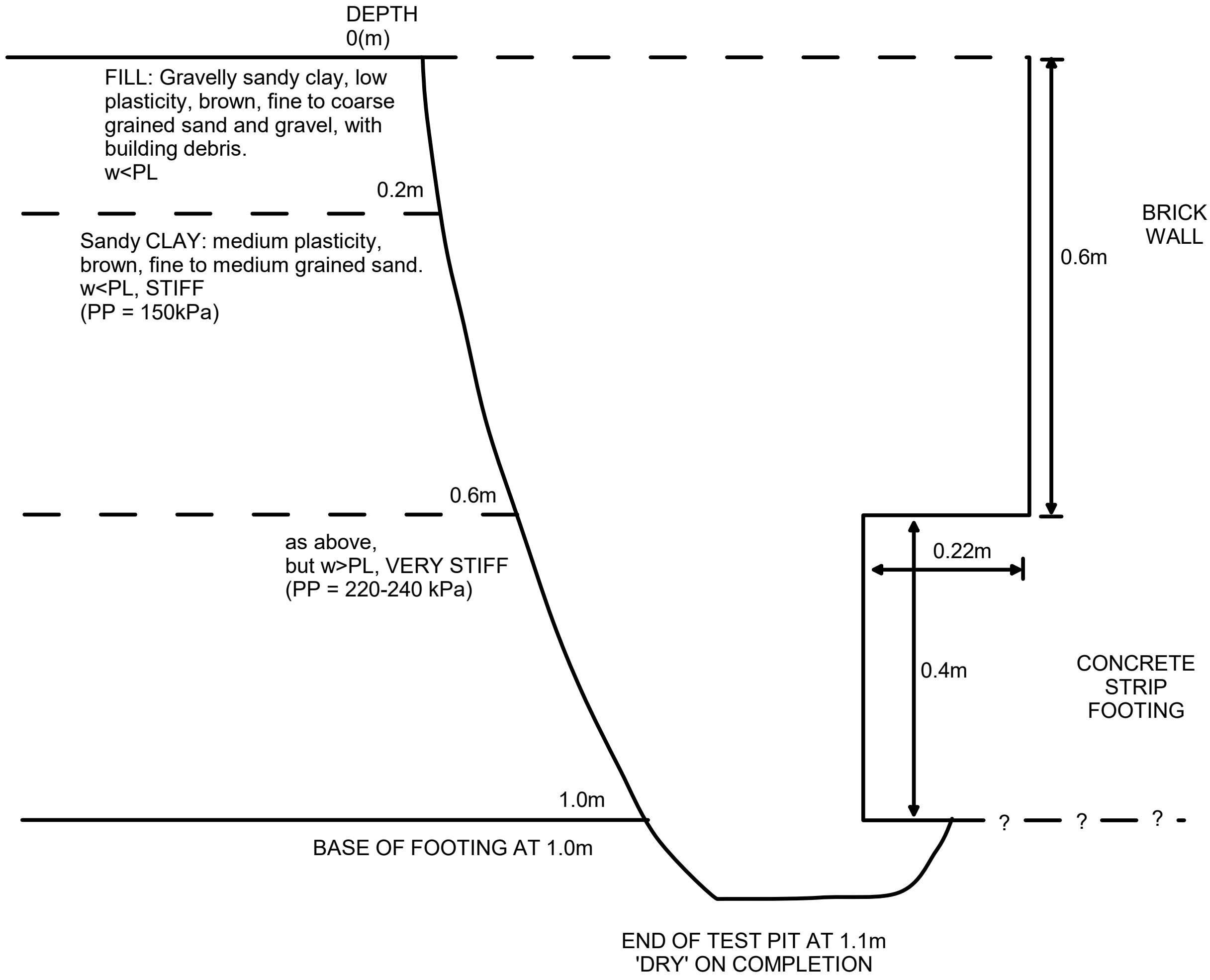


TEST PIT 102
CROSS SECTIONAL SKETCH
LOOKING SOUTH-EAST

0 0.5
Scale 1:10 @A4 Metres

JKGeotechnics

Report No. 35091UR Figure No. 4

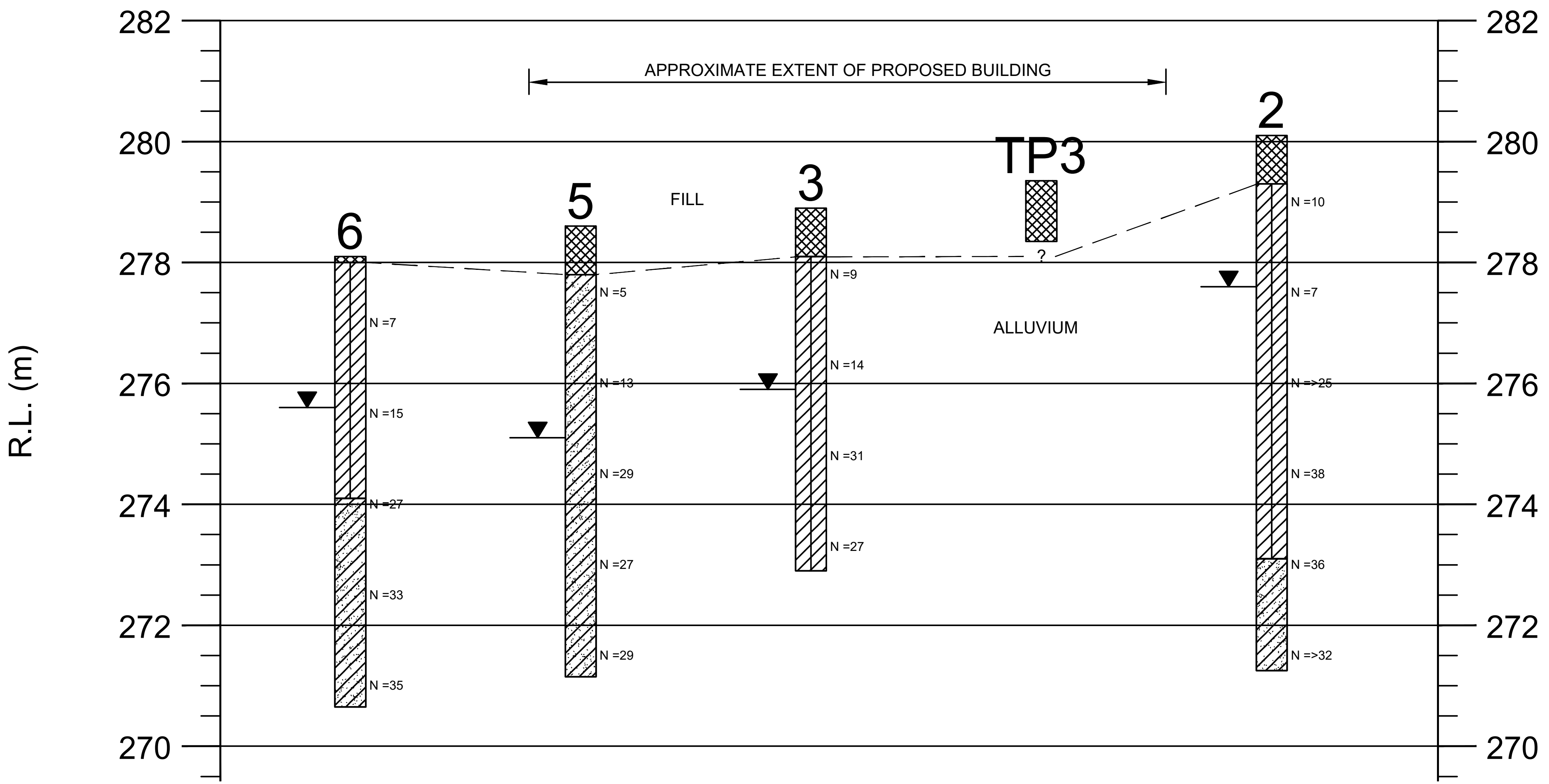


TEST PIT 103
CROSS SECTIONAL SKETCH
LOOKING SOUTH-EAST

0 0.5
Scale 1:10 @A4 Metres

JKGeotechnics

Report No. 35091UR Figure No. 5



Fill

Silty Clay

Sandy Clay

Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

0 1 2 3 4 5

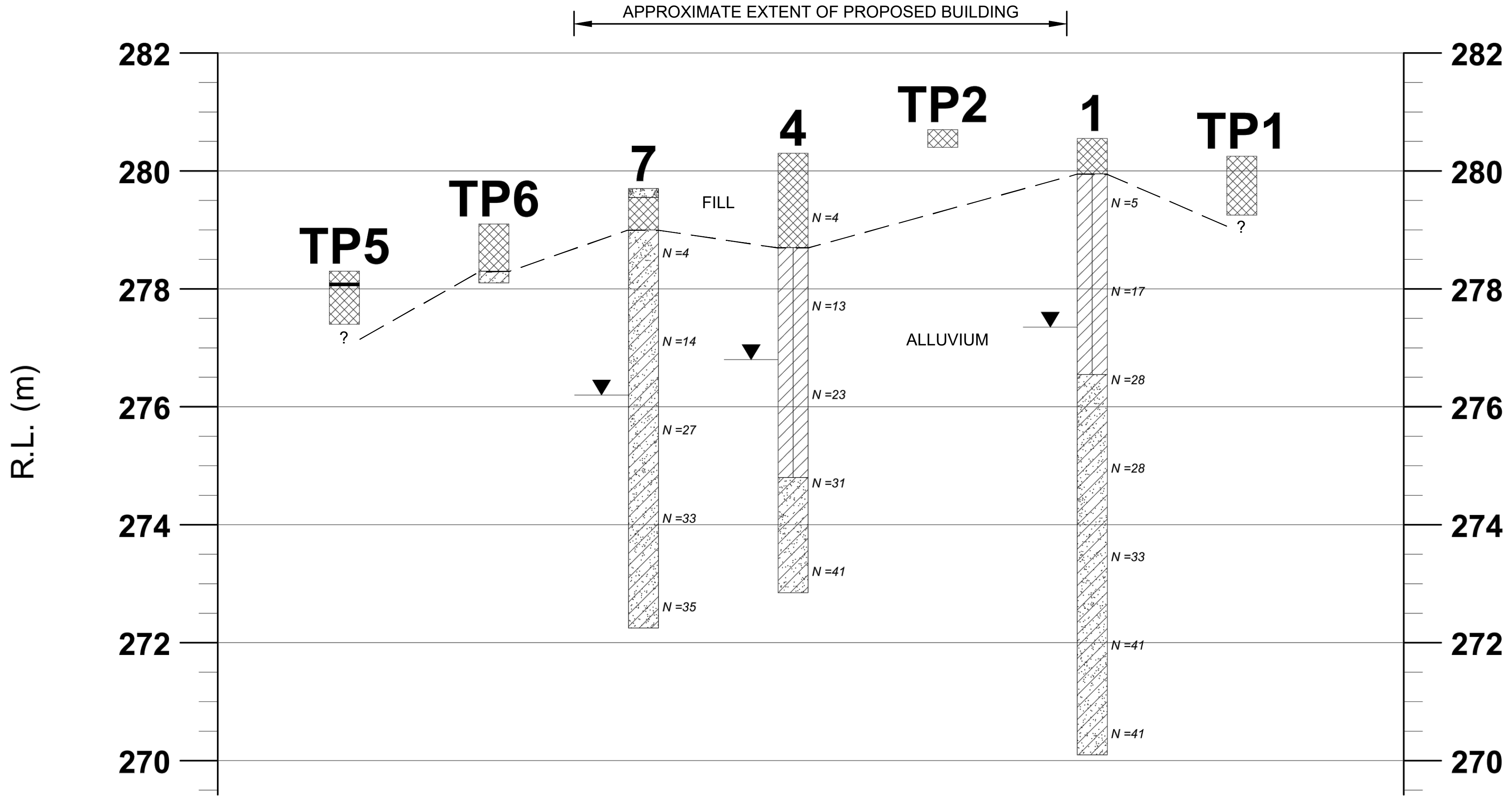
SCALE 1:100 @A4 METRES

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

Title:	**GRAPHICAL BOREHOLE AND TEST PIT SUMMARY**
Location:	GUNNEDAH HOSPITAL, MARQUIS STREET, GUNNEDAH, NSW
Report No: 35091UR	Figure No: 6

JKGeotechnics

PLOT DATE: 12/08/2022 12:48:09 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\35000'S\35091UR GUNNEDAH\CAD\35091UR.DWG



Fill

Asphaltic/ Bituminous Paving

Sandy Clay

Concrete

Silty Clay

Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

0 1 2 3 4 5

SCALE 1:100 @A4 METRES

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

Title:	**GRAPHICAL BOREHOLE AND TEST PIT SUMMARY**
Location:	GUNNEDAH HOSPITAL, MARQUIS STREET, GUNNEDAH, NSW
Report No:	35091UR
Figure No:	7

JKGeotechnics

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:

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

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

NOTES:

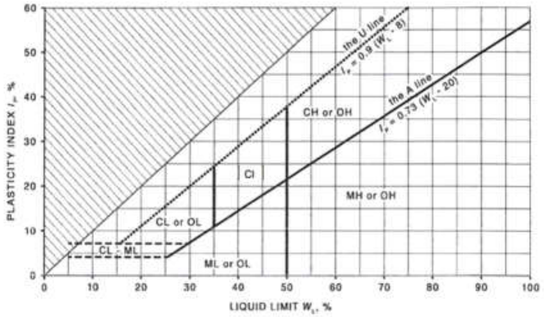
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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