REPORT TO

GARDNER WETHERILL & ASSOCIATES PTY LTD

ON

DETAILED GEOTECHNICAL INVESTIGATION

FOR

PROPOSED POLICE STATION

AT

51 BROMIDE STREET, BROKEN HILL, NSW

Date: 30 June 2020

Ref: 32665A2rpt

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Report Explanation Notes

1 INTRODUCTION

This report presents the results of a detailed geotechnical investigation for the proposed police station at 51 Bromide Street, Broken Hill, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Lindsay Henry of Gardner Wetherill & Associates Pty Ltd (GWA) by email on 13 January 2020. The commission was on the basis of our fee proposal, Ref. P50577Agwa dated 29 October 2019.

In 2019, JK Geotechnics (JKG) completed a limited scope 'Due Diligence' Geotechnical Investigation at the site for BGIS Pty Ltd; report Ref. 32665AGrpt dated 21 October 2019 [JKG 2019].

We have since been provided with the following information:

- 25% Concept Design drawings prepared by GWA (titled 'Site Plan', 'Ground Floor Plan' & 'First Floor Plan'), supplied by email on 9 April 2020;
- Survey plan prepared by Sydney Surveyors (Ref. 16781/1A dated 28 January 2020). The survey datum is the Australian Height Datum (AHD).

Based on the supplied information, we understand that a one and two storey police station, with an internal courtyard, is proposed. In a plan provided by email by GWA on 14 May 2020, the finished floor level for the proposed ground floor slab was set at RL307.02m. The proposed ground floor slab will therefore require cut and fill earthworks to a maximum depth/height of about 0.5m. Structural loads typical of this type of development have been assumed. An on-grade car park, with crossovers off Blende and Beryl Streets, are proposed on the north-eastern side of the new building.

The purpose of the investigation was to assess the subsurface conditions at eight borehole locations and, based on the information obtained, to present our updated comments and recommendations on earthworks, retaining walls, site classification to AS2870-2011, building footings, soil aggression, earthquake design parameters, and the car park pavement.

This report supersedes JKG 2019. We have not referred to the shallow borehole information or Dynamic Cone Penetrometer (DCP) test results from JKG 2019 in the preparation of the current report.

This geotechnical investigation was carried out in conjunction with a Detailed Site Investigation (Stage 2) by our environmental consulting division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref. E32665PHrpt2 dated 12 March 2020.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 4 February 2020 and comprised the drilling and testing of eight boreholes (BH101 to BH108), at the locations shown on the attached Figure 2, to depths between 1.95m and 6.0m below existing grade. The boreholes were auger drilled using a LandCruiser mounted Gemco HS7 drill rig, which is equipped for site investigation purposes.

Prior to the commencement of the fieldwork, a specialist sub-contractor reviewed available 'Dial Before You Dig' information and electro-magnetically scanned the borehole locations for buried services.

The borehole locations were set out by tape measurements from existing surface features. The surface RL's indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plan, and are therefore approximate. The survey datum is AHD. The survey plan forms the basis of Figure 2.

The relative compaction/strength of the soil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on cohesive soils recovered in the SPT split-spoon sampler, and by tactile examination. The strength of the underlying bedrock was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of recovered auger cuttings and correlations with subsequent laboratory moisture content test results. Groundwater observations were also made in the boreholes. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineer (Arthur Kourtesis) was present full-time during the fieldwork to set out the borehole locations, direct the electro-magnetic scanning, nominate testing and sampling, and prepare the attached borehole logs. The Report Explanation Notes define the logging terms and symbols used.

Selected soil and rock cutting samples were returned to a NATA accredited laboratory, Soil Test Services Pty Ltd (STS), for moisture content, Atterberg Limits, linear shrinkage, Standard compaction and four day soaked CBR testing. The results are summarised in the attached STS Tables A & B.

Selected soil samples were returned to a second NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH, sulfate, chloride and resistivity testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis 236883'.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located within flat topography, and is located in the south-western half of the block bound by Bromide Street to the north-east, Blende Street to the south-east, Beryl Street to the north-west, and Kaolin Street to the south-west. The 'Kintore Headframe' Tourist Park occupies the north-eastern half of the block.

At the time of the fieldwork, the site was vacant and generally covered by igneous gravel with remnants of a former asphaltic concrete (AC) pavement in isolated locations. There was little to no vegetation, although small trees were present near the site boundaries. A train track 'loop' (part of the adjacent tourist park) ran adjacent to the site boundaries.

The 'Kintore Headframe' Tourist Park contained the historic mining headframe, which was set back approximately 15m from the north-eastern site boundary. An AC surfaced car park and driveway were located between the north-eastern site boundary and the headframe. The AC surface appeared to be in good condition based on a cursory inspection.

On the opposite sides of Beryl Street and Blende Street, were single storey structures, mostly houses. On the opposite side of Kaolin Street was a large open area, which was a part of Broken Hill Public School.

3.2 Subsurface Conditions

The 1:100,000 series Broken Hill Stratigraphy map (Geological Survey of NSW, Edition 1, 1989) indicates the site to be underlain by the 'Globe-Vauxhall Schist Zone' within the Broken Hill Group of the Willyama Supergroup. Generally, the boreholes encountered fill, overlying residual soils, then schist (metamorphic) bedrock at depth. Reference should be made to the attached borehole logs for specific details at each location. Graphical borehole summaries are presented as Figures 3 to 5. A summary of the encountered subsurface conditions is provided below:

Fill

Fill, predominantly comprising sandy clayey silt, and to a lesser extent clayey sand, was encountered in all boreholes to depths between 0.1m (BH104) and 1.2m (BH101 & BH107). Inclusions of gravel, cobbles and AC fragments were found in the fill. Based on the SPT results and limited hand penetrometer testing, the deeper fill in BH101, BH102, BH107 and BH108 was assessed to be either moderately or well compacted.

Residual Soils

Residual soils comprising silty sandy clay, silty clays and sandy clayey silts were encountered below the fill in all boreholes. The residual clays were of medium plasticity and of very stiff to hard strength. The residual silts were of low plasticity and of very stiff to hard strength. Inclusions of gravels and cobbles were found in the residual soil profile.

Weathered Schist Bedrock

Weathered schist bedrock was encountered in most boreholes, except for the shallower boreholes (BH103, BH106) and the deeper BH108, at the depths and RLs tabulated below:

Borehole	Depth to Bedrock (m)	Bedrock Surface RL (mAHD)
BH101	5.0[1]	302.0
BH102	3.5	303.3
BH104	3.5	303.1
BH105	4.8[1]	301.2
BH107	5.6[1]	301.3
BH108	>6.0	<300.7

Note 1: The bedrock was only proven for a length of either 0.1m or 0.2m.

The schist bedrock on first contact was generally distinctly or slightly weathered and of medium to high strength. In BH105, the schist was extremely weathered and of hard (soil) strength. In BH101, BH105 and BH107, the schist was only proven for a length of either 0.1m or 0.2m due to auger refusal; hence, it is possible that the schist was a 'floater' or a boulder, and not necessarily bedrock. In all five boreholes where the schist was encountered, auger refusal occurred.

We note that there are limitations in assessing rock strength based on a combination of auger penetration resistance, tactile examination of recovered auger cuttings and correlations with laboratory moisture content test results, and in some instances the assessed strength may vary from the actual strength by one order of rock strength.

Groundwater

The boreholes were 'dry' during and a short time after completion of drilling. We note that the groundwater levels may not have stabilised within the limited observation period. No long-term groundwater level monitoring was carried out.

3.3 Laboratory Test Results

The moisture content and Atterberg Limits test results confirmed our field classification of the site soils. The Atterberg Limits and linear shrinkage test results confirmed the sampled silty fill from BH101 to be of low plasticity with a slight potential for shrink-swell reactivity with changes in moisture content. Similarly, the residual silty clays from BH104 and BH105 were confirmed to be of medium plasticity with a moderate to high potential for shrink-swell reactivity.

The results of the moisture content tests carried out on recovered rock cutting samples correlated well with our field assessment of bedrock strength.

The four-day soaked CBR tests carried out on a sandy silt fill sample from BH102 and on a residual silty clay sample from BH103 resulted in values of 25% and 14%, respectively, when compacted to at least 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The samples were compacted prior to CBR testing at close to their Standard Optimum Moisture Contents (SOMC), which were up to 5.1% 'wet' of their respective insitu moisture contents.

The results of the soil aggression testing are tabulated below:

Borehole	Sample Depth (m)	Soil Description	Soil pH	Soil Chloride (mg/kg)	Soil Sulfate (mg/kg)	Resistivity in Soil (ohm.m)
BH102	0.5-0.95	Silty Fill	7.8	4,800	4,400	2.3
BH102	1.5-1.95	Residual Silty Clay	8.4	1,900	540	6.6
BH104	1.5-1.95	Residual Silty Clay	8.3	2,800	2,600	3.5
BH105	3.0-3.45	Residual Silty Clay	8.4	1,400	690	8.0

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

We consider the following to be the primary geotechnical issues for the proposed police station:

- Removal of trees which could result in an increase in swell movements in the vicinity of the trees.
- Presence of 'uncontrolled' fill to depths ranging between 0.1m (BH104) and 1.2m (BH101 & BH107).
- Presence of surficial silty fill which is susceptible to rapid loss in strength when wet.
- Presence of residual clays with a moderate to high potential for shrink-swell movements with changes in moisture content.
- Presence of low insitu soil moisture contents which will warrant a Modified compaction specification rather than a Standard compaction specification.
- Limited proving of bedrock across the site.

The effects of the above geotechnical issues on design and construction are detailed in the sections which follow.

4.2 Removal of Existing Trees

The proposed development will require removal of several existing trees. We note that the existing trees have likely caused localised 'drying out' of the surrounding soils. Removal of the trees will therefore lead to the recovery of the soil moisture content, resulting in differential swell movements in the vicinity of the trees and its root system (which can extend for a significant distance from the trunk). The swell movements generated by the removal of the trees are a differential movement, and are in addition to the expected shrink-swell movements which occur in cohesive soils due to weather related natural moisture changes.

It is likely that moisture equilibrium in the soils, following removal of the tree stumps and roots, could take one to two years to develop. We therefore strongly recommend that the trees be removed as early as possible ahead of construction so as to reduce the effects on the proposed police station.

4.3 Existing Fill

No details on the existing fill (ie. placement method, compaction specification, density test records, etc.) have been provided to us. The borehole testing generally indicated the fill to be moderately or well compacted. However, there is no direct correlation between the SPT results, hand penetrometer readings and the insitu densities of the fill. The tests are also affected by friction developed during the driving of the SPT, the presence of gravel within the fill and the moisture content of the fill. Nonetheless, they provide a qualitative guide.

Based on the above, and without documentation proving the fill was placed and compacted to an appropriate specification, we consider the existing fill to be uncontrolled.

The fill is therefore considered to be an unsuitable bearing stratum for on-grade floor slabs and footings. However, it may be a suitable subgrade for the proposed car park pavement provided the subgrade preparation works detailed in Section 4.4.4 below are carried out.

4.4 Earthworks

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

4.4.1 Site Drainage

The subgrade at the site is expected to undergo substantial loss in strength when wet, particularly the silt subgrade. Furthermore, the clay subgrade is expected to have a moderate to high shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained subgrade may become untraffickable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

4.4.2 Site Preparation

Following removal of the of the train track 'loop' and existing trees (including their root balls), all topsoil, root affected soils, remnants of previous pavements and any deleterious fill or contaminated soil should be stripped. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as formwork fill or engineered fill. They may however be reused for landscaping purposes subject to confirmation by JKE. Reference should be made to the JKE report for guidance on the offsite disposal of soil.

Care must be taken not to undermine or remove support from the site boundaries during the stripping works and during excavation down to design levels. Excavation of the soil profile can be completed using hydraulic excavators and/or dozers.

Raising of site levels below the proposed building can be carried out using formwork fill, as we have recommended in Section 4.6.2 to suspend the ground floor slab off the footings. For the proposed car park, raising of site levels up to design subgrade level will need to be carried out using engineered fill.

4.4.3 Formwork Fill

Site won silt and clay soils can be reused as formwork fill on condition that approval from JKE has been obtained, and that the material is free of organic matter and contains a maximum particle size not exceeding 100mm. Formwork fill should be nominally compacted in maximum 250mm thick loose layers using a large static pad-foot roller. The material should be wetted up to close to its Modified Optimum Moisture Content (MOMC). Care must be taken not to over-moisten the clays and silts as the material will soften, and not to over-compact the material as this may result in additional, and differential, long-term swell movements below the ground floor slab.

4.4.4 Subgrade Preparation

Following stripping and excavation, the subgrade below the proposed car park should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

Subgrade heaving during proof-rolling may occur in areas where the soils have become 'saturated' and/or where under-compacted existing fill may be present, including test pit backfill from the recent JKE investigation. Heaving areas should be locally removed to a stable base and replaced with engineered fill, as outlined in Section 4.4.5 below. Alternative subgrade improvement options, as appropriate, should be provided by the geotechnical engineer following the proof rolling inspection.

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

Engineered fill must be used to raise site levels below the proposed car park.

4.4.5 Engineered Fill

General

From a geotechnical perspective, site won silts and clays are considered suitable for re-use as engineered fill on condition that approval has been obtained from JKE and that they are free of organic matter and contain a maximum particle size not exceeding 75mm. Silts and clays should be thoroughly blended to improve the workability of the former soil type.

If there is a shortfall of site won materials, then we recommend that further advice be sought from JKG in relation to material testing and compaction specification, once the material has been sourced. Notwithstanding, the imported material must be Virgin Excavated Natural Material (VENM) and have a soaked CBR value of at least 3% when compacted to 95% of Modified Maximum Dry Density (MMDD) and at MOMC.

Engineered fill comprising site won materials should be compacted in maximum 250mm thick loose layers using a large vibratory pad-foot roller to a minimum density ratio of 95% of MMDD and at a moisture content within -2% and +1% of MOMC. Based on the results of the investigation, we expected that the excavated materials will require moisture conditioning in order to conform to the above moisture specification. Care must be taken not to over-moisten the silty soils as they will become difficult to compact.

Service Trenches

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of compaction plant that can be placed in trenches, backfilling should be carried out in maximum 150mm thick loose layers and compacted using a trench roller, a pad foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a 'Wacker Packer'). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also reduce to 50mm. The compaction specification provided above is applicable.

Earthworks Inspection and Testing

Density tests should be carried out on the engineered fill to confirm the above specification is achieved, as outlined below:

- The frequency of density testing for general engineered fill should be at least one test per layer per $500m^2$ or one test per $100m^3$ distributed reasonably evenly throughout the full depth and area, or 3 tests per visit, whichever requires the most tests (assumes maximum 250mm thick loose layers).
- The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 150mm thick loose layers). This implies that at each test location, two compacted layers will be tested simultaneously.

Level 2 testing in accordance with Section 8 of AS3798-2007 is considered appropriate for this project. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by the Client or their representative, and not by the contractor.

4.4.6 Warning

The long-term successful performance of the car park pavement 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.

4.4.7 Permanent Batter Slopes

Where permanent batter slopes of soil cuts or of fill embankments are proposed, we recommend that they be graded at no steeper than 1V on 2H. Flatter slopes may be preferred for mowing. Surface erosion protection, for example, topsoil and turf, etc. should be provided to the permanent batter slopes. Kerbs and gutters, or dish drains should also be provided along the crest of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system.

4.5 Retaining Walls

Free-standing cantilevered retaining walls supporting areas where some movement can be tolerated and which are independent of the proposed building, should be designed using a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient (K_a) of 0.33 for the soil profile, assuming a horizontal retained surface.

Free-standing cantilevered retaining walls supporting areas where movements are undesirable (eg. due to presence of movement sensitive buried services, etc.) or are incorporated into the proposed building (eg. ground floor walls in the areas of cut), should be designed using a triangular lateral earth pressure distribution with an 'at-rest' earth pressure coefficient (K_0) of 0.5 for the soil profile, assuming a horizontal retained surface.

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

Any surcharge loads affecting the retaining walls (eg. construction traffic, pavement/slab loads, compaction stresses during backfilling, inclined retained surfaces, etc.) should be allowed in the design using the appropriate earth pressure coefficient from above.

The retaining walls should be designed as permanently drained. Subsurface drains behind free-standing cantilevered retaining walls should incorporate (1) an appropriately sized 'ag' pipe with filter sock, surrounded by (2) free draining, single size, durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate, and encapsulated within (3) a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion. All drainage water should be piped to the stormwater system.

Free-standing cantilevered retaining walls independent of the proposed building and founded in residual clay and/or silt of at least very stiff strength may be designed for an allowable bearing pressure of 150kPa. Alternatively, free-standing cantilevered retaining walls independent of the proposed building and founded in the existing uncontrolled fill may be designed for an allowable bearing pressure of 50kPa. Movement joints should be provided at no more than about 5m centres in order to accommodate likely differential movements (ie. settlement and shrink-swell).

The passive lateral toe resistance for free-standing cantilevered retaining walls independent of the proposed building and founded in residual clay and/or silt of at least very stiff strength may be estimated using a 'passive' earth pressure coefficient (K_p) of 3.0 (but with a Factor of Safety of at least 2.0 to limit deformations), assuming horizontal ground in front of the wall. For retaining wall footings founded in the uncontrolled fill, the K_p value should reduce to 2.8. The embedment depth design must take into account any nearby localised excavations in front of the wall, such as for footings and service trenches.

The retaining wall footing excavations should be cleaned out, inspected and Dynamic Cone Penetrometer (DCP) tested (as appropriate) by a geotechnical engineer (prior to the installation of the reinforcement cage), and poured without delay. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering.

4.6 Footings

4.6.1 Site Classification to AS2870-2011

Due to the presence of uncontrolled fill and the abnormal soil moisture conditions generated by tree removal and tree retention, the site strictly classifies as Class 'P' in accordance with AS2870-2011 'Residential Slabs and Footings'. Notwithstanding, due to the size and nature of the proposed building, the standard designs provided in AS2870-2011 do not apply. As such, the footings will need to be designed using engineering principles.

As a guide however, characteristic surface movements in the order of 90mm (ie. equivalent to a Class 'E-D' site in accordance with AS2870-2011) should be anticipated for design purposes.

4.6.2 Geotechnical Design

Based on the results of the investigation, we recommend that the proposed building and external structures be uniformly supported on piles socketed into the underlying schist bedrock. The ground floor slab should be fully suspended off the piled footings.

Conventional bored piles socketed at least 0.3m into low strength or stronger schist bedrock may be designed for an allowable end bearing pressure of 1000kPa. Sockets formed below the minimum 0.3m length requirement may be designed for allowable shaft adhesion values of 100kPa in compression and 50kPa in tension (uplift), on condition that the pile shaft is suitably roughened to a Roughness Class equivalent to at least R2. R2 roughness is defined as grooves of depth 1mm to 4mm, and width greater than 2mm, and at a spacing of 50mm to 200mm.

Consideration should be given to designing the piles, where possible, in end bearing only as the drilling of sockets through the medium and high strength schist will be difficult.

The provided design pressures are based upon serviceability criteria of deflections at the pile base of less than 1% of the pile diameter.

Due to the presence of medium and high strength (and possibly stronger) schist, a full copy of this report should be provided to the prospective piling contractors to ensure that appropriate high torque drill rigs and equipment (eg. rock augers, coring buckets, downhole hammer, etc.) are brought to site.

Conventional bored piles should be cleaned-out, inspected and poured on the same day as drilling. The piling should be inspected by a geotechnical engineer during the initial stages and then periodically during the works to confirm that a satisfactory bearing stratum has been achieved.

Based on the expected shrink-swell nature of the site soils, we strongly recommend that all ground beams between pile heads and the suspended ground floor slab be poured over void formers. The void formers must be able to accommodate swell movements of 75mm. Therefore, a minimum 100mm thick collapsible cardboard void former should be used.

Due to the shrink-swell nature of the site soils, garden beds should be avoided adjacent to the proposed building as moisture ingress into the subgrade at these locations could cause movement and damage to nearby structural elements. If garden beds are required, particularly for the proposed internal courtyard, then further advice should be sought from JKG with respect to detailed design (ie. planter boxes, moisture barriers, subsoil drainage, etc.).

Furthermore, to reduce rainwater sheeting flows off the external walls from entering the subgrade, we recommend that all joints between the proposed new building and external concrete pavements/footpaths (including the internal courtyard pavement) be infilled using a flexible 'Mastic' sealer.

Attention is drawn to precautionary and foundation maintenance measures outlined in Appendix B of AS2870-2011 for a Class 'E' site. All surfaces adjacent to the proposed building must be graded away to promote surface water run-off.

4.6.3 Soil Aggression

The laboratory test results have indicated alkaline subsoil conditions, as well as low soil sulfate and chloride contents. The laboratory soil resistivity test results have indicated moderate conditions to steel piles.

In accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation' and Table 5.2 of AS2870-2011, the exposure classification to concrete piles/footings is 'non-aggressive' and 'A1', respectively.

4.6.4 Earthquake Design Parameters

The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia' (including Amendments 1 & 2):

- Hazard Factor (Z) = 0.08
- Site Subsoil Class = Class C_e

4.7 Car Park Pavement

Based on the laboratory test results, established correlations between CBR, linear shrinkage and Plasticity Index, and our experience with similar soils, we recommend that the proposed car park pavement be designed for a CBR value of 3% or a short-term Young's modulus of 22MPa for the compacted soil subgrade. The laboratory CBR test results are not considered to be representative of the site soils, and were most likely artificially elevated due to the concentrated presence of gravels in the test samples.

If the proposed car park is to include a concrete pavement, then it should be supported on an unbound granular sub-base. The sub-base should be at least 100mm thick and comprise good quality fine crushed rock such as DGB20 (RMS QA Specification 3051 unbound granular material) and compacted using a large smooth drum roller to a minimum density ratio of 98% of MMDD. Adequate moisture conditioning to within 2% of MOMC should be provided during placement so as to reduce the potential for material breakdown during compaction. The sub-base material would provide more uniform slab support and would reduce 'pumping' of subgrade 'fines' at joints due to vehicular movements. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

If the proposed car park is to include an AC pavement, then we recommend that the basecourse material comprises DGB20 (RMS QA Specification 3051). The basecourse material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to a minimum density ratio of 98% of MMDD. Adequate moisture conditioning to within 2% of MOMC should be provided during placement. We further recommend that all sub-base materials comprise DGS20 or DGS40 (RMS QA Specification 3051). The sub-base material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 95% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement.

Density tests should be regularly carried out on the granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 1000m^2, or three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing of pavement material compaction is considered appropriate. Again, the GTA should be directly engaged by the Client or their representative.

In order to protect the pavement edge, subsoil drains should be provided around the perimeter of the proposed car park, with invert levels of at least 200mm below subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.8 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:

1. Pre-construction meeting to discuss the earthworks.
2. Proof-rolling inspections.
3. Additional advice if imported materials are to be brought onto site for reuse as engineered fill.
4. Density testing of all engineered fill and granular pavement materials to Level 2 control by a GTA.
5. Retaining wall footing inspections, and DCP testing, as appropriate.
6. Pile inspections.
7. Additional advice if garden beds are to be placed adjacent to the new building, including within the internal courtyard.

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 JKG accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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 JKG. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

ABN 43 002 145 173

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

Client:	JK Geotechnics	**Ref No:**	32665A2
Project:	Proposed Police Station	**Report:**	A
Location:	51 Bromide Street, Broken Hill, NSW	**Report Date:**	25/02/2020
		Page 1 of 1	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
101	0.50 - 0.80	6.5	29	21	8	3.0
101	1.50 - 1.95	8.2	-	-	-	-
101	3.00 - 3.45	10.4	-	-	-	-
101	4.50 - 4.80	6.1	-	-	-	-
102	4.10 - 4.60	1.1	-	-	-	-
104	0.50 - 0.95	15.8	49	17	32	14.5
104	4.50 - 4.90	0.7	-	-	-	-
105	0.50 - 0.95	13.4	43	15	28	12.5
105	4.80 - 4.85	7.3	-	-	-	-
107	5.60 - 5.70	0.6	-	-	-	-

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 17/02/2020.
- Sampled and supplied by client. Samples tested as received.
- BH 105 (depth: 4.80-4.85m) tested on small deeper fraction.

NATA

NATA Accredited Laboratory Number:1327

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the items tested or sampled.

25/02/2020
Authorised Signature / Date
(D. Treweek)

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

115 Wicks Road
Macquarie Park, NSW 2113
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North Ryde, Bc 1670
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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32665A2
Project:	Proposed Police Station	**Report:**	B
Location:	51 Bromide Street, Broken Hill, NSW	**Report Date:**	25/02/2020
		Page 1 of 1	

BOREHOLE NUMBER		BH 102	BH 103
DEPTH (m)		0.00 - 1.00	0.50 - 1.50
Surcharge (kg)		9.0	9.0
Maximum Dry Density (t/m^3)		2.06 STD	1.99 STD
Optimum Moisture Content (%)		9.4	11.8
Moulded Dry Density (t/m^3)		2.03	1.95
Sample Density Ratio (%)		99	98
Sample Moisture Ratio (%)		94	100
Moisture Contents			
Insitu (%)		4.3	7.6
Moulded (%)		8.9	11.9
After soaking and			
After Test, Top 30mm(%)		12.0	15.1
	Remaining Depth (%)	11.0	13.5
Material Retained on 19mm Sieve (%)		7*	0
Swell (%)		0.0	0.0
C.B.R. value:	@2.5mm penetration	25	14

NOTES: Sampled and supplied by client. Samples tested as received.
- Refer to appropriate Borehole logs for soil descriptions
- Test Methods: AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 17/02/2020.
- * Denotes not used in test sample.

NATA

NATA Accredited Laboratory Number:1327

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the items tested or sampled.

25/02/2020
Authorised Signature / Date
(D. Treweek)

All services provided by STS are subject to our standard terms and conditions. A copy is available on request.

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS 236883

Client Details

Client	JK Geotechnics
Attention	Arthur Kourtesis
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32665AG, Broken Hill**
Number of Samples	4 Soil
Date samples received	17/02/2020
Date completed instructions received	17/02/2020

Analysis Details

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

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

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

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

Report Details

Date results requested by	24/02/2020
Date of Issue	24/02/2020

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

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil					
Our Reference		236883-1	236883-2	236883-3	236883-4
Your Reference	UNITS	BH102	BH102	BH104	BH105
Depth		0.5-0.95	1.5-1.95	1.5-1.95	3.0-3.45
Date Sampled		04/02/2020	04/02/2020	04/02/2020	04/02/2020
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	20/02/2020	20/02/2020	20/02/2020	20/02/2020
Date analysed	-	20/02/2020	20/02/2020	20/02/2020	20/02/2020
pH 1:5 soil:water	pH Units	7.8	8.4	8.3	8.4
Chloride, Cl 1:5 soil:water	mg/kg	4,800	1,900	2,800	1,400
Sulphate, SO4 1:5 soil:water	mg/kg	4,400	540	2,600	690
Resistivity in soil*	ohm m	2.3	6.6	3.5	8.0

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	236883-2
Date prepared	-			20/02/2020	[NT]	[NT]	[NT]	[NT]	20/02/2020	20/02/2020
Date analysed	-			20/02/2020	[NT]	[NT]	[NT]	[NT]	20/02/2020	20/02/2020
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	104	#
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	107	#
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments

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

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

JKGeotechnics

BOREHOLE LOG

Borehole No.
101
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 307.0m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 6.75 HRS			0			FILL: Clayey sand, fine to medium grained, grey brown, trace of silt and fine to coarse grained igneous gravel.	D			APPEARS WELL COMPACTED
		N > 15 16,15/ 150mm END				FILL: Sandy clayey silt, low plasticity, red brown, fine to medium grained sand, trace of fine grained gravel.	w<PL		>600 >600	
			1		CI	Silty sandy CLAY: medium plasticity, red brown, fine to medium grained sand, trace of fine to coarse grained gravel and cobbles.	w<PL	Hd		RESIDUAL TOO FRIABLE FOR HP READINGS
		N = 12 4,5,7	2							
		N = 15 1,5,10	3			Silty sandy CLAY: medium plasticity, red brown mottled dark grey, fine grained sand, trace of fine grained gravel.				TOO FRIABLE FOR HP READINGS
		N > 12 15,12/ 150mm END	4			as above, but red brown, orange brown and light grey, with fine to medium grained gravel.				
			5		-	SCHIST: medium grained, grey and dark grey.	SW	M-H		GLOBE-VAUXHALL SCHIST ZONE HIGH 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 5.2m				'TC' BIT REFUSAL (POSSIBLY 'FLOATER')
			6							
			7							

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BOREHOLE LOG

Borehole No.
102
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 306.8m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION AND AFTER 1.25 HRS		N = 20 9,10,10	0			FILL: Sandy clayey silt, low plasticity, grey brown, fine to medium grained sand, with medium to coarse grained gravel and cobbles.	w<PL			APPEARS WELL COMPACTED
		N = 10 2,5,5	1		CI	Silty CLAY: medium plasticity, red brown, with fine grained sand.	w<PL	VSt-Hd		RESIDUAL TOO FRIABLE FOR HP READINGS
		N = 12 2,5,7	2 3			as above, but red brown and light grey.				
			4		-	SCHIST: medium grained, light grey, dark grey and orange brown, with clay seams.	DW	M		GLOBE-VAUXHALL SCHIST ZONE MODERATE 'TC' BIT RESISTANCE
			5 6 7			END OF BOREHOLE AT 4.6m				'TC' BIT REFUSAL

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BOREHOLE LOG

Borehole No.
103
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 305.9m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 clayey silt, low plasticity, red brown, fine to medium grained sand.	w<PL			
		N > 13 15,13/ 100mm REFUSAL	1		CI	Silty CLAY: medium plasticity, red brown, with fine grained sand.	w<PL	Hd	>600 >600 >600	RESIDUAL
		N = 15 5,7,8							>600 >600 >600	
			2			END OF BOREHOLE AT 1.95m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **104**

1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD

Project: PROPOSED POLICE STATION

Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 306.6m

Date: 4/02/2020 **Datum:** AHD

Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 2.25 HRS			0			FILL: Sandy clayey silt, low plasticity, grey brown, fine to medium grained sand, trace of fine to medium grained gravel.	w<PL			
					CI	Silty CLAY: medium plasticity, red brown, with fine grained sand, trace of fine to medium grained gravel.	w<PL	Hd		RESIDUAL
	DS	N = 16 4,7,9							>600 >600 >600	
			1							
	DS	N = 20 4,9,11							>600 >600 >600	
			2							
			3							
	DS	N = 16 1,3,13							>600	
					-	SCHIST: medium grained, light grey, dark grey and orange brown, with clay seams and iron indurated seams.	DW	M-H		GLOBE-VAUXHALL SCHIST ZONE HIGH 'TC' BIT RESISTANCE
			4							
	DS									
			5			END OF BOREHOLE AT 4.9m				'TC' BIT REFUSAL
			6							
			7							

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BOREHOLE LOG

Borehole No.
105
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 306.0m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 3 HRS			0			FILL: Sandy clayey silt, low plasticity, grey brown, fine grained sand, with medium to coarse grained gravel and asphaltic concrete fragments.	w<PL			
	DS	N = 20 7,8,12			CI	Silty CLAY: medium plasticity, red brown and grey brown, with fine grained sand, trace of fine grained gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
			1					VSt-Hd		
	DS	N = 8 4,3,5								TOO FRIABLE FOR HP READINGS
			2							
	DS	N = 7 3,3,4	3							
			4		ML	Sandy clayey SILT: low plasticity, red brown, fine grained sand.				
	DS	N > 17 4,10, 7/50mm REFUSAL			-	Extremely Weathered schist: sandy clayey SILT, low plasticity, light grey, grey and orange brown, fine grained sand.	XW	Hd		GLOBE-VAUXHALL SCHIST ZONE
			5			END OF BOREHOLE AT 5.0m				'TC' BIT REFUSAL (POSSIBLY 'FLOATER')
			6							
			7							

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BOREHOLE LOG

Borehole No.
106
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 305.8m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 30 MINS						0			FILL: Sandy clayey silt, low plasticity, grey brown, fine to medium grained sand, trace of medium to coarse grained gravel.	w<PL			
				■	N > 20 9,13,7/ 50mm REFUSAL	1		CI	Silty CLAY: medium plasticity, red brown, with fine grained sand, trace of fine to medium grained gravel.	w<PL	Hd	>600 >600	RESIDUAL
				■	N = 15 4,5,10							>600 >600	
						2			END OF BOREHOLE AT 1.95m				
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No. 107

1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 306.9m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 5.75 HRS		N = 10 8,7,3	0			FILL: Sandy clayey silt, low plasticity, grey brown, fine to medium grained sand, with fine to coarse grained gravel and asphaltic concrete fragments.	w<PL			APPEARS MODERATELY COMPACTED
		N = 8 2,3,5	1		CI	Silty CLAY: medium plasticity, red brown, with fine to medium grained sand, trace of fine grained gravel.	w<PL	VSt- Hd	>600	RESIDUAL
		N = 11 2,5,6	2			Silty sandy CLAY: medium plasticity, red brown and orange brown, fine to medium grained sand, trace of fine to medium grained gravel.				TOO FRIABLE FOR HP READINGS
		N > 5 5,5/0mm REFUSAL	3, 4, 5			Silty CLAY: medium plasticity, orange brown and red brown mottled grey and light grey, with fine grained sand, trace of fine to coarse grained gravel and cobbles.				
					-	SCHIST: medium grained, grey and orange brown.	DW	H		GLOBE-VAUXHALL SCHIST ZONE
			6, 7			END OF BOREHOLE AT 5.7m				'TC' BIT REFUSAL (POSSIBLY 'FLOATER')

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BOREHOLE LOG

Borehole No.
108
1/1

Client: GARDNER WETHERILL & ASSOCIATES PTY LTD
Project: PROPOSED POLICE STATION
Location: 51 BROMIDE STREET, BROKEN HILL, NSW

Job No.: 32665A2 **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 306.7m
Date: 4/02/2020 **Datum:** AHD
Plant Type: GEMCO HC7 **Logged/Checked by:** A.C.K./A.J.

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 AND AFTER 4.25 HRS			0			FILL: Sandy clayey silt, low plasticity, grey brown, fine to medium grained sand, trace of medium grained gravel.	w<PL			APPEARS MODERATELY COMPACTED
	DS	N = 12 6,6,8								
			1		ML	Sandy clayey SILT: medium plasticity, red brown, fine grained sand.	w<PL	VSt- Hd		RESIDUAL
					CI	Silty CLAY: medium plasticity, red brown, with fine grained sand, trace of fine grained gravel.		Hd		
	DS	N = 16 4,7,9							>600 >600 >600	
			2							
	DS	N = 13 3,5,8	3						>600 >600 >600	
			4		ML	Sandy clayey SILT: low plasticity, red brown and orange brown, fine grained sand, trace of fine to medium grained gravel.				
	DS	N > 14 1,6,8/50mm REFUSAL								TOO FRIABLE FOR HP READINGS
			5							
			6			END OF BOREHOLE AT 6.0m				
			7							

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IMAGE SOURCE: http://maps.au.nearmap.com.OCT. 2019

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

Title:	SITE LOCATION PLAN
Location:	51 BROMIDE STREET BROKEN HILL, NSW
Report No: 32665A2	Figure: 1

JKGeotechnics

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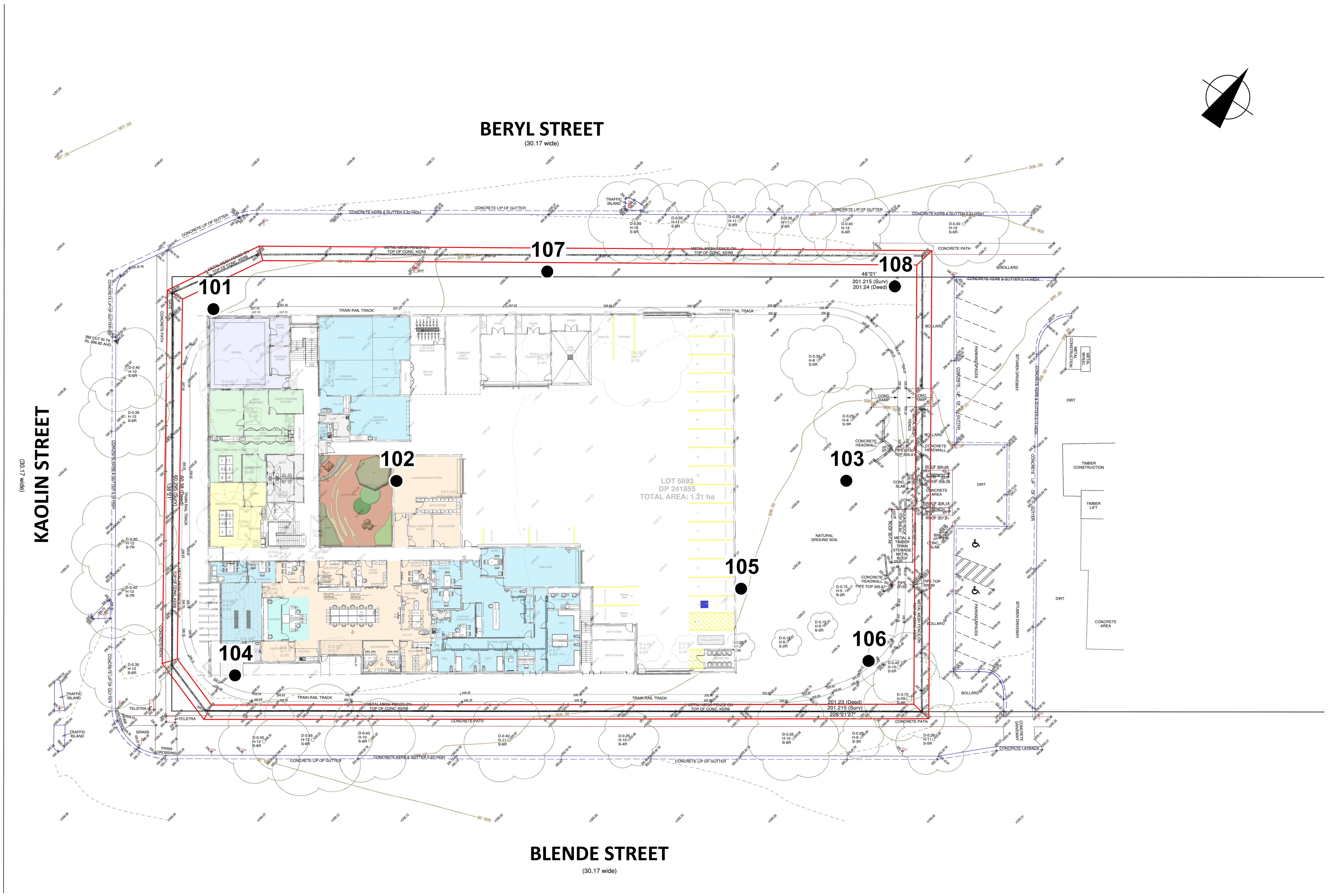

0 5 10 15 20 25
SCALE 1:500 @A3 m

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

Title:	BOREHOLE LOCATION PLAN
Location:	51 BROMIDE STREET BROKEN HILL, NSW
Report No: 32665A2	Figure: 2

JKGeotechnics

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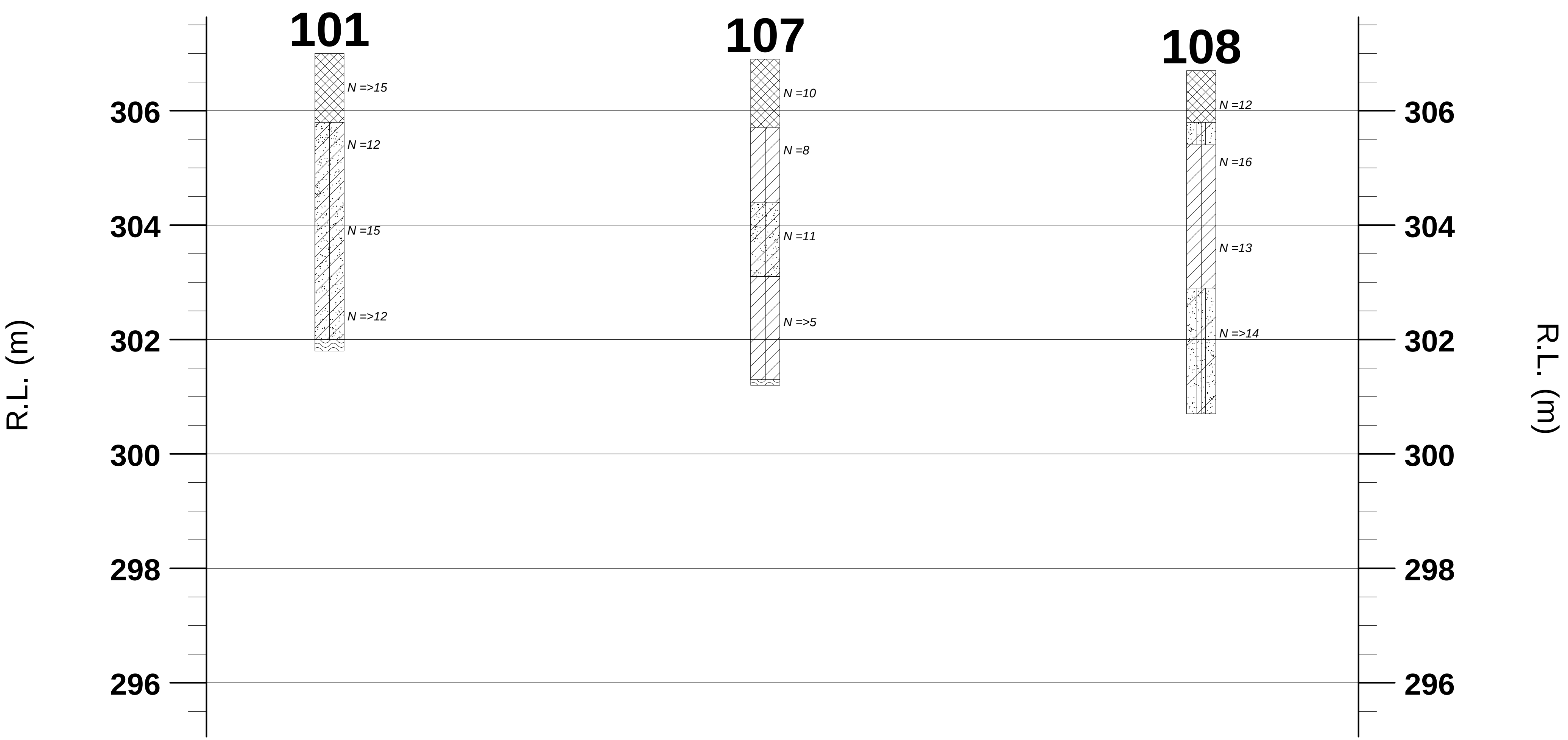
GRAPHICAL BOREHOLE SUMMARY



Fill
Silty Clay
Sandy Silty Clay
Sandy Clayey Silt
Phyllite, Schist

Nc SOLID CONE BLOW COUNTS PER 150mm

N SPT "N" VALUE

NOTE: REFER TO BOREHOLE LOGS

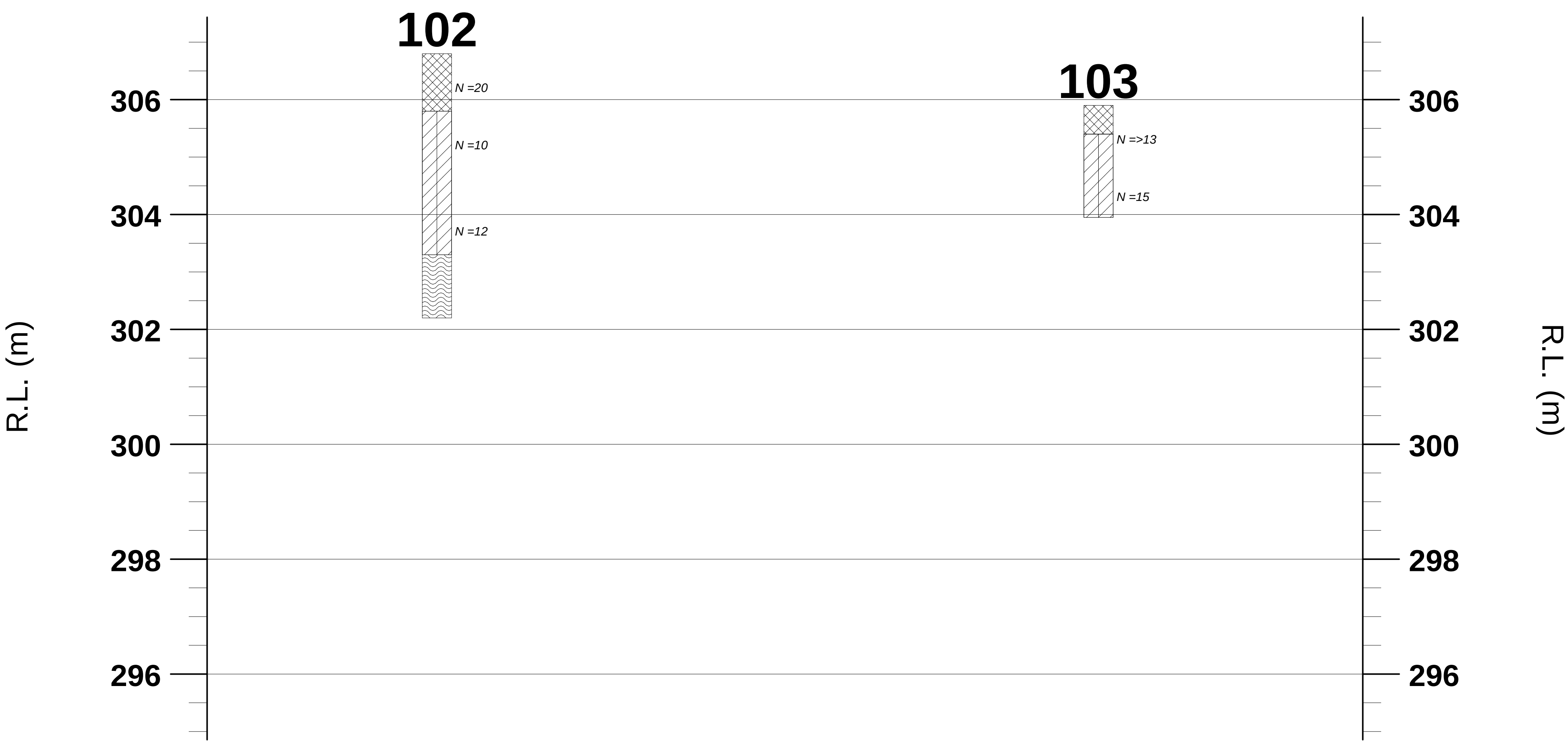
Scale: 1 : 100 (vert) ; NTS (horiz)

JKGeotechnics

Job No.: 32665A2

Figure No.: 3

GRAPHICAL BOREHOLE SUMMARY



Fill
Silty Clay
Phyllite, Schist

N — SPT "N" VALUE
Nc — SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

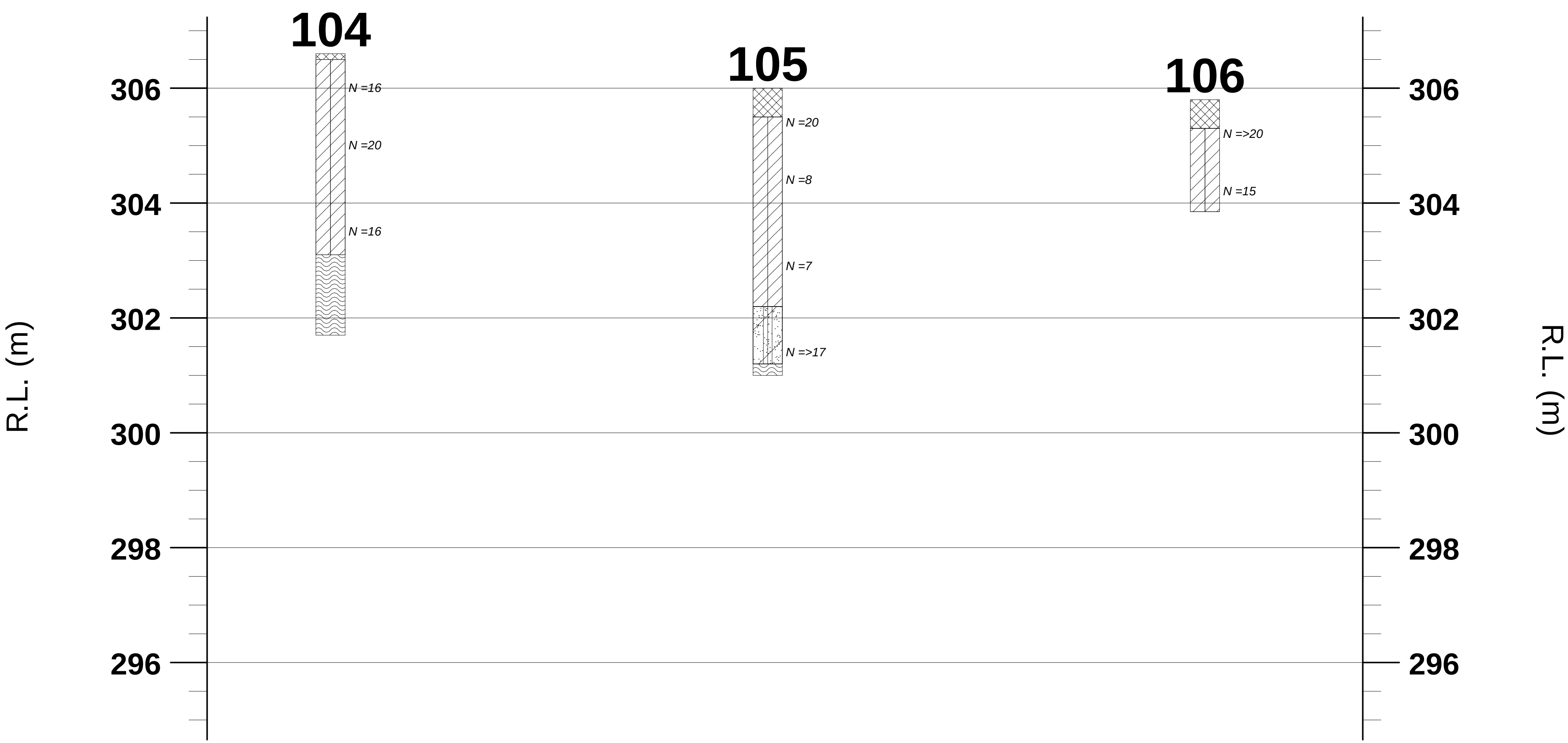
Scale: 1 : 100 (vert) ; NTS (horiz)

JKGeotechnics

Job No.: 32665A2

Figure No.: 4

GRAPHICAL BOREHOLE SUMMARY



Fill

Sandy Clayey Silt

Silty Clay

N SPT "N" VALUE

Phyllite, Schist

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JKGeotechnics

Job No.: 32665A2

Figure No.: 5

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

FILL

TOPSOIL

CLAY (CL, CI, CH)

SILT (ML, MH)

SAND (SP, SW)

GRAVEL (GP, GW)

SANDY CLAY (CL, CI, CH)

SILTY CLAY (CL, CI, CH)

CLAYEY SAND (SC)

SILTY SAND (SM)

GRAVELLY CLAY (CL, CI, CH)

CLAYEY GRAVEL (GC)

SANDY SILT (ML, MH)

PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

CONGLOMERATE

SANDSTONE

SHALE/MUDSTONE

SILTSTONE

CLAYSTONE

COAL

LAMINITE

LIMESTONE

PHYLLITE, SCHIST

TUFF

GRANITE, GABBRO

DOLERITE, DIORITE

BASALT, ANDESITE

QUARTZITE

OTHER MATERIALS

BRICKS OR PAVERS

CONCRETE

ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

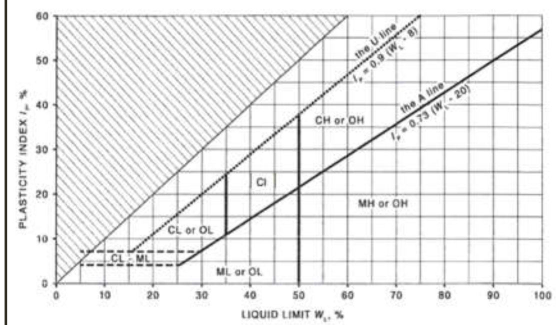
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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