REPORT TO

GROUP GSA

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED POLICE STATION

AT

16-18 THREDBO TERRACE, JINDABYNE, NSW

Date: 17 December 2020

Ref: 33612Arpt

JKGeotechnics

www.jkgeotechnics.com.au

T: +61 2 9888 5000

JK Geotechnics Pty Ltd

ABN 17 003 550 801

Report prepared by:

Andrew Jackaman
Principal | Geotechnical Engineer

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

DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33612Arpt	Final Report	17 December 2020

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

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

a) JKG's proposal in respect of the work covered by the Report;

b) The limitations defined in the Client's brief to JKG;

c) The terms of contract between JKG and the Client, including terms limiting the liability of JKG.

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

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

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

Table of Contents

<u>ATTACHMENTS</u>

STS Table A: Moisture Content, Atterberg Limits & Linear Shrinkage Test Report
STS Table B: Four Day Soaked California Bearing Ratio Test Report
Envirolab Services 'Certificate of Analysis 256957'
Borehole Logs 1 to 4
Dynamic Cone Penetration Test Results Sheet (5 to 7)
Figure 1: Site Location Plan
Figure 2: Test Location Plan
Vibration Emission Design Goals
Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Police Station at 16-18 Thredbo Terrace, Jindabyne, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Janine Graves of Group GSA in an email dated 26 October 2020. The commission was on the basis of our fee proposal, Ref. P52522A dated 2 September 2020.

We have been provided with the following information:

- Architectural design drawings prepared by Group GSA (Project No. 200005, Drawing Nos. JIN-AR-0100^{D}, JIN-AR-0101^{D}, JIN-AR-0102^{E}, JIN-AR-0103^{F}, JIN-AR-0104^{A}, JIN-AR-0200^{F}, JIN-AR-0201^{E}, JIN-AR-0300^{C}, JIN-AR-0301^{C}, JIN-AR-0400^{C}, JIN-AR-0401^{C}, JIN-AR-0550^{B}, JIN-AR-0700^{C} & JIN-AR-0701^{A});
- Survey drawings prepared by Crux (Drawing No. 122588-SU-DT-001, Sheets 1 to 4, Revision A, dated 19 November 2020). The survey datum is the Australian Height Datum (AHD).

Based on the supplied information, we understand that the proposed development will comprise demolition of the existing buildings and structures, and construction of new Police Station building, accommodation building, carport, vehicle storage facility, and driveway. More specific details are summarised below:

- The proposed Police Station will be a two-storey building, and the ground floor slab will be constructed at approximately RL929.0m.
- The proposed Police Accommodation will be a two-storey building, and the ground floor slab will be constructed at approximately RL931.5m.
- The rear of the proposed carport slab will be constructed at approximately RL930.45m. The slab will grade down to the north at 2.5%.
- The proposed vehicle storage facility will be a single-storey building, and the ground floor slab will be constructed at RL930.0m.
- A new driveway will surround the proposed Police Station building, providing access to the carport and vehicle storage facility.
- The proposed development will require terracing of the hillside and will predominantly require excavation to a maximum depth of about 3m. Minor filling to a maximum height of about 0.5m will be required for the proposed Police Station building.

Based on a conversation with Rory Dale of Northrop Consulting Engineers on 17 December 2020, we understand that an allowable bearing pressure of 600kPa can be accommodated for footing design.

The purpose of the investigation was to assess the subsurface conditions across the site and, based on the information obtained, to present our comments and recommendations on site preparation, excavation conditions and drainage, batter slopes, retaining walls, earthworks, site classification to AS2870-2011, footings, soil aggression to concrete, earthquake design parameters, and external pavements.

This report confirms and amplifies our preliminary emailed geotechnical advice, Ref. 33612Alet dated 4 December 2020.

The geotechnical investigation was carried out in conjunction with a 'Preliminary (Stage 1) Site Investigation' by our environmental division, JK Environments. Reference should be made to the separate JK Environments report, Ref. E33612PTrpt.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 23 & 24 November 2020 and comprised the works outlined below. The test locations are shown on the attached Figure 2.

- Four boreholes (BH1 to BH4) were auger drilled and tested to refusal depths between 1.6m and 5.7m below existing grade. The boreholes were completed using our track mounted JK205 drill rig, which is equipped for site investigation purposes. Concrete pavements, where encountered, were diatube cored with water flush. The relative density/strength of the soil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split-spoon sampler, and by tactile examination. The strength of the underlying bedrock was assessed by observation of the auger penetration resistance when using a twin-pronged tungsten carbide (TC) bit, together with examination of recovered auger cuttings and correlations with subsequent laboratory moisture content test results.

- Three Dynamic Cone Penetrometer (DCP) tests (DCP5 to DCP7) were completed to refusal depths between 1.3m and 2.0m below existing grade, to assess the relative density/strength of the soil profile, and to attempt to probe down to the bedrock surface.

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

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

Further details of the methods and procedures employed in the investigation, including the penetration limitations of the DCP, 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 and DCP test locations, direct the electro-magnetic scanning, nominate testing and sampling, and prepare the attached borehole logs and DCP test results sheet. 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 and weathered rock samples were returned to a second NATA accredited analytical laboratory, Envirolab Services Pty Ltd, for soil pH and sulfate testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis 256957'.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located on a north-facing hillside which grades at approximately 7°. Thredbo Terrace bounds the site to the north. The site itself generally grades down to the north-east at approximately 6°.

At the time of the fieldwork, the western half of the site contained three, single-storey clad buildings, awning structures and a covered BBQ area on a concrete slab. Immediately behind (to the south of) the northernmost building was a grouted boulder retaining wall, which was up to approximately 2m high and supported the adjoining (higher lying) building. Within the eastern half of the site were a single-storey clad house and a clad high-bay shed. The northern side and a short portion of the eastern side of the shed were supported by concrete block retaining walls, which ranged in height between 0.3m and 1.1m. The buildings, shed and retaining walls appeared to be in good external condition based on a cursory inspection.

The southern end of the site had been cut into the hillside. The neighbouring vacant property to the south was predominantly supported by retaining walls. Towards the south-eastern corner of the site, the neighbouring properties to the south and east were supported by either single or terraced concrete block retaining walls with a maximum overall height of 1.3m. Behind the southern-most buildings within the western half of the site, the neighbouring property to the south was supported by a concrete retaining wall. The western half of the concrete retaining wall ranged in height between 0.2m and 0.6m. The eastern half of the concrete retaining wall was underlain by a sub-vertically cut soil batter. Towards the eastern end, the concrete retaining wall was supported by a concrete bucket pier which extended below the lower ground surface level. Elsewhere, the soil below the concrete retaining wall had eroded out, undermining the wall. The overall height of the concrete retaining wall and underlying soil batter was 0.9m. A tonalite (igneous) boulder was present in front of the retaining wall between the two buildings.

A concrete driveway extended centrally through the site off Thredbo Terrace to a concrete hardstand located towards the rear. On the eastern side of the hardstand was a gravel surfaced parking area with remnants of asphaltic concrete surfacing. The concrete pavements were generally in good condition, however a concrete surfaced reinstated service trench was in poor condition with cracking. Immediately behind the building within the eastern half of the site was a gravel surfaced parking area. Grassed areas and scattered small to medium-sized trees surrounded the buildings and pavements on site.

The neighbouring property to the west of the site (14 Thredbo Terrace) contained a single-storey brick house, with a partial basement garage level, which was set back 2m from the common boundary and appeared to be in good condition when viewed from within the subject site and from its road boundary. Ground surface levels across the common boundary were similar over the central and southern portions, but varied by up to 1m at the northern end where the subject site was supported by a slightly battered back, ad-hoc constructed, dry-stacked boulder wall.

The neighbouring property to the east of the site (24 Thredbo Terrace) was vacant and grass covered. Ground surface levels across the common boundary were generally similar, except at the southern end, where the concrete block retaining walls (as discussed above) stepped down onto the subject site.

The neighbouring higher lying property to the south of the site was an open grass covered reserve, which contained numerous tonalite outcrops and boulders, which were assessed to be of at least medium strength when struck with a geological hammer.

3.2 Subsurface Conditions

The 1:250,000 series geological map of Bega-Mallacoota (Geological Survey of NSW, Geological Series Sheet SJ/55-4 and Part of Sheet SJ/55-8) indicates the site to be underlain by Jindabyne Tonalite of the Kosciusko Batholith Igneous Suites. Tonalite is an igneous rock and resembles granite.

Generally, the boreholes encountered concrete or gravel surfacing, overlying fill and/or residual soils, then weathered tonalite bedrock at relatively shallow depths. Reference should be made to the attached borehole logs and DCP test results sheet for specific details at each location. A summary of the subsurface conditions encountered in the boreholes and indicated by the DCP testing is provided below:

Surfacing

Concrete pavements, of either 130mm or 180mm thickness, were encountered in BH2, BH3 and BH4. Steel reinforcement was observed in the recovered concrete cores from BH2 and BH3 only. A granular sub-base layer was not found below the concrete pavements.

At BH1, a 50mm thick, partially bound, gravel surfacing was encountered.

Fill

Clayey fill with igneous gravel inclusions was found below the concrete pavements in BH2 and BH4 to depths of 0.4m and 0.5m, respectively.

Residual Soils

Residual soils comprising clayey sand (BH1 & BH2) and sandy clay (BH3 & B4) were encountered below the pavements/fill. The clayey sand was of medium dense relative density. The sandy clay was of low to medium plasticity and of hard strength.

Weathered Tonalite Bedrock
Extremely weathered tonalite of either hard or dense strength (soil properties) was encountered in the boreholes at depths between 0.5m (BH1) and 1.5m (BH2 & BH4), and indicated by the DCP tests at depths between 1.0m (DCP5) and 2.0m (DCP6). The extremely weathered rock profile contained stronger bands. Based on the known weathering pattern of tonalite, higher strength core stones (boulders) should be expected within the extremely weathered rock and overlying soil profiles, as can be seen outcropping beyond the southern site boundary.

In BH1, BH3 and BH4, the tonalite improved to distinctly weathered and very low or low strength below depths of 1.6m, 1.9m and 4.0m, respectively. At 3.7m depth in BH1, 5.0m depth in BH3 and 5.5m depth in BH4, high strength tonalite was encountered. At 3.8m depth in BH1, 5.1m depth in BH3 and 5.7m depth in BH4, auger refusal occurred in the high strength rock.

In BH2, auger refusal occurred on the surface of high strength rock at 1.6m depth. It is possible that auger refusal occurred on a core stone.

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 indicated the sampled residual sandy clay of low or medium plasticity from BH3 and BH4 to have a slight to moderate potential for shrink-swell reactivity with changes in moisture content.

The four day soaked CBR tests carried out on residual sandy clay samples from BH3 and BH4 resulted in values of 4% and 18%, respectively, when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 4.5kg. The samples were compacted prior to CBR testing at close to their Standard Optimum Moisture Contents (SOMC), which were generally within 1.8% 'wet' of their respective insitu moisture contents. During the four day soaking period, a swell of 3.5% was measured on the BH3 clayey sample.

The results of the soil and extremely weathered rock aggression testing are tabulated below:

Borehole	Sample Depth (m)	Material Description	Soil/Rock pH	Soil/Rock Sulfate (mg/kg)
BH1	0.5-0.95	Extremely Weathered Tonalite (Clayey Sand)	8.0	<10
BH2	0.5-0.95	Clayey Sand	7.9	20
BH4	0.13-0.5	Clayey Fill	8.0	10
	1.5-1.95	Extremely Weathered Tonalite (Sand)	7.3	<10

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

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Preparation

4.1.1 Dilapidation Survey

Prior to commencement of excavation, we recommend that a detailed internal and external dilapidation survey report be completed on the neighbouring house to the west (14 Thredbo Terrace). Dilapidation reports provide a record of existing conditions prior to commencement of excavation. The dilapidation report would therefore be used as a benchmark against which to set vibration limits during rock excavation and fill compaction (if appropriate), and for assessing possible future claims for damage arising from the works.

The owner of the neighbouring property should be asked to confirm in writing that the dilapidation report presents a fair assessment of existing conditions. As the dilapidation report will be relied upon for the assessment of potential damage claims, it must be carried out thoroughly with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc) and photographed.

The dilapidation report should be reviewed by JK Geotechnics and the structural engineer (Northrop Consulting Engineers) prior to commencement of excavation.

4.1.2 Vibration Monitoring

We recommend that regular quantitative vibration monitoring be carried out on the neighbouring house to the west whenever rock hammers and fill compaction plant are operating. The vibration monitoring must commence at the start of rock hammering, and then periodically throughout the remainder of the rock excavation work. The locations of the vibration monitors should be assessed following review of the dilapidation survey report, and should be jointly nominated by JK Geotechnics and the acoustic consultant.

Due to the remoteness of the site and the likely lack of nearby acoustic consultants, it may be prudent to carry out trials at the commencement of rock excavation and at the commencement of fill/granular pavement layer compaction with JK Geotechnics and the acoustic consultant present on site. During these trials we can jointly assess the appropriateness of the rock excavation and compaction methods.

The vibrations on the neighbouring house should be tentatively limited to a peak particle velocity of 5mm/s, subject to review of the dilapidation survey report. If higher vibrations are recorded, then they should be measured against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the associated vibration frequency. Reference should be made to Section 4.2.2 if it is confirmed that transmitted vibrations are excessive during rock excavation.

4.1.3 Demolition and Stripping

Site preparation will include demolition of the existing buildings, internal retaining walls and pavements, removal of existing trees (including their root balls) and stripping of grass, topsoil, root affected soils and any deleterious or contaminated fill. Reference should be made to the JK Environments report for guidance on the offsite disposal of site soils. Care must be taken during site stripping and subsequent excavation works so as not to undermine or remove support from the site boundaries, particularly the boundary retaining walls.

4.1.4 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 clayey 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 their root systems (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 separate to the expected shrink-swell movements which occur in clayey 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 development. In order to mitigate against the risk of differential swell movements, the stump and the lateral primary root system of each tree should be grubbed out in order to remove as much of the localised 'dried out' clayey soils as possible.

4.2 Excavation

Prior to any excavation commencing, reference should be made to the NSW Government 'Code of Practice Excavation Work' dated January 2020.

4.2.1 Excavation Conditions

The proposed development will require excavations to a maximum depth of about 3m below existing grade. We expect that excavation of the soil and extremely weathered tonalite bedrock profiles and much of the very low and low strength tonalite bedrock could be carried out using a 'digging bucket' fitted to a large hydraulic excavator (say, at least 25 tonnes), using a ripping tyne where necessary.

Large core stones (boulders) should be expected to be encountered within the soil and extremely weathered bedrock profiles. Hydraulic rock hammers will most likely need to be used to break down excavated core stones. Notwithstanding, hydraulic rock hammers will be required where medium and high strength tonalite is encountered. Based on the nature of tonalite, it is possible that very high strength (or even stronger) bedrock may be encountered within the proposed excavation depths; for example, below 1.6m depth in BH2 where auger refusal was encountered. Additional cored boreholes will assist in confirming the depth and strength of the more competent tonalite bedrock so that excavation contractors can more accurately predict the cost of the works. Further, the cored borehole information would reduce the potential for latent conditions. We could provide a fee proposal for the additional investigation if requested to do so.

4.2.2 Potential Vibration Risks

We reiterate that rock excavations using hydraulic rock hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to the neighbouring house to the west. As discussed in Section 4.1.2, we recommend that regular quantitative vibration monitoring be carried out from the commencement of rock excavation as a safeguard against possible vibration induced damage. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a smaller rock hammer and/or drill and split methods.

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Maintain the rock hammer oriented towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate the hammer in short burst only, to reduce amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances and should be provided with a full copy of this report.

4.2.3 Temporary Cut Batter Slopes

All excavations should be temporarily battered back at no steeper than 1 Vertical (V) on 1 Horizontal (H) for stability considerations and to facilitate compaction of engineered fill (behind retaining walls) up against the cut faces. Based on the nature of the soil and extremely weathered bedrock profiles, we cannot justify steeper temporary cut batter slopes without soil nailing, which will be costly. Surcharge loads (including plant and stockpiles) must be kept well back from the crests of the temporary batter slopes. Competent bedrock, where encountered in the excavations, can be tentatively cut vertically subject to geotechnical inspection.

All temporary cut batter slopes must be inspected by an experienced geotechnical engineer to confirm that no untoward conditions exist. As required, the geotechnical engineer must nominate stabilisation measures (eg. flattening of cut slopes, rock bolts, shotcrete, etc.).

If temporary batter slopes need to extend into the neighbouring properties to the south and east, then permission must be sought from the respective owners. If permission is denied and/or temporary batter slopes are not preferred, then a cast insitu retention system, as discussed in Section 4.3.1 below, must be installed prior to the commencement of excavation.

4.2.4 Drainage

Groundwater inflows may occur as local seepage flows above the soil/bedrock interface, and through open fractures within the bedrock profile, particularly after heavy rain. We recommend that all cut faces and retaining walls incorporate spoon drains and/or subsoil drains to intercept potential seepage. Groundwater seepage inflows are expected to be controllable by gravity drainage methods. Discharge from the drainage system should be piped to the stormwater system once approval from the relevant authority has been obtained. The excavations will need to be monitored as it progresses by the builder, JK Geotechnics and the civil engineer (Northrop Consulting Engineers) to confirm the drainage requirements.

4.3 Excavation Retention

4.3.1 Retention Systems

Cantilever block retaining walls can be constructed along the toes of temporary cut batter slopes. If temporary batter slopes cannot be accommodated or are not preferred, then cast-insitu retention systems will need to be installed prior to the commencement of excavation. In this case, we recommend that the proposed vertical cuts in the soil and tonalite bedrock profiles be supported by either contiguous pile walls (in areas which are highly sensitive to lateral movement), or soldier pile walls with shotcrete infill panels elsewhere.

The piles must be progressively shotcreted. Based on the limited depths of the excavations, we expect that the piled walls will be designed as cantilevered structures. If deformations behind the piled walls are to be limited, then the walls will need to be progressively anchored as the excavation proceeds (ie. once the restraining point has been uncovered). For the anchored solution, further advice should be sought from JK Geotechnics.

The piles must be embedded well below bulk excavation level (including nearby footings and service trenches) at suitable depths to satisfy stability considerations.

Conventional bored piles can be used for the construction of the piled walls. Due to the presence of medium and high strength (and possibly stronger) tonalite, as well as core stones (boulders) within the soil and extremely weathered bedrock profiles, appropriate high torque drill rigs and equipment (eg. rock augers and coring buckets) should be brought to site. Penetration through core stones and the medium and high strength (and stronger) tonalite will be difficult. Consideration should therefore be given to the use of downhole hammer (percussive) drilling equipment. Each bored pile hole should be cleaned out, inspected by a geotechnical engineer, and poured on the same day as drilling. The prospective piling contractors should be provided with a full copy of this report so that appropriate drilling rigs and equipment (eg. rock augers, coring buckets, downhole hammer, etc.) are brought to site.

Construction of the piled walls must be of high quality. For soldier pile walls, the shotcrete infill panels must be completed without delay to reduce the shrinkage of clayey soils immediately outside the excavation, and the loss of sandy soils between the piles. Such shrinkage of clayey soils and loss of sandy soils could result in ground subsidence immediately behind the walls. The construction sequence should be fully specified and carefully controlled to reduce potential movements.

4.3.2 Retention Design Parameters

The major consideration in the selection of earth pressures for the design of the retention systems is the need to limit deformations occurring outside the excavation. The characteristic earth pressure coefficients and subsoil parameters provided below may be adopted for design.

Free-standing cantilevered block retaining walls as well as cantilevered piled walls (all independent of the proposed new buildings) should be designed using a triangular lateral earth pressure distribution with an 'active' earth pressure coefficient (K_a) of 0.35 for the soil and weathered bedrock profiles, assuming a horizontal retained surface. In areas where the ground behind the walls is sensitive to lateral movements, the walls should be designed using a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (K_0) of 0.55 for the soil and weathered bedrock profiles, assuming a horizontal retained surface.

A bulk unit weight of 20kN/m^3 should be adopted for the soil and weathered bedrock profiles.

Any surcharge loads (including compaction stresses during backfilling, inclined backfill, pedestrian traffic, nearby retaining walls and their backfill, etc.) affecting the walls should be allowed in the design using the appropriate earth pressure coefficient from above.

All retaining walls should be designed as permanently drained. For cantilevered block retaining walls, subsurface drains 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. For contiguous pile walls, weep hole outlets (also known as spitter pipes) should be provided between piles at the base of the excavation, at a horizontal spacing no greater than 1.2m, and should incorporate a non-woven geotextile filter fabric at the inserted end to reduce subsoil erosion. For soldier pile walls, a strip drain should be provided mid-span between adjacent piles (prior to shotcreting) with a basal weep hole outlet. All drainage water should be piped to the stormwater system.

The passive lateral toe resistance for retaining walls (cantilevered block and piled walls) founded in a combination of residual sands of at least medium dense relative density, residual clays of hard strength and weathered tonalite bedrock 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 soldier piles spaced at least three diameters apart, the passive restraint may be calculated using 3 x K_p to account for the three-dimensional case. The embedment depth design must take into account any nearby localised excavations in front of the wall, such as for building footings and service trenches. For piled walls, we recommend that the upper 0.3m depth of the socket should not be taken into account to allow for disturbance and tolerance effects during excavation.

If competent bedrock is encountered at the retaining wall footing base, then the wall could be secured against sliding by the provision of dowels, which must be designed in shear. Permanent dowels must be designed for corrosion resistance and for long-term durability (eg. stainless steel or hot dipped galvanised with protective sheathing).

4.4 Earthworks

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

Minor filling to a maximum height of about 0.5m will be required for the proposed Police Station building. The proposed fill will raise site levels at the north-eastern corner of the ground floor slab and will be used to shape the landscaping in front of the Police Station building.

4.4.1 Subgrade Preparation

Following the site preparation works detailed in Section 4.1.3, the soil subgrade in the fill and pavement areas should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 10 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 clayey soils have become 'saturated' and/or where under-compacted existing fill exists. Heaving areas should be locally removed to a stable base and replaced with engineered fill, as outlined 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.

4.4.2 Engineered Fill

General

Engineered fill must be used to raise site levels in order to reduce post-construction settlements. From a geotechnical perspective, site won clayey and sandy soils, including the extremely weathered tonalite, are considered suitable for reuse as engineered fill on condition that approval has been obtained from JK Environments, and that they are free of organic matter and contain a maximum particle size not exceeding 75mm. Excavated more competent tonalite bedrock will most likely require crushing in order to confirm to the maximum particle size requirement, and therefore should not be reused as engineered fill.

Engineered fill comprising site won materials should be compacted in maximum 250mm thick loose layers using a large pad-foot roller to a density ratio of at least 98% of SMDD and at a moisture content within 2% of SOMC.

If the earthworks contractor wishes to employ vibrating rolling for compaction, then vibration monitoring trials will need to be completed on the neighbouring house to the west, as discussed in Section 4.1.2. Notwithstanding, there will inherently be an exclusion zone adjacent to the western site boundary where static rolling will be required.

Edge Compaction

In order to achieve adequate edge compaction where fill platforms are proposed, we recommend that the outer edge of each fill layer extend a horizontal distance of at least 0.5m beyond the design geometry. The roller must extend over the edge of each placed layer in order to seal the batter surface. On completion of filling, the excess under-compacted edge fill should be trimmed back to the design geometry.

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.

Retaining Wall Backfill

As for services trenches, retaining wall backfilling must also be carried out using engineered fill in order to reduce post-construction settlements. Compaction of the engineered backfill should be carried out using a hand operated vertical rammer compactor for the lower layers and immediately behind the wall in the upper layers. Elsewhere a small static roller can be used. As per services trenches, backfilling should be carried out in 150mm thick loose layers and the maximum particle size of the backfill material should be no more than 50mm. The compaction specification provided above is applicable.

Compaction of engineered fill behind retaining walls is very difficult. The use of a single sized durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate (free of fines), which do not require significant compactive effort is often preferred if good performance is a priority; at least in the lower layers. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion. Provided the aggregate backfill is placed as recommended above, density testing would not be required. The geotextile should then be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of engineered fill.

If vibratory compaction is to be adopted for service trench and retaining wall backfill, then vibration monitoring trials will need to be carried out on the neighbouring house to the west.

Earthworks Inspection and Testing

Density tests must be carried out on the engineered fill to confirm the above specifications are 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.
- The frequency of density testing for retaining wall backfill (for material other than single sized aggregate) should be at least one test per two layers per 50m^2 (assumes maximum 150mm thick loose layers). Again, this implies that at each test location, two compacted layers will be tested simultaneously.

We consider that Level 2 testing of fill compaction, in accordance with Section 8 of AS3798-2007, is appropriate for this development. 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 earthworks contractor or service installation sub-contractors.

4.4.3 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 batter slopes may be required for lawn mowing.

Surface erosion protection, for example, quick establishing grass, turf or proprietary systems (such as those provided by Geofabrics Australasia, Global Synthetics, etc.) should be provided to the permanent batter slopes. Drainage (eg. kerbs and gutters, dish drains, etc.) should also be provided, where possible, along the crests of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system.

4.5 Footings

4.5.1 Site Classification to AS2870-2011

Based on the proposed cuts and fills, the presence of existing fill, which we can only be assumed to be 'uncontrolled', and the abnormal soil moisture conditions generated by tree removal and the demolition of the existing buildings and pavements, the site strictly classifies as Class 'P' in accordance with AS2870-2011 'Residential Slabs and Footings'. As such, the footings will need to be designed using engineering principles.

As a guide however, characteristic surface movements in the order of 40-60mm (ie. equivalent to a Class 'H1' site in accordance with AS2870-2011) should be anticipated for design purposes. This movement will be differential where a portion of the building is founded in (stable) weathered rock and the remainder in reactive clayey soils.

4.5.2 Geotechnical Design

We expect that weathered tonalite bedrock will be exposed at, or located a short distance below the proposed bulk excavation levels for each of the proposed buildings and carport. We therefore recommend that all building and carport footings be uniformly supported in extremely weathered (or better quality) tonalite bedrock. Ground floor slabs should be fully suspended off the footings.

Pad and strip footings, raft slab edge and internal beams, and block retaining wall footings uniformly founded in extremely weathered (or better quality) tonalite may be designed for an allowable bearing pressure of 600kPa. Similarly, conventional bored piles socketed at least 0.3m into extremely weathered (or better quality) tonalite may be designed for an allowable end bearing pressure of 600kPa. Pile sockets formed below the minimum 0.3m length requirement may be designed for allowable shaft adhesion values of 60kPa in compression and 30kPa in tension (uplift), on condition that the pile shaft is suitably roughened.

We forewarn that the presence of core stones within the soil and extremely weathered bedrock profiles will make footing excavation, particularly pile drilling, very difficult. Consideration should therefore be given to the use of downhole hammer (percussive) drilling equipment, as discussed in Section 4.3.1. Where possible, the piles should be designed in end bearing only.

Where cantilever block retaining walls penetrate existing fill are founded on a combination of residual sands of at least medium dense relative density, residual clays of hard strength and/or weathered tonalite bedrock, then their footings can be designed for an allowable bearing pressure of 150kPa. Where the foundation material comprises medium dense residual sands, the footing excavations should be at least 0.5m deep. Construction/expansion joints should be provided at about 3m centres and at, or near the change in foundation materials in order to accommodate differential shrink-swell movements.

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.

Pad, strip, beam and retaining wall footing excavations should be cleaned out, inspected (and DCP tested, as appropriate) by a geotechnical engineer (prior to the installation of reinforcement cages), and poured on the same day as excavation. If delays in pouring the footings are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering.

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, where in contact with the clayey soils, be poured over void formers. The void formers must be able to accommodate swell movements of about 30mm. Therefore, a minimum 50mm thick collapsible cardboard void former should be used. We strongly recommend that an inspection of the bulk excavation levels be carried out by an experienced geotechnical engineer to delineate the areas where void formers will be required; for example, below the northern portion of the proposed Police Station building, and possibly the northern portion of the vehicle storage facility. Void formers are not required where weathered bedrock is exposed at bulk excavation level.

In order to reduce rainwater sheeting flows off external walls from entering the subgrade, we recommend that all joints between the proposed new buildings and external concrete pavements/footpaths 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 'H1' site. All surfaces adjacent to the proposed buildings must be graded away to promote surface water run-off.

4.5.3 Earthquake Design Parameters

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

- Hazard Factor (Z) = 0.09
- Site Subsoil Class = Class B_e

4.5.4 Soil and Rock Aggression

The soil and rock aggression test results have indicated alkaline conditions, as well as low sulfate contents. In accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation' and Table 4.8.1 of AS3600:2018 'Concrete Structures', the exposure classification to concrete piles and footings is 'mild' and 'A2', respectively, as there is a potential for groundwater flows above the bedrock surface during and following rainfall events.

4.6 External Pavements

Based on the laboratory test results, we recommend that the proposed external pavements be designed for a CBR value of 4%, or a short-term Young's modulus of 25MPa for the compacted soil subgrade.

Concrete pavements 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 (TfNSW QA Specification 3051 unbound granular material) and compacted using a large smooth drum roller to a density ratio of at least 98% of Modified Maximum Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement 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.

For asphaltic concrete pavements, we recommend that the basecourse material comprises DGB20 (TfNSW QA Specification 3051). The basecourse material should be compacted using a large smooth drum roller to a density ratio of at least 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 (TfNSW QA Specification 3051). The sub-base material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to a density ratio of 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 along the southern, eastern and western (high) sides of the proposed pavement areas, 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.

4.7 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. Additional cored boreholes to further assess the depth and quality of the competent tonalite bedrock so that excavation contractors can more accurately predict the cost of the works;
2. Dilapidation survey report on 14 Thredbo Terrace;
3. Piled wall inspections;
4. Vibration monitoring;
5. Groundwater monitoring of seepage inflows;
6. Inspection of all temporary cut batter slopes;
7. Proof-rolling inspections;
8. Density testing of all engineered fill and granular pavement materials to Level 2 control by a GTA;
9. Inspection of the bulk excavations to assess where void formers are required;
10. Pad, strip, beam and pile footing inspections;
11. Retaining wall footing inspections, and DCP testing, as appropriate.

5 GENERAL COMMENTS

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

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

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

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

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

SOIL TEST SERVICES

ABN 43 002 145 173

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

Client:	JK Geotechnics	**Ref No:**	33612A
Project:	Proposed New Police Station	**Report:**	A
Location:	16 Thredbo Terrace, Jindabyne, NSW	**Report Date:**	11/12/2020
		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 %
1	3.00 - 3.50	2.9	-	-	-	-
3	0.50 - 0.95	11.4	50	21	29	10.5
3	2.50 - 3.00	7.9	-	-	-	-
3	4.50 - 5.00	5.6	-	-	-	-
3	5.00 - 5.10	0.6	-	-	-	-
4	0.50 - 0.95	11.0	28	18	10	4.5
4	4.00 - 4.50	2.2	-	-	-	-
4	5.00 - 5.50	2.4	-	-	-	-

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: 30/11/2020.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

11/12/2020
Authorised Signature / Date
(D. Treweek)

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

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	33612A
Project:	Proposed New Police Station	**Report:**	B
Location:	16 Thredbo Terrace, Jindabyne, NSW	**Report Date:**	14/12/2020
		Page 1 of 1	

BOREHOLE NUMBER	BH 3	BH 4
DEPTH (m)	0.13 - 1.20	0.50 - 1.50
Surcharge (kg)	4.5	4.5
Maximum Dry Density (t/m^3)	1.83 STD	1.95 STD
Optimum Moisture Content (%)	12.9	10.9
Moulded Dry Density (t/m^3)	1.79	1.90
Sample Density Ratio (%)	98	98
Sample Moisture Ratio (%)	99	102
Moisture Contents		
Insitu (%)	11.1	9.9
Moulded (%)	12.8	11.1
After soaking and		
After Test, Top 30mm(%)	24.6	14.8
Remaining Depth (%)	19.5	13.3
Material Retained on 19mm Sieve (%)	0	0
Swell (%)	3.5	0.0
C.B.R. value:		
@5.0mm penetration	4.0	18

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: 30/11/2020.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
This document shall not be reproduced except
In full without approval of the laboratory. Results relate only to
the items tested or sampled.

14/12/2020

Authorised Signature / Date
(D. Treweek)

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

CERTIFICATE OF ANALYSIS 256957

Client Details

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

Sample Details

Your Reference	**33612A, Jindabyne**
Number of Samples	4 Soil
Date samples received	30/11/2020
Date completed instructions received	30/11/2020

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.

Report Details

Date results requested by	07/12/2020
Date of Issue	07/12/2020

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		256957-1	256957-2	256957-3	256957-4
Your Reference	UNITS	BH1	BH2	BH4	BH4
Depth		0.5-0.95	0.5-0.95	0.13-0.5	1.5-1.95
Date Sampled		23/11/2020	23/11/2020	24/11/2020	24/11/2020
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	01/12/2020	01/12/2020	01/12/2020	01/12/2020
Date analysed	-	01/12/2020	01/12/2020	01/12/2020	01/12/2020
pH 1:5 soil:water	pH Units	8.0	7.9	8.0	7.3
Sulphate, SO4 1:5 soil:water	mg/kg	<10	20	10	<10

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-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			01/12/2020	1	01/12/2020	01/12/2020		01/12/2020	[NT]
Date analysed	-			01/12/2020	1	01/12/2020	01/12/2020		01/12/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	8.0	8.0	0	102	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	108	[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.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1/1

Client: GROUP GSA
Project: PROPOSED NEW POLICE STATION
Location: 16-18 THREDBO TERRACE, JINDABYNE, NSW

Job No.: 33612A **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 932.6m
Date: 23/11/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** A.C.K./A.J.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND AFTER 1 HR	ES		0		FILL: Gravel, fine to medium grained, igneous, dark grey, partially bound.	M			
				SC	Clayey SAND: fine to medium grained, light brown, trace of fine to medium grained tonalite gravel.	M	MD		RESIDUAL
	ES, DB	N = 31 8,9,22		-	Extremely Weathered tonalite:clayey SAND, fine to coarse grained, light grey and dark grey, trace of fine to medium grained feldspar inclusions.	XW	D		JINDABYNE TONALITE
	DB		1						
	DS	N = SPT 9/50mm REFUSAL							
			2		TONALITE: fine to coarse grained, light grey, dark grey and brown, with fine to medium grained feldspar inclusions and medium strength iron indurated bands.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
							L		LOW RESISTANCE
	DS		3						
						SW	H		HIGH RESISTANCE
			4		END OF BOREHOLE AT 3.8m				'TC' BIT REFUSAL
			5						
			6						
			7						

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1/1

Client: GROUP GSA
Project: PROPOSED NEW POLICE STATION
Location: 16-18 THREDBO TERRACE, JINDABYNE, NSW

Job No.: 33612A **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 931.4m
Date: 23/11/20 **Datum:** AHD
Plant Type: JK205 **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			CONCRETE: 180mm.t				8mm DIA. REINFORCEMENT 140mm TOP COVER
	ES				-	FILL: Silty sandy clay, low plasticity, dark brown, fine to medium grained sand, with fine to coarse grained igneous gravel.	w>PL			
	ES, DS	N = 17 4,8,9	1		SC	Clayey SAND: fine to coarse grained, dark grey, light grey and brown, with fine to medium grained tonalite gravel.	M	MD		RESIDUAL
	DS	N = SPT 9/50mm REFUSAL			-	Extremely Weathered tonalite: clayey SAND, fine to coarse grained, light grey, dark grey and brown.	XW	(D)		JINDABYNE TONALITE VERY LOW TO LOW 'TC' BIT RESISTANCE
			2			END OF BOREHOLE AT 1.6m				'TC' BIT REFUSAL
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1/1

Client: GROUP GSA
Project: PROPOSED NEW POLICE STATION
Location: 16-18 THREDBO TERRACE, JINDABYNE, NSW

Job No.: 33612A **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 933.1m
Date: 24/11/20 **Datum:** AHD
Plant Type: JK205 **Logged/Checked by:** A.C.K./A.J.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		CONCRETE: 130mm.t				6mm DIA. REINFORCEMENT 95mm TOP COVER
	N = 21 7,8,13	1	CI	Sandy CLAY: medium plasticity, brown mottled light grey, fine to medium grained sand, trace of fine to medium grained tonalite gravel.	w<PL	Hd	>600 >600 >600	RESIDUAL
	N > 29 14,20, 9/50mm REFUSAL		-	Extremely Weathered tonalite: sandy CLAY, low to medium plasticity, light grey, dark grey and brown, fine to medium grained sand.	XW	Hd	>600 >600 >600	JINDABYNE TONALITE VERY LOW 'TC' BIT RESISTANCE
		2, 3, 4, 5		TONALITE: fine to coarse grained, light grey, dark grey and brown, with extremely weathered bands.	DW	VL-L		LOW RESISTANCE
				TONALITE: medium to coarse grained, light grey.	SW	H		HIGH RESISTANCE 'TC' BIT REFUSAL
		6, 7		END OF BOREHOLE AT 5.1m				

Samples columns: ES, U50, DB, DS

JKGeotechnics

BOREHOLE LOG

Borehole No. **4**

Client: GROUP GSA

Project: PROPOSED NEW POLICE STATION

Location: 16-18 THREDBO TERRACE, JINDABYNE, NSW

Job No.: 33612A **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 929.2m

Date: 24/11/20 **Datum:** AHD

Plant Type: JK205 **Logged/Checked by:** A.C.K./A.J.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND AFTER 1 HR		0		CONCRETE: 130mm.t				NO OBSERVED REINFORCEMENT
			-	FILL: Sandy clay, low to medium plasticity, dark brown, fine to medium grained sand, trace of fine grained igneous gravel.	w≈PL			
	N = 19 5,9,10	1	CL	Sandy CLAY: low plasticity, dark brown mottled light grey and orange brown, fine to medium grained sand.	w<PL	Hd		RESIDUAL TOO FRIABLE FOR HP TESTING
	N = 33 21,9,24 N = SPT 13/100mm REFUSAL	2 3	-	Extremely Weathered tonalite: SAND, fine to coarse grained, light grey, dark grey and brown, with silt fines and very stiff to hard clay bands.	XW	D	360 420	JINDABYNE TONALITE VERY LOW 'TC' BIT RESISTANCE LOW RESISTANCE WITH VERY LOW BANDS
		4 5		TONALITE: fine to coarse grained, light grey and dark grey mottled orange brown.	DW	VL L		LOW RESISTANCE
				as above, but light grey and dark grey.	SW	H		HIGH RESISTANCE
		6 7		END OF BOREHOLE AT 5.7m				'TC' BIT REFUSAL

JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	GROUP GSA
Project:	PROPOSED NEW POLICE STATION
Location:	16-18 THREDBO TERRACE, JINDABYNE, NSW

Job No.	33612A	Hammer Weight & Drop: 9kg/510mm
Date:	23-11-20	Rod Diameter: 16mm
Tested By:	A.C.K.	Point Diameter: 20mm

Test Location	5	6	7				
Surface RL	≈927.5m	≈931.5m	≈934.0m				
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	7	2	3				
100 - 200	3	9	1				
200 - 300	4	9	3				
300 - 400	5	6	2				
400 - 500	6	4	4				
500 - 600	9	5	4				
600 - 700	15	6	5				
700 - 800	14	7	6				
800 - 900	11	7	6				
900 - 1000	11	12	8				
1000 - 1100	21	14	9				
1100 - 1200	20	11	10				
1200 - 1300	35	5	15				
1300 - 1400	END	6	19				
1400 - 1500		7	9				
1500 - 1600		6	9				
1600 - 1700		7	12				
1700 - 1800		8	21				
1800 - 1900		10/80mm	30				
1900 - 2000		REFUSAL	27				
2000 - 2100			END				
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

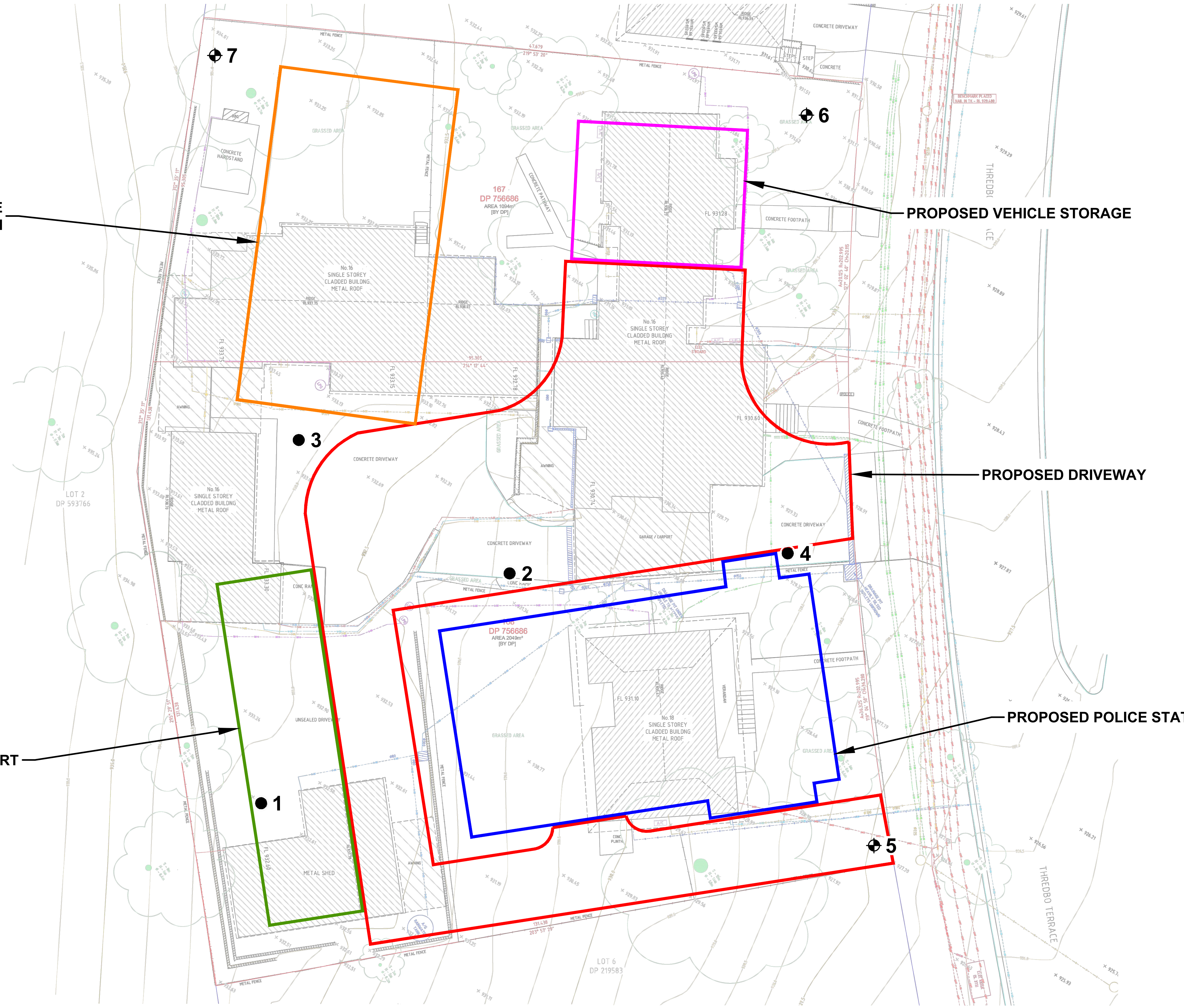
AERIAL IMAGE SOURCE: MAPS.SIX.NSW.GOV.AU

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

Title:	**SITE LOCATION PLAN**
Location:	16-18 THREDBO TERRACE, JINDABYNE, NSW
Report No: 33612A	Figure No: 1

JKGeotechnics

PLOT DATE: 15/12/2020 8:50:04 AM DWG FILE: Y:\33000'S\33612A JINDABYNE\CAD\33612A.DWG



LEGEND

- BOREHOLE
- DCP TEST

SCALE 1:300 @A3 METRES (0, 3, 6, 9, 12, 15)

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

Title: **TEST LOCATION PLAN**

Location: 16-18 THREDBO TERRACE, JINDABYNE, NSW

Report No: 33612A

Figure No: 2

JKGeotechnics

PLOT DATE: 15/12/2020 8:49:46 AM DWG FILE: Y:\33000's\33612A JINDABYNE\CAD\33612A.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

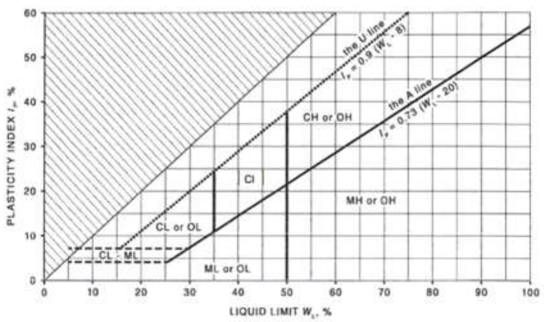
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	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)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)** VERY LOOSE ≤ 15 0 – 4 LOOSE > 15 and ≤ 35 4 – 10 MEDIUM DENSE > 35 and ≤ 65 10 – 30 DENSE > 65 and ≤ 85 30 – 50 VERY DENSE > 85 > 50 Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		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.
Extremely Weathered		XW		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.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	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.
Moderately Weathered		MW		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.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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