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INTERPRETATIVE REPORT

TO

UNIVERSITY OF TECHNOLOGY, SYDNEY

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED BUILDING 02 BASEMENT EXTENSION, ASRS AND THOMAS STREET BUILDING

AT

CNR JONES AND THOMAS STREETS, ULTIMO, NSW

28 January 2011

Ref: 24546SPrpt2

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EXECUTIVE SUMMARY

University of Technology, Sydney (UTS) commissioned Jeffery & Katauskas Pty Ltd (J&K), to undertake a geotechnical investigation for the proposed Building 02 Basement Extension project which will incorporate an Automated Storage and Retrieval System (ASRS). The excavation will be below the southern corner of the Alumi Lawn near the corner of Thomas and Jones Streets, Ultimo, NSW. Our environmental division, Environmental Investigation Services (EIS) has prepared an Environmental Site Assessment Report (Ref: E24546Krpt) dated January 2011 which also included a waste classification of the site soils.

The proposed development includes the excavation of a basement extending to finished floor reduced levels (RL) of RL-2.1mAHD and RL-5.0mAHD. Bulk excavation to achieve these design levels will extend to a maximum depth of approximately 20.5m below the existing surface levels.

The boreholes have generally revealed a subsurface profile comprising a variable thickness of fill and residual silty clays over weathered shale and then sandstone bedrock. Groundwater levels measured in PVC standpipes were inconclusive, but possibly relate to a groundwater level in the order of RL7mAHD to RL10mAHD. Based upon observation of the existing Building 02 basement cuts and nearby projects, groundwater inflows are expected to be minor.

Based on the investigation results, the principal geotechnical issues associated with the proposed development will be the shoring of the soil, shale and more weathered sandstone at the top of the excavation faces, dewatering of the excavation and assessing ground movements outside the excavation, in particular with regard to the footings for the existing Building 02 and the Multi-Purpose Sports Hall (MPSH) currently under construction, which contains excavation to about RL3.0mAHD.

It is also possible that there will be some rubble within the backfilled basement of a previous building which fronted Jones Street to the south-west, and possibly some old footings from that development. The drilling of retention piles may be detrimentally affected by the presence of buried infrastructure and old footings.

There were a significant number of inclined joints observed within existing cut faces around the perimeter of the proposed excavation, and so there is the potential for unstable wedges of sandstone to be exposed, which may necessitate rock bolting or over-excavation. The most likely area requiring stabilisation by rock bolting would be along the north-eastern and south-eastern faces, though the south-eastern face will be of much lower height. There will be a significantly lower risk of stabilisation being required on the north-western and north-eastern faces.

The bulk excavations will extend below the groundwater table, however we expect that flows through the bedrock will be relatively low, and so disposal of the seepage will be required. The EIS report recorded traces of petroleum hydrocarbons and chloroform in samples of the groundwater, and therefore it will be necessary to negotiate disposal with Council closer to the time of construction. The preferred disposal option would be to the stormwater system. However, the water quality will have to satisfy any conditions imposed by Council. Alternative disposal options (in order of expense) would be to the sewer as Trade Waste or to tankers.

The bedrock underlying the site will provide appropriate support for the envisaged building loads, and will allow the use of pad and strip footings below basement level.

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1 INTRODUCTION

This interpretative geotechnical report has been prepared for the proposed University of Technology, Sydney (UTS) Building 02 Basement Extension, Automated Storage and Retrieval System (ASRS) project located at the Ultimo campus of UTS. This report was commissioned by Mr Berlin Ng of UTS (Purchase Order Number 48605, dated 13 October 2010) on the basis of the Consultancy Agreement between UTS and Jeffery & Katauskas Pty Ltd.

As a later addition to the contract, we were also asked to drill an additional three boreholes along the Thomas Street side of the site so that preliminary recommendations could be provided for the proposed future Nexus Building; details of this building were not available at the time of reporting, though we expect this will comprise a multi-level concrete framed building over at least one basement level, and that construction will be undertaken following the completion of the basement extension and ASRS project.

The geotechnical and environmental investigations were carried out between the 6th and 21st December 2010. The factual results of the geotechnical investigation are presented in our Factual Report (Ref. 24546SPrpt) dated 28 January 2011. Our environmental division, Environmental Investigation Services (EIS) has prepared an Environmental Site Assessment Report (Ref: E24546Krpt) dated January 2011 (which also included a waste classification of site soils). The reader is referred to these reports for additional information.

Based on the provided architectural plans listed in the Table in Appendix A, we understand the proposed development comprises the an excavation of about 70m by 33m to a reduced level of -2.1m AHD (which relates to a maximum depth of the order of 17.4m below the existing surface levels). This proposed excavation will extend to the northern cut face of the existing Building 02 basement to the south-east, to within about 3m from the Jones Street frontage to the south-west, to the modified basement access ramp to the north-east, and within about 24m from the Thomas Street frontage to the north-west.

The western portion of the basement will house an Automated Storage and Retrieval System (ASRS), while in the eastern part of the basement there will be large storage rooms as well as plant rooms and smaller storage and operational areas. In the proposed ASRS area, the basement will be extended deeper, to RL -5.0mAHD (about 20.2m depth); the north-eastern, north-western and south-western faces of this deeper excavation are coincident with the basement excavation above, while the southern face of this excavation will be set about 5m off the north-western face of the existing Building 02 basement excavation.

We understand from the brief that the proposed column loads for the building within this excavation are expected to be of the order of 3500 kN.

The purpose of this report is to provide comments, recommendations and detailed geotechnical design advice on excavation, retention, de-watering of the excavation, footing design, site classification for earthquake design, pavement design and geotechnical constraints.

There is also an early works package relating to proposed alterations to the Building 01 and 02 basement access ramps which comprises widening the existing ramp to provide truck access to the loading area on the upper basement level (Level 02) at RL8.5mAHD, and providing a new steeper vehicle access to the lower basement parking area (Level 1.5) at RL 5.75mAHD. We presume these widening works will comprise cut and cover construction techniques.

2 GEOTECHNICAL AND ENVIRONMENTAL ISSUES AND CONSTRAINTS

The boreholes have generally revealed a subsurface profile below the Alumni Lawn comprising fill over residual silty clay soils and then weathered shale and sandstone bedrock. It appears that groundwater will be present at a reduced level of about RL7mAHD to RL10mAHD, though the bedrock is expected to be relatively tight based upon information from this and previous nearby projects, and so relatively low flows are expected. Further details are presented in our Factual Report (Ref. 24387WRrpt) dated 14 January 2011.

Based on the investigation results, the principal geotechnical issues associated with the proposed development will be:

- The stability of the fill, residual soil, and upper weathered shale and sandstone;
- Excavation of the predominantly high strength sandstone bedrock;
- The potential for excavation to cause distress to the adjacent developments, infrastructure and services;
- Limiting ground surface movements outside the excavation;
- Progressive assessment of the excavation faces to determine any necessary stabilisation measures for potentially loose blocks of sandstone; and
- The design and constructions of footings for the proposed development.

Design considerations for these issues are presented in the following sections of the report.

Some additional issues and considerations will be:

Old Footing Systems and Infrastructure: There is the distinct likelihood that traces of old footing systems and basement retaining walls will be present where the previous Jones Street Building stood. The drilling of shoring piles may be detrimentally affected by the presence of such features.

There are several existing services which enter the site, predominantly from the Thomas Street frontage. These will need to be diverted or capped prior to commencing excavation.

Disposal of Groundwater Seepage: As discussed in the EIS report, there were traces of petroleum hydrocarbons and chloroform present in the groundwater samples. Negotiation will be needed with Sydney City Council prior to disposal of this seepage to the stormwater (by pumping from sumps). If permission cannot be gained for disposing of this seepage to the stormwater, it may be necessary to dispose to the sewer as trade waste, or in the short term to transport it from site in trucks. The inflow rates are expected to be quite low.

Permanent Support of Rock Face: the majority of the excavation face will be in sandstone bedrock which is expected to be largely self supporting. However, it is also foreseeable that there will be potentially unstable loose blocks of sandstone exposed during the excavation which require stabilisation. If these features cannot be braced by the building in the long term, then permanent rock bolts will be required, and we understand these would require a design life of 150 years. Such bolts will be expensive and slow to install and could lead to delays in the excavation process.

3 COMMENTS AND RECOMMENDATIONS

3.1 Excavation

3.1.1 General

The adjacent Buildings 01 and 02 and paved surfaces lie on or close to the site boundaries. During excavation, there is the potential to damage or de-stabilise these neighbouring buildings and paved surfaces (including any buried services).

Excavation will therefore need to be completed carefully using suitably experienced (and insured) contractors.

It is unknown at this stage whether the south-western basement wall of the Jones Street building was removed during demolition, and this could have an impact on the type and design of a shoring system along that boundary. For example, if that basement wall and footings have been left in place close to that boundary, it may be necessary to locally excavate to remove the footings prior to backfilling and installing shoring piles. We therefore recommend several test pits be excavated along the Jones Street boundary of the site to ascertain whether the basement wall was removed as part of the demolition prior to the commencement of piling.

3.1.2 Demolition and Excavation

Site access constraints are not expected to limit the size of plant to be used at the site. On this basis, we expect the surface excavations to be completed using hydraulic tracked excavators up to say 40 tonne size.

Excavation recommendations provided below should be complemented by reference to the Code of Practice *'Excavation Work'*, Cat No 312 (31 March 2000), by WorkCover NSW.

The proposed excavation for the basement extension will extend to depths ranging between 17.5m and 20.5m. These excavations will extend through the fill and residual soils, weathered shale and sandstone, with the majority of the excavation being through high strength sandstone bedrock.

The excavation of the fill (Unit 1) and natural silty clay (Unit 2) should be readily achievable using bucket attachments to the above mentioned tracked excavators. The Unit 3 shale and some of the Unit 4 sandstone could be removed using ripping tynes on the excavators, while the portions of the Unit 4 sandstone of medium strength or higher is likely to require the use of hydraulic rock breaker attachments to the excavators, or a combination of sawing with large diameter rock saws and ripping. The Unit 5 sandstone will be of predominantly high strength, with relatively few horizontal defects, and this will also require the use of large excavator mounted rock hammers, large rock saws or rock grinder attachments to the excavators. Alternatively, it is likely that much of the rock could be ripped with dozers of D10 size, with some rock hammering of higher strength bands. Allowance should be made for relatively low productivity for the excavation of Unit 5, and also for full time monitoring of the vibrations within Buildings 01 and 02.

3.1.3 Potential Vibration Risks

Where rock breakers are used, we recommend that the rock breaker be continually orientated towards the face and be operated in short bursts only to reduce amplification of vibrations. Grid saw cuts around the perimeter and through the proposed excavation area may also assist in controlling vibrations.

When using the rock breakers or concrete saws, the resulting dust should be suppressed by spraying with water.

As large excavation equipment will be required for the practical excavation of the rock, we recommend that full-time quantitative vibration monitoring of adjacent structures be undertaken while the rock breakers are being used to confirm that peak particle velocities fall within acceptable limits. The monitoring equipment should include geophones capable of measuring vibration in all three dimensions, and provide a vector sum of the vibrations. The monitoring equipment must also contain an audible or visible alarm which warns when allowable vibration levels are exceeded. Subject to viewing the dilapidation reports mentioned below, we recommend that the peak particle velocities along the site boundaries do not exceed say 5mm/sec. We note that this vibration limit will reduce the risk of vibration damage to the neighbouring buildings and structures. However, these vibrations may still result in discomfort to occupants of the neighbouring buildings. If potentially damaging vibrations are occurring, it will be necessary to use lower energy equipment such as smaller hammers. Alternatively, as mentioned above, grid-sawing techniques can be used to dampen ground vibrations.

3.1.4 Dilapidation Survey Reports

Prior to demolition and excavation commencing, we recommend that detailed dilapidation reports be compiled on the lower levels of the north-western side of Building 02 and the western corner of Building 01. These reports can then be used as benchmark to show construction activities are not causing any distress which may be noted in the buildings.

3.1.5 Seepage

Based on the results of the investigation, and experience from nearby sites such as the Multi-Purpose Sports Hall (MPSH) and the Frasers site to the south of Broadway, minor groundwater seepages would be expected to occur, particularly from the soil rock interface, and through defects within the rock mass. However, we expect that seepage would be readily controlled using conventional sump and pump techniques. Reference should be made to the basement drainage section below for estimates of the volumes to be pumped from the excavation.

3.1.6 Geotechnical Inspections

Where rock faces are being cut vertically, it will be necessary for inspections to be undertaken by the geotechnical engineers at no more than 2m height intervals. The purpose of the inspections is primarily to look for any defects which due to their orientation may cause potentially unstable blocks or wedges of sandstone in the rock faces. The presence of such blocks or wedges is most likely in the face adjacent to the proposed access ramps to the north-east. Unless such blocks can be removed, it will be necessary to provide support such as rock bolts. Where the blocks may be braced from the structure in the long term, temporary rock bolts may be used, while permanent bolts would be required where this cannot occur. Further details of permanent rock bolts and rock anchors are provided below.

3.1.7 Trafficability

The residual silty clay had a very low soaked CBR value (1.0%), confirming these soils lose strength rapidly on contact with water. The existing fill and weathered shale is similarly likely to soften significantly on contact with water. Therefore, there could be significant trafficability problems following rain.

There will be areas such as the site entry where it will be important to maintain access after wet weather, and in these areas, consideration should be given to providing a 0.3m thick layer of select granular material such as crushed concrete to improve access. An even higher confidence in performance following wet weather can be achieved by placing a layer of geogrid such as Tensar SS20 on the surface prior to placing the granular fill.

3.2 Retention

3.2.1 Boundary Conditions and Batter Slopes

During the early works on the basement access ramps, we expect that the soils on the south-western side of the ramps will be battered (the other side of the ramp area has already been excavated for the previous MPSH and Building 04 developments).

Also, it is likely that temporary batters will be used along the north-western side of the proposed basement extension.

Along the south-eastern side of the proposed basement, the excavation for the adjacent development has already extended to approximately RL0.5mAHD, and the additional excavation along that side of the proposed basement will be in sandstone bedrock which would generally be considered to be self-supporting.

Along the south-western side of the proposed basement extension, and along the north-western side of the future Nexus Building, the soils and upper weathered bedrock will need to be supported by a shoring system as there is insufficient space to form batter slopes within the boundary (assuming for the Thomas Street building basement extends close to the street frontage).

Along the majority of the north-eastern boundary of the proposed basement, adjacent to the early works package access ramps, the new access ramps over most of the length of the basement will extend onto the sandstone bedrock. However, particularly the northern end of these ramps may be founded on the Unit 4 sandstone containing significant seams, and the portion adjacent to the proposed Nexus Building will be founded on the weathered shale and residual silty clay. Therefore, when the level of excavation reaches the base of these ramps, the excavation should be undertaken with close geotechnical inspection so the rock quality can be inspected; some additional underpinning of the new ramps and permanent stabilisation of the rock face by rock bolted reinforced shotcrete faces may be required.

Batter slopes in the fill, residual soils and extremely low strength rock should be battered at no steeper than 1 Vertical (V) in 1 Horizontal (H) for the temporary case, and 1V in 2H for the permanent case.

Batters in the weathered rock of very low and low strength should be limited to 1V in 0.5H for heights to 2.5m, and 1V in 1H for heights of greater than 2.5m, for both the temporary and permanent conditions. Rock of medium strength or stronger may be assumed to be self-supporting, subject to geotechnical inspection during the excavation.

Surcharge loads such as from plant or building materials must be kept well clear of the crests of these batters. Permanent batters should be protected against erosion such as by shotcrete panels pinned to the face with appropriate drainage behind the panels.

3.2.2 Retention Methods

We understand that UTS generally prefers buildings to be constructed within free standing basements such that the shoring does not rely on the building for support. However, along portions of the Jones Street boundary, where the proposed basement extension will extend to within about 3m of the boundary, the conditions disclosed in BH201 will require support to a depth of about 8m. There would be insufficient room to then construct a gravity retaining wall between the proposed basement excavation line and the site boundary, and the construction of the retaining wall would anyway require shoring along the street frontage.

A similar situation occurs along the Jones and Thomas Street sides of the proposed future Nexus Building, where the construction of a basement will require shoring of the soils along the boundary.

In both of these locations, it will be necessary to install shoring along or near the boundary, and to use temporary anchors for short term support and bracing from the structure for long term support; alternatively permanent rock anchors for support would require the formation of easements for support within the adjacent road corridors which we expect would be difficult or impossible to negotiate. The shoring should be designed to support everything above the Unit 5 sandstone.

Apart from the shoring in these areas, there may be the need for retaining walls in other areas of the site, such as possibly along the north-western side of the proposed basement until the Nexus Building is built (unless a permanent batter will be left along this side of the basement).

Design earth pressures for these retaining walls are provided in the following section of this report. Appropriate surcharge loads should also be added to these design pressures, and drainage should be installed behind these faces to permanently drain pore pressures.

Ground anchors for temporary support will extend beyond the site boundaries and permission from Council will be required prior to the installation of the anchors. Such anchors should be designed in accordance with the parameters provided in the following section of this report.

Construction of the retaining system and anchors should be of a high quality and only experienced contractors should be employed.

3.2.3 Retention Design Parameters

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. In addition, the stiffness of the retention system will also have a significant impact on the control of deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of temporary or permanent systems to retain the existing soils.

- In these conditions, a conventional soldier pile wall with reinforced shotcrete infill panels would be suitable. Such shoring should be designed for a trapezoidal lateral earth pressure with a maximum lateral earth pressure of 6H kPa, where H is the height of material to be shored in metres. This maximum pressure should apply over the central 60% of the height of the shoring, reducing to zero at the crest and toe of the shoring. Where the shoring will be supporting rock of at least very low strength (but which requires shoring due to seams and joints), the maximum lateral earth pressure may be reduced to 4H for that portion of the shoring.
- For progressively anchored or propped walls which retain areas highly sensitive to lateral movement (such as where there are settlement sensitive structures or services within say 10m of the excavation), a uniform rectangular earth pressure distribution of 8H kPa should be adopted.
- Any surcharge affecting the walls (e.g. due to traffic, adjacent footings, construction loads, etc) should be allowed for in the design using the appropriate 'at rest' earth pressure coefficients (K_o), reported in the 'Retaining Wall Design Parameters' table, below.
- For piles embedded into the sandstone bedrock of at least medium strength, an allowable lateral toe resistance of 300kPa may be adopted for the short term while the anchors are being installed; the lateral restraint will be lost when excavation extends past this socket.
- The anchors should have minimum free lengths and bond lengths of 4m and 3m respectively, and the bond length must be entirely behind a line drawn upward at 1V in 1H from the base of the shoring. Anchors founded within the shale or sandstone of at least very low strength may be designed for an allowable bond of 100kPa, while bonds of 250kPa and 500kPa may be adopted for the medium and high strength sandstone respectively.
- All anchors should be proof tested to at least 130% of their working load, and then the lock-off load must be recorded. Lift-off tests should then be completed

on at least 25% of these anchors approximately three days following lock off. If the anchors have lost more than 10% of their lock-off load, then additional lift-off tests should be completed after a further three days. If further losses are recorded, then all anchors should be tested, and remediation or replacement of the unacceptable anchors undertaken.

Should soil/structure interaction programs (such as program Wallap) be used for the design of the shoring systems, the following parameters should be adopted.

RETAINING WALL DESIGN PARAMETERS									
Material	**Bulk Density kN/m^3**	**E' MPa**	**Eu MPa**	**Poissons Ratio**	**Effective Friction Angle**	**Effective Cohesion kPa**	**Ka**	**Ko**	**Kp**
Unit 1 Fill	18	15	15	0.3	25°	0	0.41	0.58	2.46
Unit 2 Residual Silty Clay	20	50	50	0.3	30°	2	0.33	0.50	3.00
Unit 3 Weathered Shale	21	100	100	0.3	30°	2	0.33	0.50	3.00
Unit 4 Class III to V Sandstone	22	200	200	0.3	32°	20*	0.31	0.53	3.25
Unit 5 Sandstone	23	1,000	1,000	0.2	40°	1,000*	N/A	N/A	N/A

*: The effective cohesion values for the rock mass should be assessed in relation to the likely presence of defects within the rock mass, in particular the defect characteristics and shear strength.

3.2.4 Excavation Related Ground Movements Including Stress Relief

It is likely that the excavation will induce some movements of the adjacent ground that falls within the area of influence of the excavation. The extent of influence can be defined as extending a horizontal distance back from the excavation perimeter equal to at least 1.5 times the excavation depth. In clays, lateral movements even for relatively stiff cantilevered walls (construction of good workmanship), could possibly be of the order of 1% of the excavated depth. Precedence suggests that for propped or anchored walls which are designed on the basis of the uniform lateral earth pressure of 8H kPa lateral and vertical movements will probably be close to 0.1% of the shoring depth.

The actual wall movements are highly dependent on the construction sequence, detailing and quality of installation and should be closely monitored in critical areas.

There is a relatively high horizontal in-situ rock stress within the Sydney region (commonly of the order of 1MPa to 3MPa), and excavations into the rock release these stresses and cause subsequent deflection; the magnitude of these stresses are such that these movements cannot be overcome by shoring/anchoring. These stress relief movements are in addition to the movements of shoring systems supporting the near surface soils and weathered rock. The usual range of stress relief for excavations into the sandstone bedrock within Sydney is between 0.5mm and 1.0mm per metre depth of excavation into the rock. As the Building 01 and 02 basements already extend to a reduced level of about 0.5mAHD, significant relief of the locked-in stresses is likely to have already occurred above the existing basement level. Therefore a realistic prediction of stress relief movement could probably adopt 0.5mm/m depth of excavation above RL0.5mAHD, and 1.0mm/m below RL0.5m AHD. Therefore, we expect stress relief movements would be limited to about 12mm, with only about 5-6mm of lateral movement being expected below the adjacent Building 01 and Building 02 footings. We would expect the vertical settlement associated with the stress relief to be about half of the lateral movement.

Further detailing of the stress relief movements and vertical settlement associated with the relief of the high in-situ horizontal stress would require detailed finite element analysis which was beyond the scope of this investigation. For accurate modelling the measurement of in-situ stresses would also be necessary but would be very costly.

3.2.5 Permanent Rock Bolt and Rock Anchor Details

Where rock bolts or rock anchors are required to supply support for the life of the project, which the brief nominates as 150 years, the bolts and anchors will need careful detailing and very good workmanship. Such anchors and bolts will need to be fully encapsulated within a corrugated polyethylene sheath with double grouting of the anchors and bolts. The head details must also provide corrosion protection, such as by having the entire head of the anchor or bolt encapsulated within a grease filled box which still allows future inspection and maintenance of the anchors/bolts. We consider that these features should be inspected on a 30 to 50 year basis.

A specification for the installation of rock anchors and rock bolts is provided in the attached Appendix C, together with diagrams showing suitable corrosion protections measures.

3.3 Basement Drainage

We have undertaken modelling of the groundwater inflows to the proposed basement using the computer based two-dimensional seepage software 'seep/w'. As mentioned in our factual report, an equivalent mass permeability of 10^{-8}m/sec would generally be considered suitable for the Class I and II sandstone which forms Unit 5. However, it is also likely that the equivalent permeability will be lower than this based upon observation of nearby excavations, and that 10^{-9}m/sec or 10^{-10}m/sec may be more appropriate.

Our models have adopted a groundwater level at RL10mAHD, with the base and sides of the excavation being free draining. The results from the model with a permeability of 10^{-8}m/sec within the bedrock are shown on Figure B1 in Appendix B, and these show that the amount of drawdown in the water table adjacent to the basement in the long term would be to approximately RL3m AHD. As this drawdown is predominantly with Class I and II sandstone, we consider that it will have no noticeable effect on adjacent structures. The flux provided by the model of $6.3 \times 10^{-7} m^3/sec$ would relate to an inflow rate of 160 litres/hour, or about one quarter to one eighth the flowrate of a typical garden tap.

As mentioned above, this is likely to be a conservative estimate, as the permeability is likely to be significantly less. Figure B2 presents the results from the model with a permeability of 10^{-9}m/sec. The drawdown in adjacent groundwater level is relatively similar to the first model, though the predicted inflow is reduced to 16 litres/hour.

We would therefore expect the flowrates to be manageable using conventional sump and pump techniques.

However, this modelling has been based upon mass permeability of the rock. While it appears unlikely from the adjacent excavations, there could be features such as open joints, joint swarms or igneous intrusions through the site, all of which have the potential to conduct high volumes of water. Therefore, the excavation should be carefully monitored during excavation for such features to assess the possibility of these creating higher than expected seepage. If such features are exposed, then consideration could be given to pressure grouting with micro-fine cement to reduce inflow rates.

3.4 Footing Design

We understand that the proposed column loads will be about 3500kN. These will be founded at the base of the proposed excavation within the Unit 5, Class I to II sandstone. Therefore, pad and strip footings are considered suitable for the proposed development.

However, there are likely to be areas where footings are founded at higher level, such as retaining walls or shoring near the perimeter of the excavation. Where the excavation faces extend close to and below these higher level footings, the allowable and ultimate bearing pressures should be divided by three, and geotechnical inspection must be undertaken as the excavation proceeds below these footings to ascertain whether underpinning or additional stabilisation is required. Further advice should be gained from geotechnical engineers when the details of these footings are better understood.

The serviceability pressures provided in the following table are based upon limiting the deflections to 1% of the footing width. The ultimate pressures should be used with a geotechnical strength reduction factor of 0.5, and then assessment of the footing settlement could be based upon the elastic modulus parameters provided.

FOOTING DESIGN PARAMETERS

Material	Bulk Density kN/m^3	Elastic Modulus MPa (Vertical Loading)	Ultimate End Bearing Pressure* MPa	Serviceability End Bearing Pressure MPa
Class V Shale	21	100	3	0.7
Class V Sandstone	21	100	3	1
Class IV Sandstone	22	200	8	2
Class III Sandstone	22	200	20	3.5
Class II and Class I Sandstone	23	1000	60	6

* settlements of greater than 5% of the footing width are required to achieve the ultimate bearing pressure, and these settlements would be even higher when adjacent to deeper excavations.

The rock core recovered from the boreholes has been classified in accordance with Tables 1a and 1b of the “Engineering Classification of Shales and Sandstones in the Sydney Region”, as revised by Pells et al 1998. This paper requires the classification to be based upon the rock within the zone of influence of a particular footing (defined as 1.5 times the least footing dimension), which was not known at the time of reporting. Therefore, the classification at this stage has been based upon classifying nominal 1m sticks of core. When further details of the proposed footings are known, further geotechnical advice should be sought to confirm the bearing pressures are appropriate.

BH	Surface RL (mAHD)	Depth(m)/RL (mAHD) Top of Class V Shale	Depth(m)/RL (mAHD) Top of Class V Sandstone	Depth(m)/RL (mAHD) Top of Class IV	Depth(m)/RL (mAHD) Top of Class III	Depth(m)/RL (mAHD) Top of Class II Or Class I Sandstone
201	14.38	4.3/10.1	5.3/9.1	5.9/8.5	-	8.8/5.6
202	14.86	4.2/10.7	-	4.7/10.2	-	5.7/9.2
203	15.06	4.3/10.8	-	5.1/10.0	-	5.7/9.3
204	15.23	3.6/11.6		4.8/10.4	-	5.9/9.3
205	15.14	-	4.3/10.8	5.4/9.7	-	5.7/9.4
206	15.27	3.0/12.3	-	4.4/10.9	-	6.6/8.7
207	15.33	2.5/12.8	4.0/11.3	-	5.4/9.9	6.2/9.1
208	12.88	-	-	-	-	-
209	13.63	-	4.0/9.6	-	-	5.3/8.3
210	14.69	2.0/12.7	4.8/9.9	-	-	6.2/8.5
211	14.77	2.6/12.2	6.8/8.0	7.5/7.3	-	8.1/6.7

3.5 Subgrade Preparation and Pavement Design

Within the basement area, the sandstone bedrock will be exposed and so no particular subgrade preparation will be required. There must be a sand bedding separation layer between the sandstone and the underside of the floor slabs to permit concrete shrinkage.

Over much of the proposed access ramp area, the sandstone bedrock will be exposed, and again no particular subgrade preparation will be necessary. Toward the shallow end of the access ramps, residual silty clay and weathered shale will be exposed. In these areas, the following subgrade preparation will be required.

3.5.1 Subgrade Preparation

The earthworks recommendations provided below should be complemented by reference to AS3798-2007.

Prior to the placement of the driveway slabs, the soil subgrade should be proof rolled with a minimum 5 tonne dead weight smooth drum roller. The aim of the proof rolling is to improve near surface compaction and to identify any unstable subgrade areas. Proof rolling should be closely monitored by the site supervisor or an experienced geotechnical engineer to detect soft or unstable areas which should be locally excavated down to a stiff base and replaced with engineered fill (as defined below).

3.5.2 Engineered Fill

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm; ‘over wet’ material should also be excluded. We expect the excavated fill and natural soils sourced from the bulk excavations may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller to at least 98% of Standard Maximum Dry Density (SMDD).

To confirm the above specification has been achieved, density tests should be carried out on the engineered fill. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

Compaction of granular basecourse materials on the relatively steep ramp areas will be difficult, and so we recommend the use of a lean mix concrete subbase in this instance. Prior to the placement of the subbase, a subsoil drain should be installed across the high end of the driveway to intercept seepage flows below the pavement.

3.5.3 Pavement Design

We understand the proposed access ramp will be relatively heavily used with medium rigid vehicles. We have adopted a traffic loading of 30 vehicles per day, which over a life of 30 years results in a traffic loading of 5×10^5 CVAG's.

The sample of silty clay tested returned a soaked CBR value of 1.0% which indicates a relatively poor subgrade is present. The extremely weathered shale bedrock would be expected to return similar values. This CBR correlates with an elastic modulus for short term loading (E_S) of 7 MPa, and an elastic modulus for long term loading (E_L) of 5 MPa.

Using the above design parameters, a suitable pavement thickness design in accordance with Austroads '*Guide To Pavement Technology Part 2: Pavement Structural* Design' in this instance would comprise:

205mm of 32 MPA concrete basecourse with SL92 mesh

Over

150mm of 7MPa lean mix concrete

Over

Prepared subgrade with CBR 1.0%.

3.6 Soil and Groundwater Aggression

The soil and weathered rock samples tested returned pH values ranging between 4.4 and 6.2, and sulphate and chloride contents of less than 230mg/kg. Two groundwater samples tested returned pH values of 7.2 and 6.1, and sulphate and chloride contents of less than 360mg/kg.

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability the chemical test results on the soils have indicated that for concrete piles or structures, the conditions range from ‘Mild’ to ‘Moderate’ (with one sample bordering on ‘Severe’ based upon the pH).

Good engineering practices will be necessary to protect concrete in contact with the acidic soils of the site and the designer is referred to the guidelines given in AS2159 and AS3600 for appropriate precautionary measures.

3.7 Earthquake Design Parameters

Based on the advice provided in AS 1170.4-2007 “Structural Design Actions Part 4: Earthquake Actions the site may be assigned a Class C_e (Shallow Soil) classification and a Hazard Factor (Z) of 0.08. With regard to site Class, consideration was given to reducing the Class to B_e, however there will be some parts of the structure where there will be contact with the soil, such as the retention systems and the access ramps, and so a Class of C_e is considered more appropriate.

The site is underlain by shallow soils and predominantly sandstone bedrock, and therefore the likelihood of earthquake induced liquefaction is inconceivable.

4 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Jeffery and Katauskas Pty Ltd 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 and below the completed boreholes and test pits may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

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 Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

P.Wright.

P Wright
Senior Associate

Reviewed by:

P Stubbs
Principal
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

APPENDIX A

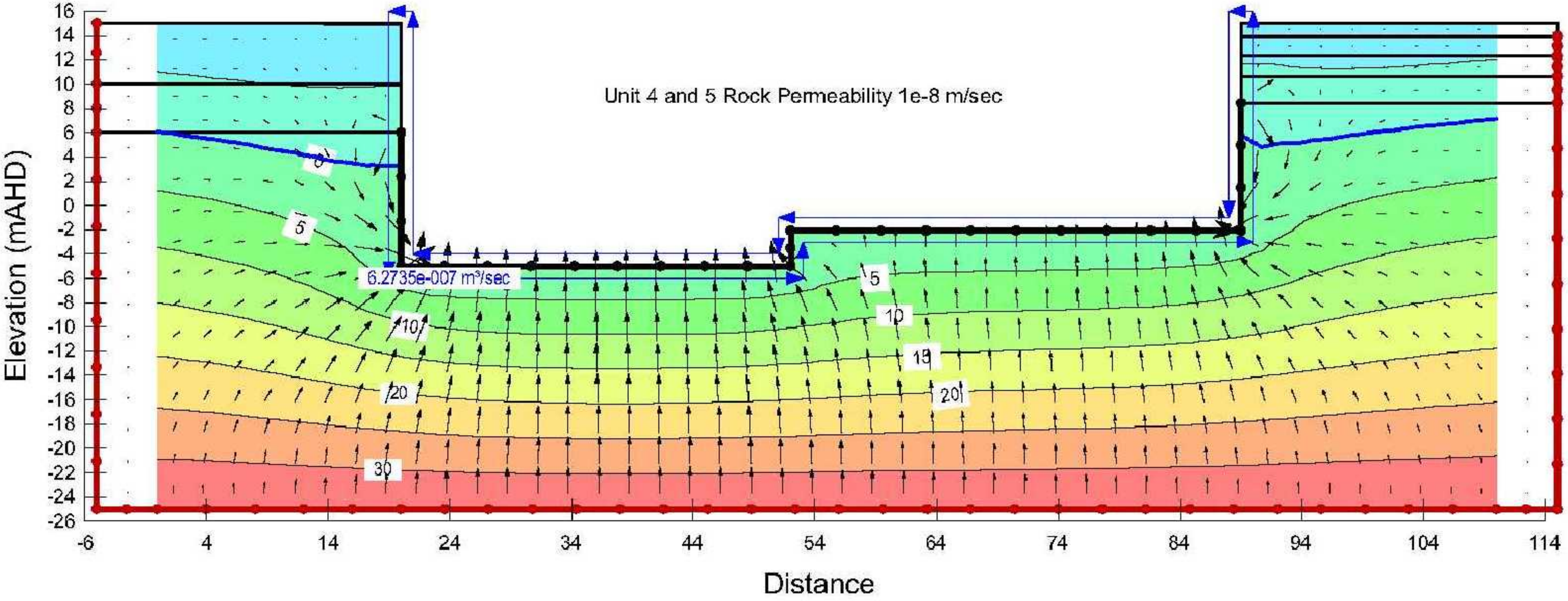
LIST OF PROVIDED ARCHITECTURAL DRAWINGS

(HASSELL LTD PROJECT NUMBER AX002858)

Drawing	Revision	Date
A000	D	15/12/10
A010	B	22/12/10
A022	B	15/12/10
A023	C	22/12/10
A024	C	22/12/10
A025	C	22/12/10
A026	A	15/12/10
A040	C	22/12/10
A041	C	22/12/10
A042	C	22/12/10
A043	C	22/12/10
A110	B	22/12/10
A123	A	15/12/10
A124	A	15/12/10
A125	B	22/12/10
A126	A	15/12/10
A141	B	22/12/10
A142	B	22/12/10

Drawing	Revision	Date
A143	B	22/12/10
A210	A	15/12/10
A220	F	15/12/10
A221	B	15/12/10
A222	F	15/12/10
A223	F	15/12/10
A224	F	15/12/10
A225	F	15/12/10
A226	D	15/12/10
A260	F	15/12/10
A261	F	15/12/10
A262	F	15/12/10
A263	F	15/12/10
A360	C	15/12/10
A910	A	15/12/10
A924	B	15/12/10
A925	B	15/12/10
A980	B	15/12/10

APPENDIX B



SEEPAGE ANALYSIS RESULTS SHOWING PRESSURE HEAD AND FLUX

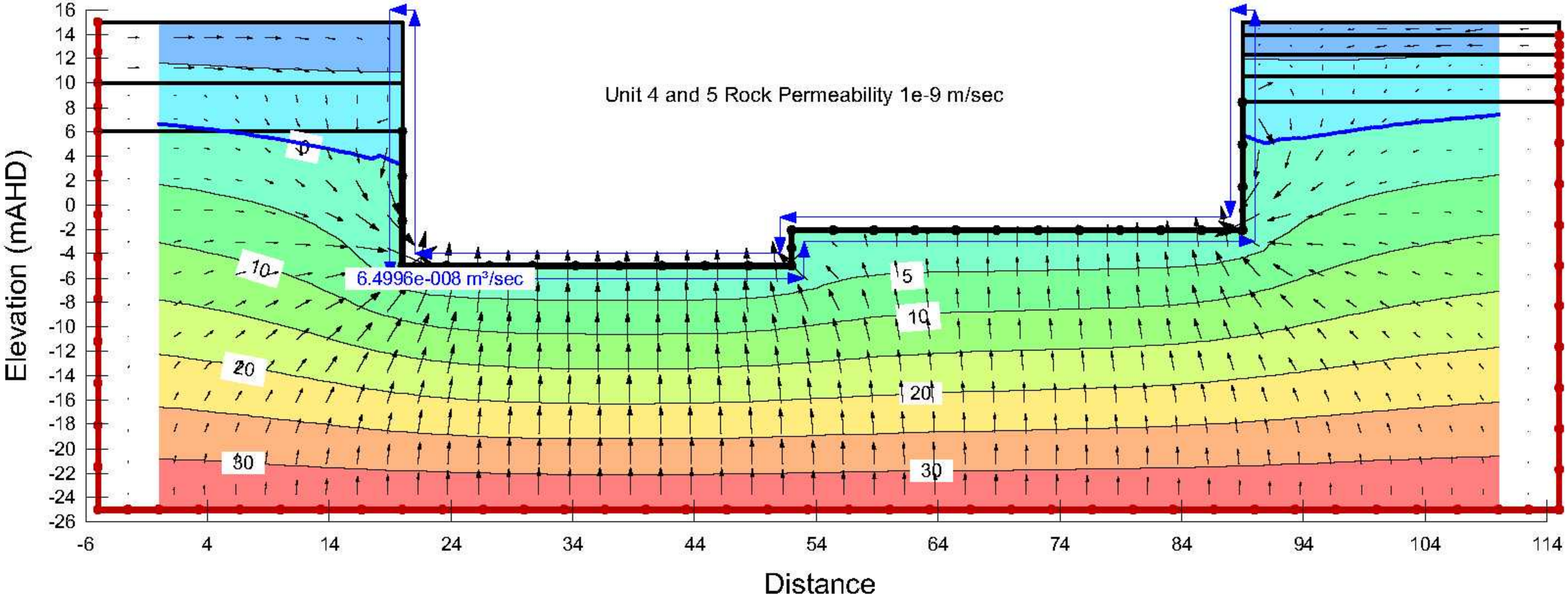
Unit 4 and unit 5 Permeability 1×10^{-8} m/sec

SCALE (m)

0 25

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 24546SP2 **Figure No. B1**



SEEPAGE ANALYSIS RESULTS SHOWING PRESSURE HEAD AND FLUX

Unit 4 and unit 5 Permeability $1x10^{-9}$ m/sec

SCALE (m)

0 25

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Report No. 24546SP2 **Figure No. B2**

APPENDIX C

PROPOSED ASRS DEVELOPMENT
SPECIFICATION FOR PERMANENT ROCK ANCHORS AND ROCK BOLTS

1 Confirmation of Scope

This specification has been prepared at an early stage of the design process, and further revisions may be appropriate when the final details of the design are known. There may also be other materials of procedures suggested by the anchoring contractor which would provide an alternative but still acceptable permanent rock bolt and rock anchor design, and such details would need to be reviewed and agreed by both the structural and geotechnical engineers.

2 Preparation

The rock face shall be scaled down to remove rock fragments that are loosened or could become dislodged from the rock face due to the installations and tensioning of rock bolts and rock anchors.

3 Safety

Care shall be taken at all times during the Works and especially during scaling down, to maintain site safety for site personnel specially those carrying out the work.

4 Drilling Rock Bolt and Rock Anchor Holes, including Cleaning Holes

a) Rock bolt and rock anchor holes are to be drilled using rotary or rotary-percussive equipment, at spacings and/or locations nominated on the drawings or as directed on site by the Geotechnical Engineer.

b) Required rock bolt and rock anchor lengths will be subject to further design, and should be drilled at least 500mm longer than the design anchor length such that incomplete cleaning does not affect bond length of bolt/anchor.

c) The minimum acceptable rock bolt / anchor hole diameter shall be subject to further design.

d) Rock bolt / anchor holes are to be drilled normal to the rock face and at an inclination as shown in the design to be completed (usually between 10° and 15° below the horizontal).

e) Prior to installation, all bolt/anchor holes are to be flush cleaned by clean water passing through a hose or delivery pipe inserted to the base of the hole. The hole will be pronounced clean once clear or almost clear water is being returned out of the hole opening. This procedure shall be supervised by the Builder to ensure it is being carried out correctly.

f) Each of the drill holes should be filled with water. If water loss is found to be greater than 0.5 litres per minute, the hole is to be initially grouted and then redrilled and retested until a satisfactory test result is obtained. Supervision of this procedure may be carried out to assess the need for grouting and redrilling. All holes with an unsatisfactory water loss are to be identified to the Geotechnical Engineer within 24 hours of the initial water test.

g) On completion of drilling and flushing, all holes should be plugged or otherwise protected to prevent entry of foreign matter.

h) The bolting/anchoring contractor is to record for each hole, date drilled, length and diameter drilled, orientation of the hole, any drilling problems or ‘weak’ seams intersected, and confirmation of satisfactory water flushing and cleaning. The details are to be provided to the Builder prior to installation of the rock bolt or anchor.

5 Rock Bolt or Rock Anchor

a) The rock materials will be subject to further design, with the materials selected for corrosion resistance.

b) Total in hole length of rock bolts or rock anchors to comply with the design which is yet to be completed.

c) The safe working load of the rock bolts/anchors shall be in accordance with the future design.

d) Care should be taken to prevent damage, kinking or bending of bolts or anchors; damaged bolts/anchors shall not be used.

e) Rock bolts/anchors are to be fully encapsulated within a corrugated polypropylene sheath for corrosion protection. The heads of the rock bolts/anchors must be encased within a box packed with grease for corrosion protection of the heads of these elements. This head protection will require further detailing.

f) Bolts/anchors shall be kept free from oil, grease, mud or any other deleterious substances; the exception is the free length of anchor strands which should be greased and sleeved to maintain the free length during grouting. The steel should not be visibly pitted or rusted.

6 Installation and Grouting

a) PVC spacers or spiders shall be provided along the length of the rock bolts or rock anchors to maintain them centrally within the drill hole. The first spacer shall not be greater than 0.5m from the top of the drill hole.

b) The sheathed rock bolt/anchor must be inserted prior to grouting and the grout delivery tube must be placed to the bottom of the hole at the same time. The grout tube must be steadily removed such that it does not displace the bolt/anchor.

c) Grout mix to surround rock bolt is to have a target water/cement ratio of 0.45 and shall be mixed to a uniform consistency prior to use. The grout shall have an average unconfined compressive strength (for cubes of not greater than 70mm size) of at least 25MPa at seven days with no single test less than 20MPa.

d) Grout is to be pumped to the base of the hole through hoses or grout tubes until the consistency of the grout mix escaping at the hole opening is the same as that being pumped in. Once this is the case, the grout tube may be withdrawn slowly such that the rate of grout exiting the hole is virtually maintained. Only when the tube is completely removed from the hole should the pumping mechanism be switched off.

e) If grout level drops below drill hole opening whilst still wet, it should be topped up until loss of grout is negligible. If the grout level cannot be maintained, then the rock bolt or anchor must be withdrawn and advice obtained from the Geotechnical Engineer.

f) Once grout is dry or almost dry, a thick, non-shrink topping grout should be packed into the hole until the grout completely covers the bolt/dowel up to the drill hole opening. The grout shall be finished flush with the surrounding rock face.

g) The bolt/anchor head assembly shall comprise an end (bearing) plate resting on a mortar pad, with a hemispherical seating washer and nut tightened against the plate in the case of a bolt, or a bearing block and wedges in the case of an anchor.

h) Where mortar pads are required, the mortar shall be non-shrink and of a strength at least equal to that of the grout. The mortar pad shall be formed to the required size and the bearing plate seated to provide uniform bearing.

i) The bolting/anchoring contractor is to record for each hole, bar length and diameter, date and time of grouting, grouting difficulties, whether the grout is sampled and if tested, the cube identification number. The details are to be provided to the Builder prior to fitting the end (bearing) plate.

7 Load Testing of Rock Bolts/Dowels

a) Rock bolts and anchors are to be load tested to at least 130% of their working load, and following lock-off, a lift off test is to be completed to confirm the lock-off load.

b) After approximately three days, lift-off tests are to be completed on at least 25% of the anchors/bolts to confirm they are holding their load. If the bolts/anchors lose more than 10% of their lock-off load, then an additional lift off test is to be completed following another three days. If additional load loss is recorded, further advice on replacement of the bolts/anchors should be sought from the geotechnical and structural engineers. If continued loss of load is encountered, all anchors/bolts should be subjected to lift-off tests to assess their load capacity.

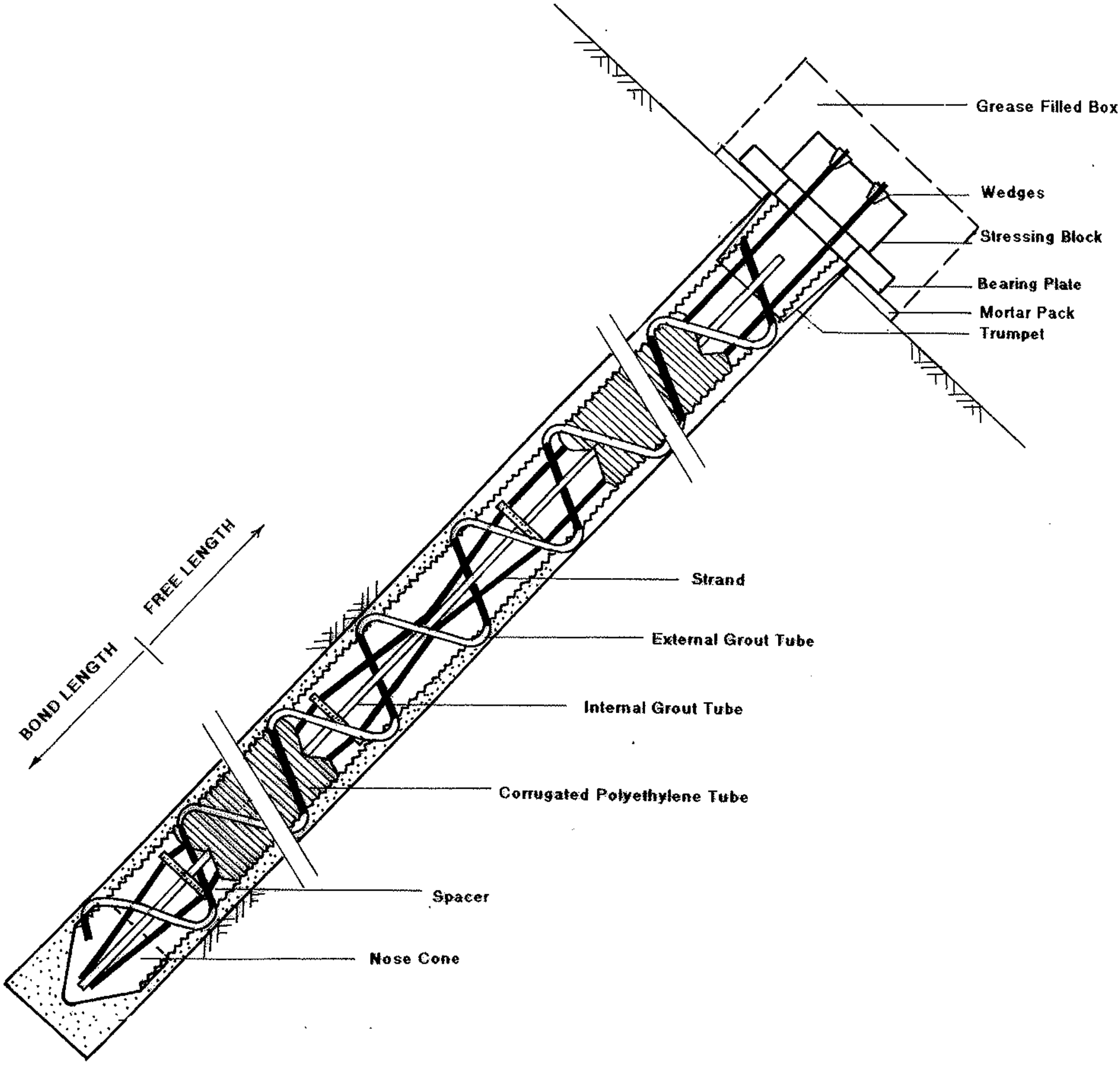
8 Australian Standards

Wherever Australian Standards exist with regard to the materials and workmanship referred to in this Specification, then they shall be deemed to apply.

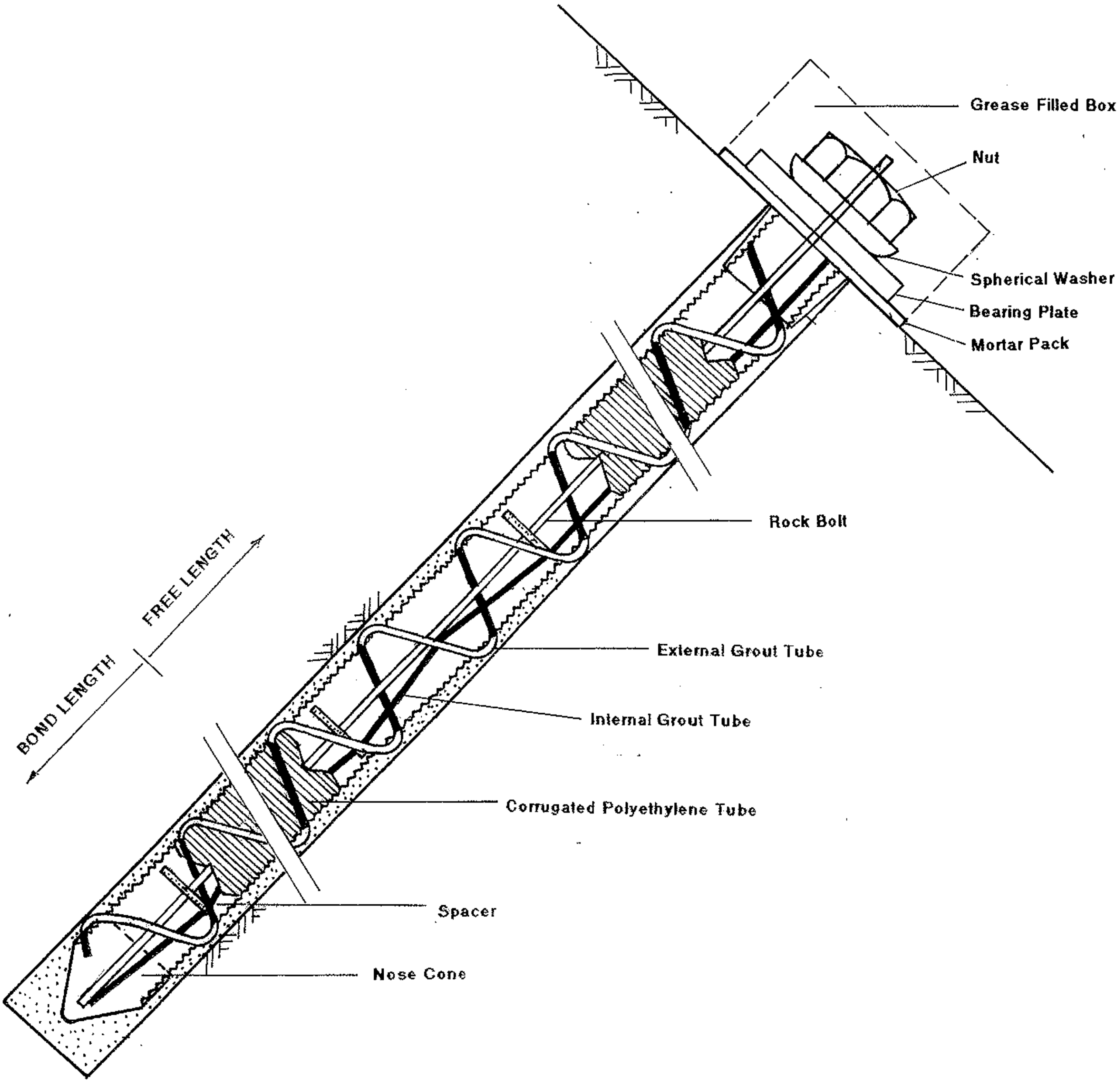
9 Inspections

a) The Builder should check each bolt/dowel to confirm that the bar length, diameter, steel grade, spacers/centralisers and any corrosion protection are in accordance with the relevant specification and drawings.

b) The Builder should check that grouting is carried out in accordance with Section 6 “Installation and Grouting”.



DETAIL FOR PERMANENT MULTI STRAND ROCK ANCHOR



DETAIL FOR PERMANENT ROCK BOLT

Jeffery and Katauskas Pty Ltd			
Report No.	24546SPrpt2	Figure No.	C2

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection 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 a senior geotechnical engineer.

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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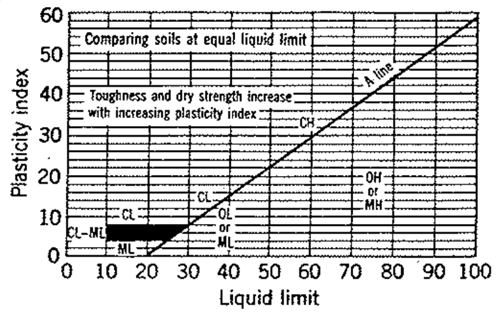
UNIFIED SOIL CLASSIFICATION TABLE

Major divisions		Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)	Group Symbols [a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is *larger* than 75 µm sieve size[b] (The 75 µm sieve size is about the smallest particle visible to naked eye) — Gravels: More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Clean gravels (little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*	
	Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Gravels with fines (appreciable amount of fines)	Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits above "A" line, with *PI* greater than 7	
Sands: More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Clean sands (little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*	
	Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Sands with fines (appreciable amount of fines)	Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7	

Laboratory classification criteria (coarse-grained soils): Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows: Less than 5%: *GW, GP, SW, SP*; More than 12%: *GM, GC, SM, SC*; 5% to 12%: *Borderline* cases requiring use of dual symbols.

Use grain size curve in identifying the fractions as given under field identification

Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	Identification Procedures on Fraction Smaller than 380 µm Sieve Size: Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
	High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	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 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%) — SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense >85 >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

Ref: Standard Sheets/Log Symbols
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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
		0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
		0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
		0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
		1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
		3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
		10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	