# Appendix 6.18

Interpretive Report Geotechnical Investigation

Prepared by Jeffery & Katauskas Pty Ltd

AS/NZS ISO 9001 Certified
Davis Langdon Certification Services

# INTERPRETATIVE REPORT

TO

## UNIVERSITY OF TECHNOLOGY, SYDNEY

ON

## GEOTECHNICAL INVESTIGATION

FOR

## PROPOSED FACULTY OF BUSINESS BUILDING

AT

## 14 TO 28 ULTIMO ROAD, ULTIMO, NSW

14 January 2011

Ref: 24387WRrpt2

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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 new faculty of business building at No. 14 to 28 Ultimo Road, Ultimo, NSW. Our environmental division, Environmental Investigation Services (EIS) has prepared a Stage 2 Environmental Site Assessment Report (Ref: E24387KBrpt) dated November 2010 which also included a waste classification of site soils. The reader is referred to these reports for additional information.

The proposed development includes the construction of a twelve storey building over one level of basement car parking over the majority of the site. The proposed finished floor reduced level (RL) of the basement car park will be at RL0.5m. Bulk excavation to achieve design subgrade levels are expected to extend to a maximum depth of approximately 5m below existing surface levels.

The boreholes and test pits have generally revealed a subsurface profile below the concrete paved surface comprising fill over natural soils predominantly comprising alluvial clays with sand bands then sandstone bedrock at moderate depth. Dolerite corresponding to an igneous dyke was encountered at depth over the south-western corner of the site. Groundwater levels were recorded at shallow depth within the soil profile.

Based on the investigation results, the principal geotechnical issues associated with the proposed development will be the stability of the proposed excavation, de-watering the excavation and limiting ground surface movements outside the excavation, in particular the old brick sewer which crosses the eastern side of the site close to the eastern site boundary (see Figure 1).

There is also the distinct likelihood that traces of old footing systems and other infrastructure associated with the former building occupying the site will be encountered during bulk excavations. The drilling of retention piles may also be detrimentally affected by the presence of buried infrastructure and old footings.

The EIS report recorded elevated concentrations of various contaminants in the fill and identified the presence of acid sulphate soils. Removal and appropriate disposal of contaminated soils will need to be completed in accordance with the advice provided in the Remedial Action Plan prepared by EIS.

The bulk excavations will extend below the perched water table present within the soil profile and de-watering of the excavation will be required. To reduce the potentially detrimental effects on neighbouring buildings and structures due to groundwater draw-down associated with de-watering of the excavation a full depth engineered retention system will need to be installed before excavations commence. The retention system will need to extend down to bedrock in order to form perimeter cut-off walls. Groundwater removed from the site during de-watering of the excavation is likely to require some treatment of the groundwater prior to disposal. 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 location of the TransGrid tunnel and associated surface infrastructure below Ultimo Road and adjacent to the southern end of the eastern site boundary, respectively will also need to be considered. Temporary ground anchors to support the retention system may extend beyond the TransGrid tunnel "proposed acquisition boundary". TransGrid will need to be contacted for further information in this regard. If permission to install anchors is not forthcoming then internal propping systems will need to be utilised over these portions of the retention system.

The basement will need to be designed as tanked as control of groundwater seepage behind conventional basement retention systems would require a sump and pump stormwater drainage system of significant capacity. Consideration could be given to drained basement whereby underfloor drainage is provided to manage long term rebound of groundwater levels beneath the site once de-watering operations have ceased. However, any basement drainage would require effective on-going maintenance and management costs associated with ensuring continual operation of the pumps and monitoring of groundwater quality prior to discharge may be regarded as prohibitive. However, such a system may not be preferred by Council, irrespective of potential groundwater quality issues.

The bedrock underlying the site will provide appropriate support for the envisaged building loads and a piled footing system will be required.

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## 1 INTRODUCTION

This interpretative geotechnical has been prepared for the proposed University of Technology, Sydney (UTS) Faculty of Business Building at 14 to 18 Ultimo Road, Ultimo, NSW. This report was commissioned by Ms Sisi 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 (J&K) dated 7 October 2010.

We note that we have completed geotechnical and environmental investigations at the site which were carried out between 11 and 21 October 2010. The factual results of the geotechnical investigation are presented in our Factual Report (Ref. 24387WRrpt) dated 14 January 2011. Our environmental division, Environmental Investigation Services (EIS) has prepared a Stage 2 Environmental Site Assessment Report (Ref: E24387KBrpt ver. 1.2) dated January 2011 (which also included a waste classification of site soils and a preliminary assessment of acid sulphate soils) and a Remedial Action Plan (Ref: E24387KBrpt ver. 2.2) dated January 2011. The reader is referred to these reports for additional information.

Based on the provided architectural plans (Drawing Numbers A2-1.01, A2-2.01 to A2-2.11 and A7-1.01 to A7-1.07 dated 9 November 2010 and A2-2.00 dated 2 November 2010) prepared by Gehry Partners, LLP and information provided by Arup we understand the following:

- The proposed new UTS Faculty of Business building will comprise construction of a twelve storey building over one level of basement car parking. The finished floor reduced level (RL) of the basement car park will be at RL0.5m; bulk excavations to achieve design subgrade levels are expected to extend to a maximum depth of about 5m below existing surface levels.

- Column loads will range between 4,500kN and 10,000kN, with a maximum line loading beneath the basement retaining walls of about 100kN/m.

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

We understand that Arup are completing detailed structural design and our report also includes specific geotechnical design parameters requested by Arup to assist with their design.

## 2 GEOTECHNICAL AND ENVIRONMENTAL ISSUES AND CONSTRAINTS

The boreholes and test pits have generally revealed a subsurface profile below the concrete paved surface comprising fill over natural soils predominantly comprising alluvial clay soils with sand bands then sandstone bedrock at moderate depth. Dolerite corresponding to an igneous dyke was encountered at depth over the south-western corner of the site and is not expected to affect the site unless pile footings penetrate significant depths into the overlying sandstone bedrock. Groundwater levels were recorded at shallow depth within the soil profile. 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 proposed excavation,
- De-watering the excavation and basement, and

- Limiting ground surface movements outside the excavation, in particular in relation to the old brick sewer which crosses the eastern side of the site close to the eastern site boundary (see Figure 1).

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, particularly the large pile caps that are believed to span over the existing brick sewer will be encountered; it is also suspected that raking piles extend down into the proposed excavation from below the western sides of the pile caps spanning the brick sewer. The drilling of retention piles may also be detrimentally affected by the presence of buried infrastructure and old footings.

***Possible Groundwater Drawdown Effects:*** The bulk excavations will extend below the perched water table present within the soil profile and de-watering of the excavation will be required. To reduce the potentially detrimental effects on neighbouring buildings and structures due to groundwater draw-down associated with de-watering of the excavation a full depth engineered retention system will need to be installed before excavations commence. The retention system will need to extend down to bedrock in order to form perimeter cut-off walls. Groundwater removed from the site during de-watering of the excavation is likely to require some treatment of the groundwater prior to disposal. 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.

***Tanked or Drained Basement Design:*** Control of groundwater seepage into the un-tanked basement would require a sump and pump groundwater drainage system of significant capacity. Consideration could be given to a drained basement design whereby underfloor drainage is provided to manage long term rebound of groundwater levels beneath the site once de-watering operations have ceased. However, any basement drainage would require effective on-going maintenance and management costs associated with ensuring continual operation of the pumps and monitoring of groundwater quality prior to discharge may be regarded as prohibitive. Further, such a system may not be preferred by Council, irrespective of potential groundwater quality issues. Our preference, therefore, is for the basement to be designed as tanked

***Presence of TransGrid Tunnel:*** The location of the TransGrid tunnel and associated surface infrastructure below Ultimo Road and adjacent to the southern end of the eastern site boundary, respectively will also need to be considered. We understand that rock bolts support the tunnel and excavation sides. Based on the information provided to us before we commenced our geotechnical information, the rock bolts supporting the shaft and tunnel sides adjacent to the southern end of the eastern site boundary are about 2.7m long. We have no information regarding the installed TransGrid tunnel support measures however we assume that they have been constructed within the “proposed acquisition boundary” of the tunnel. The northern margin of the “proposed acquisition boundary” of the tunnel appears to be off-set about 6m from the southern site boundary and lines the southern end of the eastern site boundary. We recommend that TransGrid be contacted for further information in relation to the location and extent of tunnel and excavation support measures as they may be intersected by temporary anchors installed to support the proposed excavation retention system (if adopted).

***Site Contamination and Acid Sulphate Soils:*** The EIS report (Ref: E24387KBrpt ver. 1.2) dated January 2011 identified acid sulphate soils in the natural soil profile and recorded the following elevated concentrations of contaminants in the fill:

- Elevated concentrations of total PAHs in BH112 and BH113 which are believed to be associated with the ash and slag material encountered in the fill.

- Elevated concentrations of TPH contamination most likely associated with leaks from the former USTs and the petroleum arrestor pit.

- Elevated concentrations of TPH were also encountered in fill samples obtained from the eastern portion of the site and are believed to be present here due to the brick sewer backfill acting as a contamination migration pathway.

- Elevated concentrations of arsenic, copper and zinc were encountered in groundwater samples and are considered to be the result of regional groundwater conditions rather than a site specific issue. Elevated TPH concentrations were also recorded in the groundwater and are most likely associated with the historic spills and leaks from the USTs and the petroleum arrestor pit located on the site.

Removal and appropriate disposal of contaminated soils and management of acid sulphate soils will need to be completed in accordance with the advice provided in the Remedial Action Plan (Ref: E24387KBrpt ver. 2.2, dated January 2011) prepared by EIS.

# 3 COMMENTS AND RECOMMENDATIONS

## 3.1 Demolition and Excavation

### 3.1.1 General

We note that neighbouring buildings, structures and paved surfaces line, or lie close to the site boundaries. During demolition and excavation, there is the potential to damage or de-stabilise these neighbouring buildings, structures and paved surfaces (including any buried services).

Construction of the proposed development will require demolition of the existing paved surface. Further, the excavations will encounter underground fuel storage tanks (UST's) over the north-eastern corner of the site, old pile footings close to the eastern site boundary and relic buried services infrastructure associated with the former Dairy Farmers building. Removal and appropriate disposal of contaminated soils and remaining old buried services infrastructure will need to be completed in accordance with the advice provided in the Remedial Action Plan (Ref: E24387KBrpt ver. 2.2, dated January 2011) prepared by EIS.

Demolition and excavation will therefore need to be carefully completed and this work will need to be completed using suitably experienced (and insured) contractors.

We recommend that during demolition further test pits be excavated adjacent to the existing brick sewer in an attempt to identify the nature and extent of the existing pile caps and pile footings spanning the brick sewer. The test pits would need to be temporarily shored and/or supported to allow entry by personnel as we expect the test pits to extend below 1.2m depth. The test pits should be inspected by the structural and geotechnical engineers to confirm the locations of pile caps and pile footings spanning the brick sewer and assess the best method of removal (including any appropriate temporary support measures) prior to commencement of the drilling of retention piles. The top surface of the existing pile caps spanning the brick sewer may also be damaged by the drilling of retention piles. The potential for such damage to detrimentally impact the brick sewer and/or the neighbouring buildings and structures to the east and should also be assessed by the structural engineer following inspection of the above mentioned test pits.

The natural clayey and sandy alluvial soils expected to be revealed at proposed basement subgrade level may be found to be unstable if proper site drainage is not implemented during construction. It is important to provide good drainage during construction of the basement in order to promote run-off and reduce the potential for ponding. If the clayey subgrade is exposed to periods of rainfall, softening will result and site trafficability will be poor. Consideration will need to be given to the provision of a sacrificial layer of re-cycled demolition rubble to improve the subgrade surface. It is also likely that such a layer will be required to provide a suitable working platform for tracked equipment such as piling rigs.

### 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 demolition and excavations to be completed using hydraulic tracked excavators up to say 40 tonne size.

Demolition of the existing concrete paved surface and old brick or concrete footings may well require the use of rock breaker attachments to the tracked excavators. However, demolition of the paved surfaces will need to be carried out with care so as not to damage or de-stabilise existing buildings, structures and paved surfaces lining the site boundaries. We recommend that a saw cut be provided around the perimeter of the paved surface prior to commencement of its removal. We expect a saw attachment to the excavator would be used then breaking up of the paved surface probably completed using ripping tynes or possibly rock breaker attachments and removal of the concrete pieces using a bucket attachment to the tracked excavator.

Further comments regarding use of rock breakers are provided in Section 3.1.3, below.

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, and by reference to AS3798-2007 '*Guidelines on Earthworks for Commercial and Residential Developments*'.

The outline of the proposed excavation is indicated on the attached Figure 1. Excavations down to the proposed basement subgrade level will extend to a maximum depth of about 5m with localised lift pit excavations extending an additional maximum depth of about 2m below bulk excavation level.

The proposed excavations will encounter fill and natural soils and will be readily achievable using bucket attachments to the above mentioned tracked excavators.

We reiterate that reference must be made to the Remedial Action Plan (Ref: E24387KBrpt ver. 2.2, dated January 2011) prepared by EIS for advice on off-site disposal of the excavated materials and management of acid sulphate soils.

### 3.1.3 Potential Vibration and Ground Surface Movement Risks

The generally assessed poorly or moderately to poorly compacted surficial fill encountered in our previous and current investigations may be expected to extend beyond the site boundaries. We therefore advise that sudden stop/start movements of tracked equipment should be avoided in order to reduce transmission of ground vibrations to adjoining buildings and structures, in particular the old brick sewer located close to the eastern site boundary.

Care is required with demolition where rock breakers are used as their use may result in direct transmission of ground vibrations to neighbouring buildings, structures and paved surfaces. If there is any cause for concern then demolition should cease and further advice sought. In this regard, we recommend that rock breakers initially be restricted to say less than maximum 750kg size.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, be operated one at a time and in short bursts only to reduce amplification of vibrations. In this regard we note the comments in section 3.1.2, above regarding provision of saw cuts around the perimeter of the paved surface prior to commencement of its removal. Grid saw cuts may also assist in controlling vibrations as well as subsequent removal of the paved surface.

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

We recommend that periodic 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 vibration monitoring should include the production of attenuation curves showing vibration, measured as peak particle velocity, verses distance from source for the particular equipment to provide further guidance in relation to the suitability of the equipment being used. 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 neighbouring buildings and structures to the west and east, the paved surfaces to the north, south and west and any movement sensitive buried services that may surround the site. In this regard, we strongly recommend that a detailed dilapidation survey of the brick sewer be completed, and should include a CCTV camera survey of the inside of the sewer and possibly man entry. However, we note the current sewer upgrade works being completed on behalf of Sydney Water by EPTEC and such a dilapidation survey may not be achievable; Sydney Water should be contacted for further advice in this regard.

We recommend that the owners be asked to sign the dilapidation survey reports and agree that they are a fair assessment of existing conditions, as these can then be used as a benchmark in assessing potential damage claims (due to ground surface movements and/or vibration damage).

#### 3.1.5 Seepage

Based on the results of the investigation, groundwater seepages were recorded in the natural soil profile at depths ranging between 2.4m and 7.2m and on completion of auger drilling standing water levels were recorded at depths ranging between 3.3m and 6.6m. In TP117 a standing water level was recorded at 0.8m depth (RL4.2m) several days after completion of excavation of the test pit. We also note that on the basis of the groundwater monitoring well data the highest standing water level was recorded at 3.3m to 3.6m depth (BH111 to BH113) which is equivalent to about RL1.8m to RL1.9m. The standing water level recorded in TP117 probably represents a perched water table within a more granular zone of fill materials beneath the site and the likelihood of other such perched water tables being encountered cannot be discounted.

On the basis of the above, the proposed basement excavation (and localised deeper lift pit excavations) will intersect the groundwater within the soil profile and de-watering will be required. Further comments and recommendations on dewatering are provided in Section 3.3 below.

## 3.2 Retention

### 3.2.1 Retention Methods

A piled wall retention system, which will also function as a groundwater cut-off wall, is considered suitable for the site. Due to the potentially collapsible nature of the soil profile and the need to control potential movement of neighbouring ground surfaces, buildings and structures we consider that conventional unlined bored piles are not suited to this site.

Suitable retention/cut-off systems include a Continuous Flight Auger (CFA) grout injected secant pile wall or a diaphragm wall. A modification to the diaphragm wall is proprietary in-situ soil and grout mixing systems within a low strength cement/bentonite slurry such as provided by Wagstaff or Bachy Soletanche. The system involves construction of 'interlocking' panels of up to about 2.4m width which are formed in a 'hit 1 miss 1' sequence. This is also a low vibration method, generates nominal amounts of spoil and can provide a satisfactory final wall surface.

A contiguous pile wall has been considered but allowance would need to be made for making good gaps between the piles in order to reduce the loss of retained soils and groundwater inflows, both of which the potential to cause inducement of adjacent ground surface movements. In this regard, consideration may be given to providing an internal shotcrete face to the contiguous pile wall. The shotcrete facing would need to be applied in 'lifts' of maximum 1.5m vertical height and must be applied on the same day as completion of excavation in front of the contiguous pile wall. However, this would increase the time to complete the basement walls, increase the thickness of the basement wall and a satisfactory cut-off from groundwater seepage is likely to be very difficult to achieve. The shotcrete facing would have to be designed to resist the external hydrostatic pressure.

We have also considered a sheet pile wall system and consider that it would not provide an effective cut-off system as the sheet piles may not only have difficulty penetrating the bedrock surface but may also refuse in the hard clays, on bands of weathered bedrock or be detrimentally impacted by obstructions within the fill. Furthermore, sheet piles would not be able to be incorporated into the footing system and permanent basement retaining walls would need to be constructed following bulk excavation.

Based on the above discussions, contiguous pile walls or sheet pile walls are not considered to be a feasible option. Our preference would be for a grout injected secant pile wall or a diaphragm wall. However, in relation to tanking details there are some issues associated with the use of secant pile walls and further advice is provided in Section 3.3.

The retention/cut-off system must be installed prior to excavation commencing to support the proposed vertical cuts in the soil profile. The base of the retention system should be adequately imbedded below bulk excavation level, not only to provide adequate cut-off, but also to provide the required lateral toe restraint. The natural soil profile below bulk excavation level, although generally clayey, is quite variable and the presence of continuous sand bands, particularly in the deeper alluvial soil profiles, cannot be discounted. We therefore recommend that the retention/cut-off system be socketted into bedrock.

Other lateral restraint in the form of ground anchors or internal propping will be required and must be installed progressively as the excavation proceeds (i.e. the lateral restraint must be installed once the restraining point has been uncovered). Ground anchors will extend beyond the site boundaries and permission from neighbouring property owners will be required. For the basement, we have assumed that permanent lateral support of the retaining walls will be provided by the proposed structure. If not, permanent anchors will be required which will need to be designed for corrosion resistance and for long term durability. Ground anchor design parameters are provided in Section 3.2.2, below.

There is the potential for the ground anchors to extend close to, or into the "proposed acquisition boundary" of the TransGrid tunnel. If such a situation is unacceptable to the owners of the tunnel then internal propping will be required. Such internal propping may take the form of raking props which would require reaction to be provided by pile footings located within the site and socketted into bedrock. Alternatively, temporary propping of the walls may be achieved by using a temporary bench of soil left in front of the retention system. For initial planning purposes the bench should be a 3m minimum horizontal width just below the crest of the pile capping beam and should be graded down to bulk excavation level at 1V in 1.5H. This bench design will require further detailed design evaluation to confirm that adequate stability is achieved and that associated movements would be acceptable. The bench can be removed once the floor slabs of the proposed building provide permanent support to the retaining walls. This 'top down' type construction technique would require careful construction sequencing.

Construction of the retaining system and anchors should be of a high quality and only experienced contractors should be employed. Any gaps between adjacent secant piles must be grouted immediately as soil loss and seepage could cause settlements of the ground adjoining the retention system.

### 3.2.2 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.

- Progressively anchored or propped walls where minor movements can be tolerated (i.e. along the street frontages provided there are no movement sensitive buried services or structures), we recommend the use of a uniform rectangular earth pressure distribution of 6H kPa, where H is the retained height in metres.
- For progressively anchored or propped walls which retain areas highly sensitive to lateral movement (such as along the eastern side of the basement adjacent to the existing sewer), a uniform rectangular earth pressure distribution of 8H kPa should be adopted.
- Bulk unit weights reported in the 'Retaining Wall Design Parameters' table, below should be reduced by $10kN/m^3$ for soils and bedrock below the groundwater table.
- Any surcharge affecting the walls (e.g. due to traffic, the existing brick sewer, adjacent footings, construction loads, etc) should be allowed in the design using the appropriate 'at rest' earth pressure coefficients ($K_o$), reported in the 'Retaining Wall Design Parameters' table, below.

- The tanked or drained basement retention system must be designed to withstand full hydrostatic pressures. A groundwater level of at least RL2.9m should be adopted for design although our preference would be for a groundwater level coincident with the surrounding surface level as an 'ultimate' design case. Such an 'ultimate' design case would cover the possibility of the detrimental impact of a burst water pipe adjacent to the site. Reference should be made to Section 3.4 below for further comments in this regard.

- For piles embedded into the underlying sandstone bedrock below bulk excavation level, an allowable lateral toe resistance of 300kPa for low (or higher strength) sandstone may be adopted. This value assumes excavation is not carried out within the zone of influence of the wall toe and the rock does not contain unfavourable defects, etc. The upper 0.3m depth of the sockets should not be taken into account to allow for localised variations in rock level and quality. We note that the 'passive' earth pressure for overlying soil should not be added to the lateral resistance calculated for rock sockets due to strain incompatibility.

- Rock anchors bonded at least 3m into the low (or higher) strength sandstone may be designed for an allowable bond stress of 300kPa.

- Soil anchors bonded into the natural soils may be designed using the appropriate shear strength parameters reported in the 'Retaining Wall Design Parameters' table, below.

- All anchors should be proof tested to 1.3 times the working load under the supervision of an experienced engineer or construction superintendent, independent of the anchor contractor. We recommend that only experienced contractors be considered for the anchor installation. We note that the anchors may require to be installed below the groundwater table.

In addition, we note that Arup require additional design parameters for the retained soils to assist with their computer based retaining wall design and the table below sets out the required parameters for the subsurface profile. The parameters have been based on the laboratory test results, our past experience and reference to established published data on such soil and rock parameters.

| RETAINING WALL DESIGN PARAMETERS | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Material** | **Bulk Density kN/m$^3$** | **E′ MPa** | **Eu MPa** | **Poissons Ratio** | **Effective Friction Angle** | **Effective Cohesion kPa** | **Undrained Shear Strength kPa** | **Ko** | **Mass Permeability m/s** |
| **Granular Fill** | 18 | 15 | 15 | 0.3 | 25° | 0 | N/A | 0.58 | $10^{-6}$ |
| **Clayey Fill** | 18 | 6 | 8 | 0.3 | 25° | 1 | 50 | 0.58 | $10^{-8}$ |
| **Natural Sands (Loose )** | 17 | 15 | 15 | 0.3 | 30° | 0 | N/A | 0.5 | $10^{-4}$ |
| **Natural Silty &/or clayey Sands (Loose )** | 17 | 15 | 15 | 0.3 | 28° | 0 | N/A | 0.5 | $10^{-7}$ |
| **Natural Sands (Medium Dense)** | 21 | 40 | 40 | 0.3 | 33° | 0 | N/A | 0.46 | $10^{-4}$ |
| **Natural Silty &/or clayey (Medium Dense)** | 21 | 40 | 40 | 0.3 | 33° | 0 | N/A | 0.46 | $10^{-7}$ |
| **Natural Clays (Very Soft)** | 17 | 1 | 2 | 0.3 | 25° | 1 | 10 | 0.58 | $10^{-9}$ |
| **Natural Clays (Soft)** | 17 | 2 | 4 | 0.3 | 25° | 1 | 15 | 0.58 | $10^{-9}$ |
| **Natural Clays (Firm)** | 20 | 6 | 8 | 0.3 | 25° | 1 | 40 | 0.53 | $10^{-9}$ |
| **Natural Clays (Stiff)** | 21 | 12 | 16 | 0.3 | 30° | 2 | 75 | 0.5 | $10^{-9}$ |
| **Natural Clays (Very Stiff)** | 21 | 25 | 30 | 0.3 | 30° | 2 | 150 | 0.5 | $10^{-9}$ |
| **Natural Clays (Hard)** | 21 | 45 | 60 | 0.3 | 30° | 2 | 200 | 0.5 | $10^{-9}$ |
| **Class V Sandstone** | 22 | 100 | 100 | 0.3 | 28° | 20* | N/A | 0.53 | $10^{-10}$ |
| **Class IV Sandstone** | 24 | 700 | 700 | 0.2 | 35° | 500* | N/A | N/A | $10^{-10}$ |
| **Class III Sandstone** | 24 | 1,200 | 1,200 | 0.2 | 40° | 1,000* | N/A | N/A | $10^{-10}$ |

*: 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.

### 3.2.3 Excavation Related Ground Movements

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 sands, 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 less than 0.1% of the excavation 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. Additional movements may be induced by de-watering of the excavation.

## 3.3 Excavation De-Watering

De-watering of excavations can lead to lowering of groundwater levels outside of the site if not properly designed and implemented. Further, the zone of groundwater drawdown can potentially extend well in excess of 50m outside the site boundaries and lowering of groundwater levels may cause settlement of nearby high level footings, pavements, buried services, slabs-on-grade etc.

In order to achieve a dewatered excavation during basement construction and to reduce potential settlement of the ground outside the excavation we recommend that a groundwater cut-off wall be constructed around the basement perimeter prior to de-watering the excavation. This method has the advantage that potential drawdown of groundwater outside the site is kept to a practical minimum although this clearly requires good quality construction of the cut-off walls.

Spear points or wells will be required to lower the groundwater below bulk excavation level and locally around lift pit excavations. Assuming a lowering of the groundwater to 2m below bulk excavation the volume of water to be extracted would be of the order of at least $9x10^6$ litres. The rate at which the groundwater can be extracted will be a function of the mass permeability of the soil, the capacity and number of pumps used and whether or not the groundwater can be discharged to the stormwater or sewer system (with or without on-site treatment). If the groundwater has to be treated before discharge into the sewer or stormwater system and/or if the water has to be removed from site using tankers then this will further slow down the rate at which groundwater can be extracted. Careful sequencing of the de-watering and discharge of extracted groundwater will clearly be required.

We recommend that a de-watering investigation and analysis should however first be carried out to assess pump out rates. We would also recommend that observation wells be installed immediately outside the excavation perimeter in order to monitor the groundwater levels adjacent to the excavation. Any appreciable lowering of groundwater outside the excavation indicated by the monitoring wells would give early warning of potential problems with the cut-off wall. In this event, de-watering would need to cease and advice sought from the structural and geotechnical engineers and the de-watering sub-contractor.

### 3.4 Basement Tanking or Drained Basement Details

We note the advice provided in Section 3.2.2, above with regard to design groundwater levels. The tanked basement, if selected, must be designed to withstand full hydrostatic pressures in order to control uplift of the basement floor slab. If the self weight of the building cannot provide such uplift resistance then rock anchors or piles socketted into bedrock will be required to resist such uplift forces.

A groundwater level of at least RL2.9m should be adopted for design although our preference would be for a groundwater level coincident with the surrounding surface level as an 'ultimate' design case. Such an 'ultimate' design case would cover the possibility of the detrimental impact of a burst water pipe adjacent to the site.

In order to permanently tank or waterproof the basement, the structural design must take into account the lateral and uplift pressures associated with this system. Care is required with the tanking details particularly at piled footing locations and at the perimeter pile wall-floor slab connection. In this regard, we note that there are practical difficulties in providing a waterproof seal between floor slabs and the undulating face of secant pile walls. Provision would need to be made for sealing any gaps with a proprietary waterproof sealant. Alternatively, the reinforcement cage for the slurry wall system can be modified to incorporate styrene packing at the floor slab level together with starter bars. Once the excavation reveals the styrene packing it may be removed and the starter bars bent out in preparation for forming up the floor slab-retaining wall connection.

If the basement is designed to be drained then resisting hydrostatic uplift forces will not be required. However, a drainage layer will need to be provided beneath the basement floor slab and the water discharged to the stormwater system. Discharging of the basement drainage will require a sump and pump system although in our experience Councils typically do not grant permission for such sump and pump systems to discharge onto the stormwater drainage; further advice from Council will be required in this regard. Depending on the contaminants present in the groundwater, some form of treatment may also be required prior to discharge into the stormwater drainage; on-going testing and monitoring will be required. In addition, a back-up pump (and/or generator) will need to be provided in the event of pump and/or power failure. Clearly effective building maintenance will need to be implemented if a sump and pump system is selected.

The drained basement slab should be designed to be separated from all walls, columns, footings, etc to permit relative movements (i.e. 'floating'). The concrete floor slab should be provided with effective shear connection of joints by using dowels or keys. The 'floating' slab design will also allow some leakage at the wall-slab interface in the event that the floor slab drainage does not operate effectively, say due to pump failure or localised high volumes of groundwater (such as following heavy rainfall or flood events). The capacity for leakage will prevent build-up of hydrostatic forces beneath the slab which may cause extensive cracking and damage to the floor slab.

The basement underfloor drainage should comprise a continuous layer of durable, single sized, washed gravel (e.g. 'blue metal'). A non-woven geofabric such as Bidim A34 will need to be provided at the interface between the drainage layer and the subgrade to act as a filter against sub-soil erosion. The thickness of the drainage layer will need to be determined by a hydraulic engineer but we expect the thickness to be a minimum of 100mm. The use of sand as a drainage material is not advised as there is the potential for the sand to 'clog up' particularly if the sand has a significant 'fines' (silt) content which would also have an adverse affect on its permeability.

Notwithstanding the above comments, our preference would be for a tanked basement as this option removes the need for on-going maintenance and problems associated with Council permissions and on-going testing of water quality.

Further comments on subgrade preparation are provided in Section 3.6, below.

## 3.5 Footings

Grout injected CFA piled footings socketted into bedrock and the basement retention system socketted into bedrock will be suitable for supporting the envisaged column loads which will range between 4,500kN and 10,000kN and the maximum line loading beneath the basement retaining walls of about 100kN/m. Support of such loads on the alluvial soils would only be possible using a stiff raft or piled raft which are unlikely to be as economic as a piled footing system. Hence these options have not been considered further.

We have re-produced the table of indicative bedrock classification presented in Section 3.5 of our Factual Report which was prepared in accordance with Table 1 of the "Engineering Classification of Shales and Sandstones in the Sydney Region", as revised by Pells et al 1998.

| BH | Surface RL (mAHD) | Depth(m)/RL Top of Class V | Depth(m)/RL Top of Class IV | Depth(m)/RL Top of Class III | Depth(m)/RL Top of Class II | Depth(m)/RL Top of Class I |
|---|---|---|---|---|---|---|
| 101 | 5.14 | 9.4/-4.26 | 10.1/-4.96 | 10.6/-5.46 | 11.1/-5.96 & 13.6/-8.46 | 12.2/-7.06 & 15.2/-10.06 |
| 102 | 5.29 | 9.7/-4.41 | - | 11.8/-6.51 | - | 10.3/-5.01 & 12.3/-7.01 |
| 103 | 5.01 | - | - | 9.2/-4.19 & 15.4/-10.39 | - | 11.8/-6.79 |
| 104 | 4.85 | - | - | 9.8/-4.95 | 7.7/-2.85 & 13.1/-8.25 | - |
| 105 | 4.81 | - | - | 8.5/-3.69 & 13.1/-8.29 | - | 9.6/-4.79 & 13.6/-8.79 |
| 106 | 4.94 | - | 7.2/-2.26 | 8.0/-3.06 | 11.1/-6.16 | 8.8/-3.86 & 11.8/-6.86 |
| 107 | 5.23 | 9.6/-4.37 | - | 11.4/-6.17 | 10.4/-5.17 | 12.6/-7.37 |
| 108 | 4.82 | 11.2/-6.38 & 14.8/-9.98 (Dolerite) | 11.6/-6.78 | 8.4/-3.58 | 13.0/-8.18 | 10.1/-5.28 |

These classifications should be re-considered once pile sizes/design loads are better defined to confirm the classification relative to the zone of influence for each footing.

Based on the results of the current investigations, the top surface of bedrock is expected to be encountered at depths ranging between 7.2m and 9.8m below current site surface levels, i.e. at reduced levels ranging between RL-2.26 (BH106) and RL-4.41m (BH102).

Tables of design parameters for pile footings in compression, tension and for lateral loadings (such as earthquake loadings) are set out in the following tables.

| PILE DESIGN PARAMETERS (IN COMPRESSION) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Material** | **Bulk Density kN/m$^3$** | **E MPa (Vertical Loading)** | **Ultimate End Bearing Pressure MPa** | **Serviceability End Bearing Pressure MPa** | **Ultimate Shaft Adhesion MPa** | **Serviceability Shaft Adhesion MPa** | **UCS MPa** |
| **Class V Sandstone or Dolerite** | 22 | 100 | 3 | 1 | 0.15 | 0.10 | 1 |
| **Class IV Sandstone** | 24 | 700 | 10 | 2.5 | 0.5 | 0.25 | 2 |
| **Class III Sandstone** | 24 | 1,200 | 30 | 3.5 | 0.8 | 0.35 | 7 |
| **Class II Sandstone** | 24 | 2,000 | 120 | 6 | 1.5 | 0.6 | 12 |
| **Class I Sandstone** | 24 | 2,000 | 120 | 6 | 1.5 | 0.6 | 24 |

In accordance with Table 2 of the "Engineering Classification of Shales and Sandstones in the Sydney Region", as revised by Pells et al 1998 'typical' settlements for footings founded on bedrock with the above Serviceability End Bearing Pressures would be less than 1% of the minimum footing width.

The Serviceability End Bearing Pressure for the Class II and Class I sandstone provided above may be regarded as conservative. Though not applicable to this site, for pad footings, higher Serviceability End Bearing Pressures may be adopted and would be confirmed by spoon testing of pad footing bases. However, spoon testing is not feasible for pile footings hence the adoption of more conservative values. The Serviceability End Bearing Pressure could be increased to say 8MPa for Class I and II sandstone provided cored boreholes were drilled at specific column locations where higher loads are expected so as to confirm rock quality and the required founding level.

| PILE DESIGN PARAMETERS (IN TENSION) | | | | | | |
|---|---|---|---|---|---|---|
| **Material** | **Bulk Density $kN/m^3$** | **E MPa (Vertical Loading)** | **Ultimate Shaft Adhesion MPa** | **Serviceability Shaft Adhesion MPa** | **Angle of Cone Weight (β)** | **UCS** |
| **Class V Sandstone or Dolerite** | 22 | 100 | 0.075 | 0.05 | 30° | 1 |
| **Class IV Sandstone** | 24 | 700 | 0.25 | 0.15 | 45° | 2 |
| **Class III Sandstone** | 24 | 1,200 | 0.4 | 0.2 | 45° | 7 |
| **Class II Sandstone** | 24 | 2,000 | 0.75 | 0.3 | 45° | 12 |
| **Class I Sandstone** | 24 | 2,000 | 0.75 | 0.3 | 45° | 24 |

| PILE DESIGN PARAMETERS FOR LATERAL LOADING | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Material** | **Bulk Density $kN/m^3$** | **E MPa (Lateral Loading)** | **Poissons Ratio** | **Effective Friction Angle** | **Effective Cohesion kPa** | **Undrained Shear Strength kPa** | **UCS** |
| **Class V Sandstone or Dolerite** | 22 | 100 | 0.3 | 28° | 20* | N/A | 1 |
| **Class IV Sandstone** | 24 | 700 | 0.2 | 35° | 500* | N/A | 2 |
| **Class III Sandstone** | 24 | 1,200 | 0.2 | 40° | 1,000* | N/A | 7 |
| **Class II Sandstone** | 24 | 2,000 | 0.2 | 45° | 2,000* | N/A | 12 |
| **Class I Sandstone** | 24 | 2,000 | 0.2 | 45° | 2,000* | N/A | 24 |

*: 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.

We note that the 'passive' earth pressure for overlying soil should not be added to the lateral resistance calculated for rock sockets due to strain incompatibility.

Based on information provided by contractors, secant pile walls are likely to be formed using minimum 0.45m diameter piles and diaphragm walls constructed at a minimum width of the order of 0.45m. On this basis, for a maximum line load of 100kN/m, the maximum bearing pressure would be of the order of 225kPa or 250kPa for a diaphragm wall or secant pile wall, respectively. Based on the above tables of pile design parameters, the sandstone bedrock (irrespective of Class) will be well able to support such loads. We would therefore expect the retention system socket length to be governed by lateral stability considerations.

For the envisaged column loads of between 4,500kN and 10,000kN we provide below estimates of the end bearing pressure for the two largest diameter commercially available CFA pile footings assuming all load is taken in end bearing.

| Pile Diameter | Column Load | Applied End Bearing Pressure |
|---|---|---|
| 0.9m | 4,500kN | 7.1MPa |
| 0.9m | 10,000kN | 15.7MPa |
| 1.2m | 4,500kN | 4MPa |
| 1.2m | 10,000kN | 8.8MPa |

Based on the above values and assuming a maximum Serviceability End Bearing Pressure of 6MPa pile groups would be required in most instances. To optimise the number of piles used a higher Serviceability End Bearing Pressure of the order of

8MPa would be required together with a proportion of the load carried by the rock sockets. We reiterate the above comments in relation to additional cored boreholes at specific column locations should higher Serviceability End Bearing Pressures be required. In addition, the table below sets out loads that can be supported by rock sockets (in tension or compression) for the various sandstone classes and the two pile diameters referred to above. It should also be noted that the initial 0.3m of the rock socket should be ignored in load carrying capacity calculations to allow for localised variations in rock level and quality.

**Serviceability Load Carrying Capacity (Tension and Compression) for 1m long Rock Sockets for each Sandstone Class and Pile Diameters (D)**

| Sandstone Class | Compression | Tension | Compression | Tension |
|---|---|---|---|---|
| | D = 0.9m | D = 1.2m | D = 0.9m | D = 1.2m |
| V | 280kN | 380kN | 140kN | 190kN |
| IV | 700kN | 940kN | 430kN | 570kN |
| III | 990kN | 1,320kN | 570kN | 750kN |
| II | 1,700kN | 2,260kN | 850kN | 1,130kN |
| I | 1,700kN | 2,260kN | 850kN | 1,130kN |

If significant rock sockets are required to support loads over the south-western corner of the site then pile lengths will need to be carefully considered as the zone of influence may extend to the poorer quality dolerite dyke beneath the sandstone. If this situation occurs then the Serviceability End Bearing Pressure would need to be reduced accordingly. Further advice in relation to issues and considerations with regard to the igneous dyke are provided in Section 3.6, below.

We note that the bases of CFA piles cannot be inspected and sample recovery from the drill hole is limited. Confirmation that the required quality of bedrock has been penetrated would therefore need to be based on the depth of the pile hole, reference to nearby borehole logs, additional cored boreholes drilled at specific locations (if higher Serviceability End Bearing Pressures are adopted) and information provided by the piling contractor, in particular, torque readings. Load testing of individual piles could also be considered. We recommend that the piling contractor be requested to certify the load carrying capacity of the piles.

### 3.6 Igneous Dyke Issues and Considerations

The igneous dyke (a vertical or near-vertical igneous intrusion) encountered in BH108 was located at a vertical depth of about 14.8m at the site boundary. Based on our experience at the adjoining site to the west and information contained on the TransGrid tunnel drawings, the dyke width has been estimated to be of the order of 5m and trending approximately east-west (see Figure 1).

We note that the contact between the sandstone bedrock and dolerite dyke was marked by a clay seam sloping at 75° (when corrected for the borehole inclination angle). Based on the results of BH108 and BH109 we have therefore assessed the dyke margin to be sloping down to the north at 75° and the approximate location and extent of the dyke is indicated on Figure 1. However, we have also identified a zone of the site where the dyke may sub-crop at the top surface of bedrock in the unlikely event that the dyke margin slopes down to the south at 75°.

The dolerite encountered beneath the sandstone bedrock in BH108 was assessed to be extremely to distinctly weathered and of extremely low to very low strength. Below about 16.6m vertical depth the dolerite improved to distinctly weathered and of very low to low strength. However, our experience within the adjoining site to the west indicated that the upper portion of the dyke typically comprised a residual clay profile of up to 12m thickness before extremely to distinctly weathered dolerite was encountered.

We reiterate the advice provided in section 3.5 that if significant rock sockets are required to support loads over the south-western corner of the site then pile lengths will need to be carefully considered as the zone of influence may extend to the poorer quality dolerite dyke beneath the sandstone. If this situation occurs then the Serviceability End Bearing Pressure would need to be reduced accordingly.

Conversely, whilst we are not expecting the dyke to be encountered in the retention pile holes or the pile footing holes (unless they penetrate significant depth into the sandstone bedrock) we recommend that there be a contingency within the footing design should such materials be encountered. As a guide, the residual clayey dyke materials would be of very stiff to hard strength and a serviceability end bearing pressure of 300kPa may be assumed for design purposes. Alternatively, selected footings over the south-western corner of the site may be designed to bridge over the area of the dyke, if it is encountered.

### 3.7 Basement Subgrade Preparation

If either a tanked or drained basement is selected then preparation of the subgrade will be required prior to construction of the basement floor slab and the basement de-watering will need to have lowered the groundwater level a sufficient depth below the subgrade level in order to allow earthworks plant to effectively compact the subgrade. Alternatively, if the basement floor slab is suspended then no particular subgrade preparation will be required.

The subgrade is expected to comprise the variable sandy and clayey alluvial soils. As outlined in Section 3.1.1, above trafficability is likely to be poor and consideration will need to be given to the provision of a sacrificial layer of re-cycled demolition rubble to improve the subgrade surface. If piles are to be installed from bulk excavation level, then further consideration will be required to determine the working platform thickness once the piling plant size and loads are known.

### 3.6.1 Subgrade Preparation

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

Prior to placement of basement on-ground floor slabs the soil subgrade should be proof rolled with a minimum 5 tonne dead weight smooth drum vibratory 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).

Care should also be taken when using vibrating equipment not to cause damage to adjacent existing structures close to the site. If there is any cause for concern then proof rolling should cease and further geotechnical advice sought. Alternatively, where appropriate, the static (non-vibration) mode may be used.

### 3.6.2 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 at a frequency of one test per layer per 500m$^2$ or three tests per visit, whichever requires the most tests, for general fill and one test per 2 layers per 50m$^2$ for backfill to soft or heaving areas. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

A sub-base layer of 100mm of DGB20 (or equivalent), compacted to at least 100% SMDD should be used beneath the drained basement floor slabs as they will be carrying traffic loads. The sub-base layer will be in addition to the drainage layer described in Section 3.4.

We note that if the tanked basement is selected then the floor slab will be relatively thick and the ability of the subgrade to support the traffic loads will not be a critical issue. Consequently, the subgrade preparation outlined in Section 3.6.1, above will suffice.

### 3.8 Soil and Groundwater Aggression

Based on the advice provided in AS2159-2009 “Piling Design and Installation” for corrosion protection and durability and AS3600-2009 “Concrete Structures” the chemical test results on the soils have indicated that for concrete piles or structures, an A1 or ‘Non-aggressive’ Exposure Classification may be assigned based on Table 4.8.1 of AS3600 and Table 6.4.2(C) of AS2159, respectively. With regard to the groundwater chemical test results and the advice provided in Table 4.3 of AS3600 and Table 6.4.2(A) of AS2159, for piles or concrete structures below the groundwater level a B1 or ‘Mild’ Exposure Classification may be assigned.

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 the Cement and Concrete Associations Technical Note TN57 for additional appropriate precautionary measures.

### 3.9 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 adopted.

## 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.

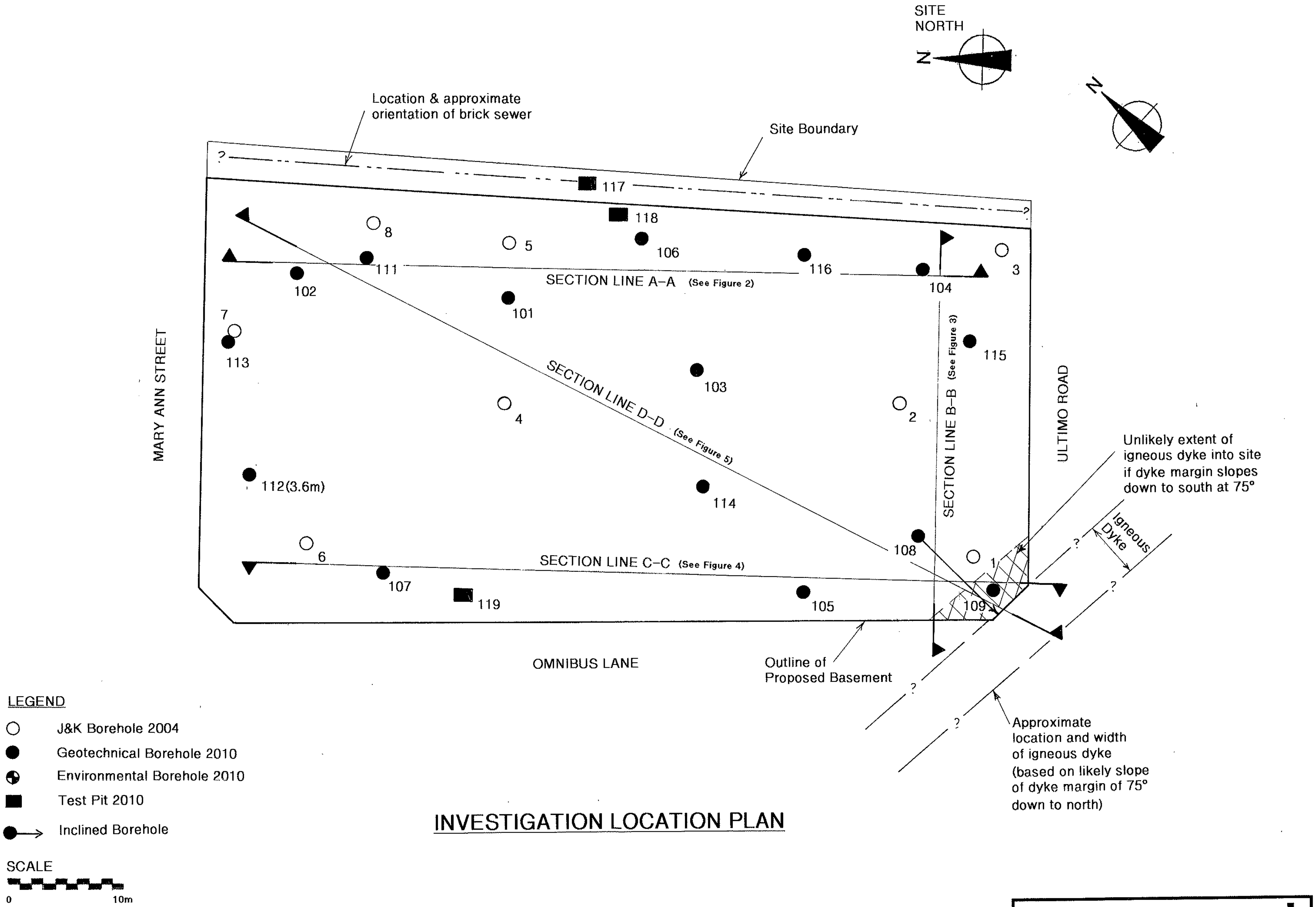
A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of 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.

Paul Roberts
Senior Associate

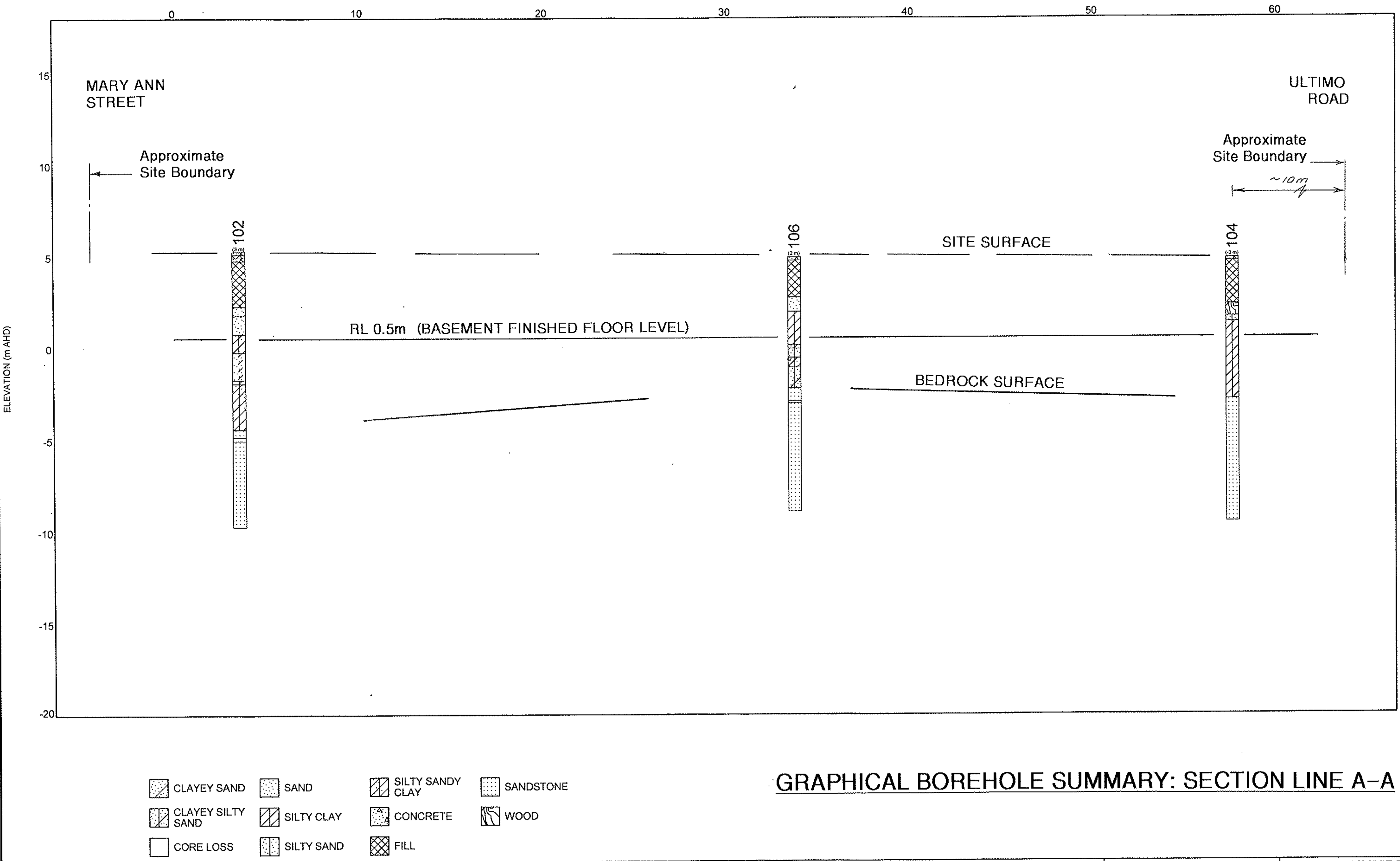
Bruce Walker
Principal
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.




## INVESTIGATION LOCATION PLAN

**Jeffery and Katauskas Pty Ltd**

Report No. 24387WR Figure No. 1




# GRAPHICAL BOREHOLE SUMMARY: SECTION LINE A–A

| No. | Amendment Description | Initials | Date |
|---|---|---|---|
| | | | |

A3 Original | This sheet may be prepared using colour and may be incomplete if copied | Coordinate System: MGA94 Zone 56 | Height Datum: AHD

PREPARED BY JEFFERY AND KATAUSKAS PTY LTD - CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DESIGNED: P.R.

REVIEWED: B.F.W.

**Jeffery and Katauskas Pty Ltd**

UNIVERSITY OF TECHNOLOGY SYDNEY
NEW FACULITY OF BUSINESS BUILDING
14-18 ULTIMO ROAD, SYDNEY, NSW
GRAPHICAL BOREHOLE SUMMARY SECTION A-A

**SCALES**

H 1:200

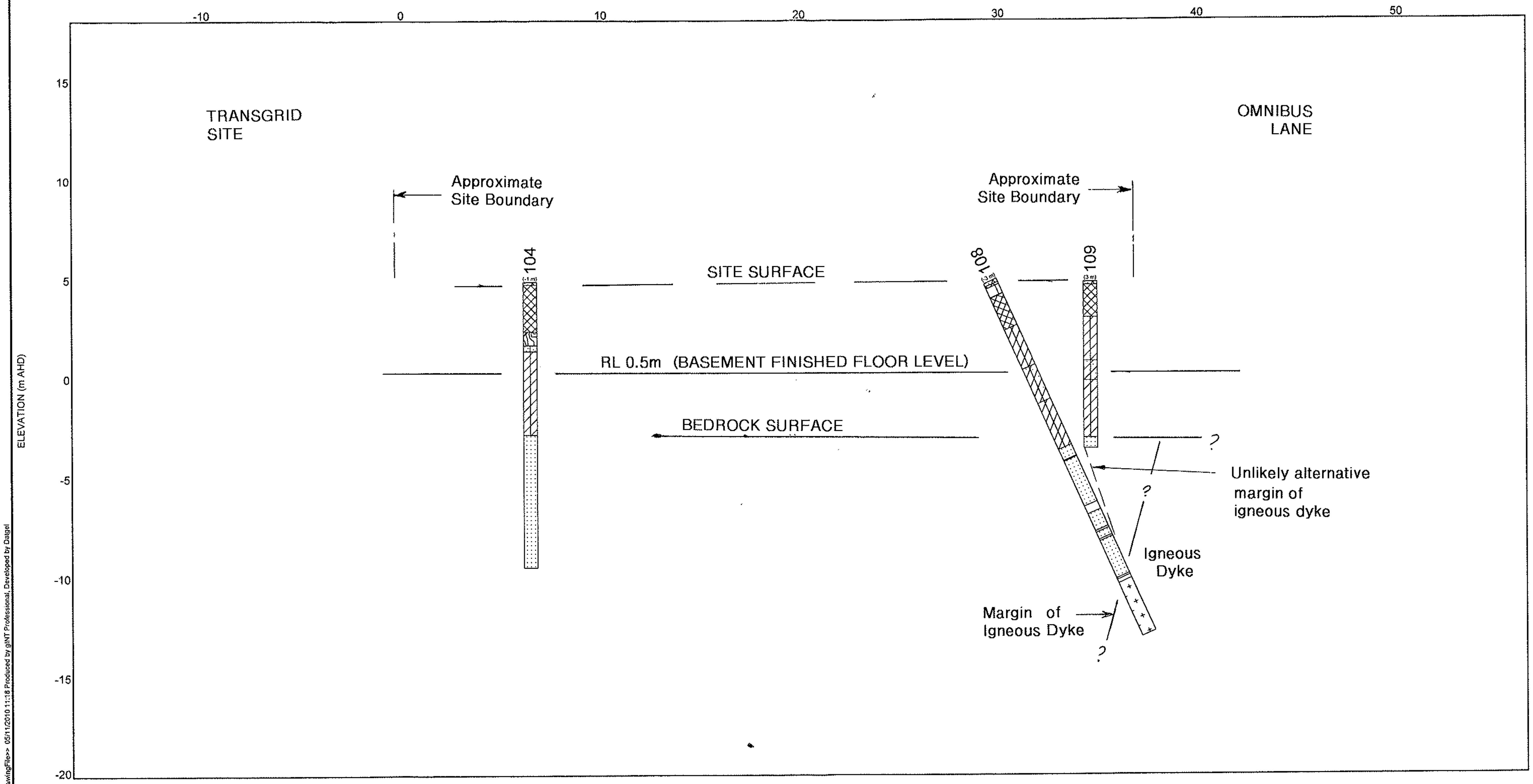
V 1:200

**JOB NUMBER**

**24387WR**

**FIGURE NUMBER**

**2**




# GRAPHICAL BOREHOLE SUMMARY: SECTION LINE B-B

BRICK
SILTY CLAY
DOLERITE, DIORITE
WOOD
CLAYEY SILTY SAND
SILTY SANDY CLAY
FILL
CORE LOSS
CONCRETE
SANDSTONE

| No. | Amendment Description | Initials | Date |
|---|---|---|---|

A3 Original | This sheet may be prepared using colour and may be incomplete if copied | Coordinate System: MGA94 Zone 56 | Height Datum: AHD

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DESIGNED: P.R.

REVIEWED: B.F.W.

**Jeffery and Katauskas Pty Ltd**

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14-18 ULTIMO ROAD, SYDNEY, NSW
GRAPHICAL BOREHOLE SUMMARY SECTION B-B

**SCALES**

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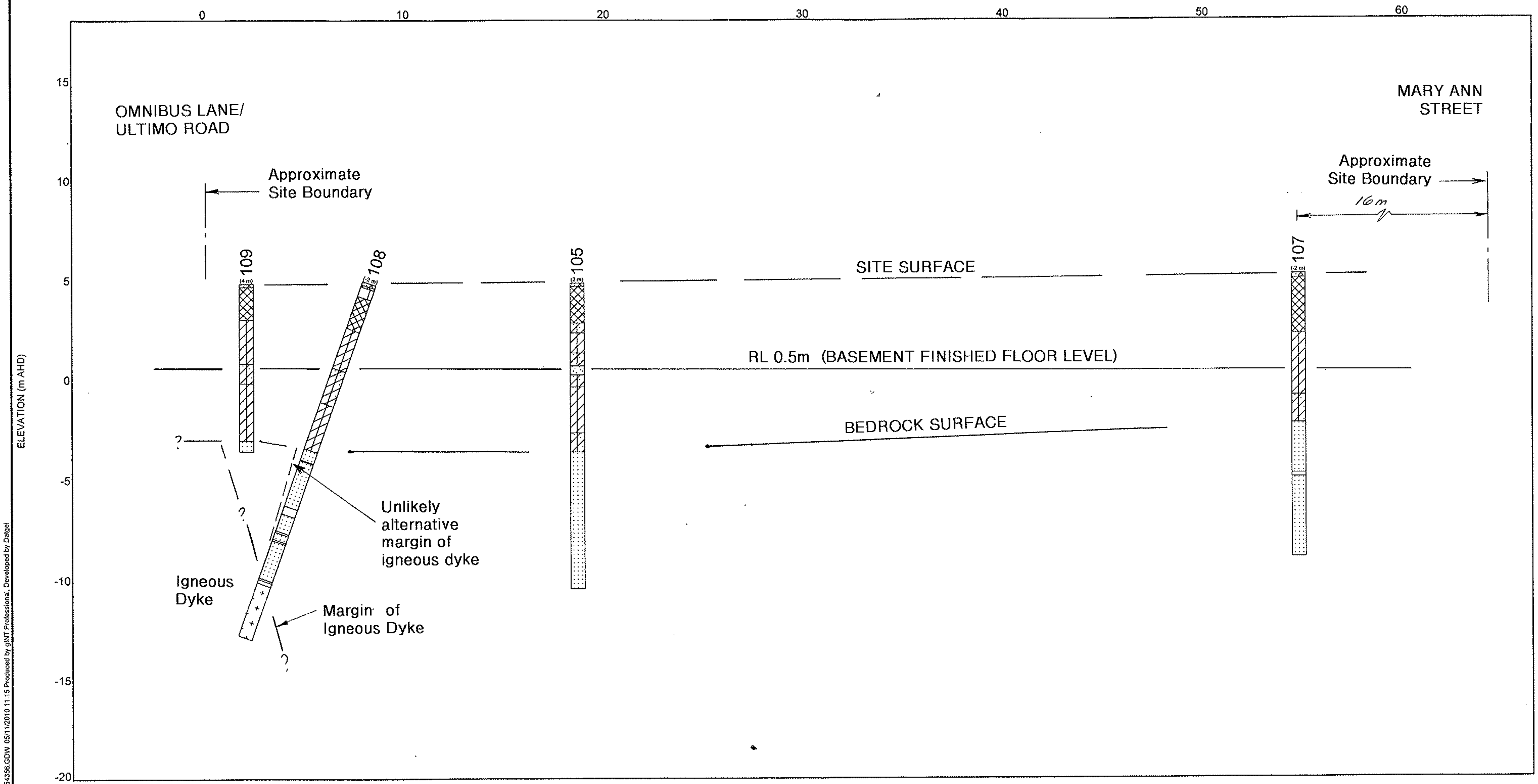
V 1:200

**JOB NUMBER**

24387WR

**FIGURE NUMBER**

3




# GRAPHICAL BOREHOLE SUMMARY: SECTION LINE C–C

- BRICK
- CORE LOSS
- SAND
- SANDY SILTY CLAY
- SILTY CLAY
- SILTY SANDY CLAY
- CONCRETE
- DOLERITE, DIORITE
- FILL
- SANDSTONE

| No. | Amendment Description | Initials | Date |
|---|---|---|---|
| | | | |

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DESIGNED: P.R.
REVIEWED: B.F.W.

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14-18 ULTIMO ROAD, SYDNEY, NSW
GRAPHICAL BOREHOLE SUMMARY SECTION C-C

**SCALES**
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V 1:200

**JOB NUMBER**
24387WR

**FIGURE NUMBER**
4

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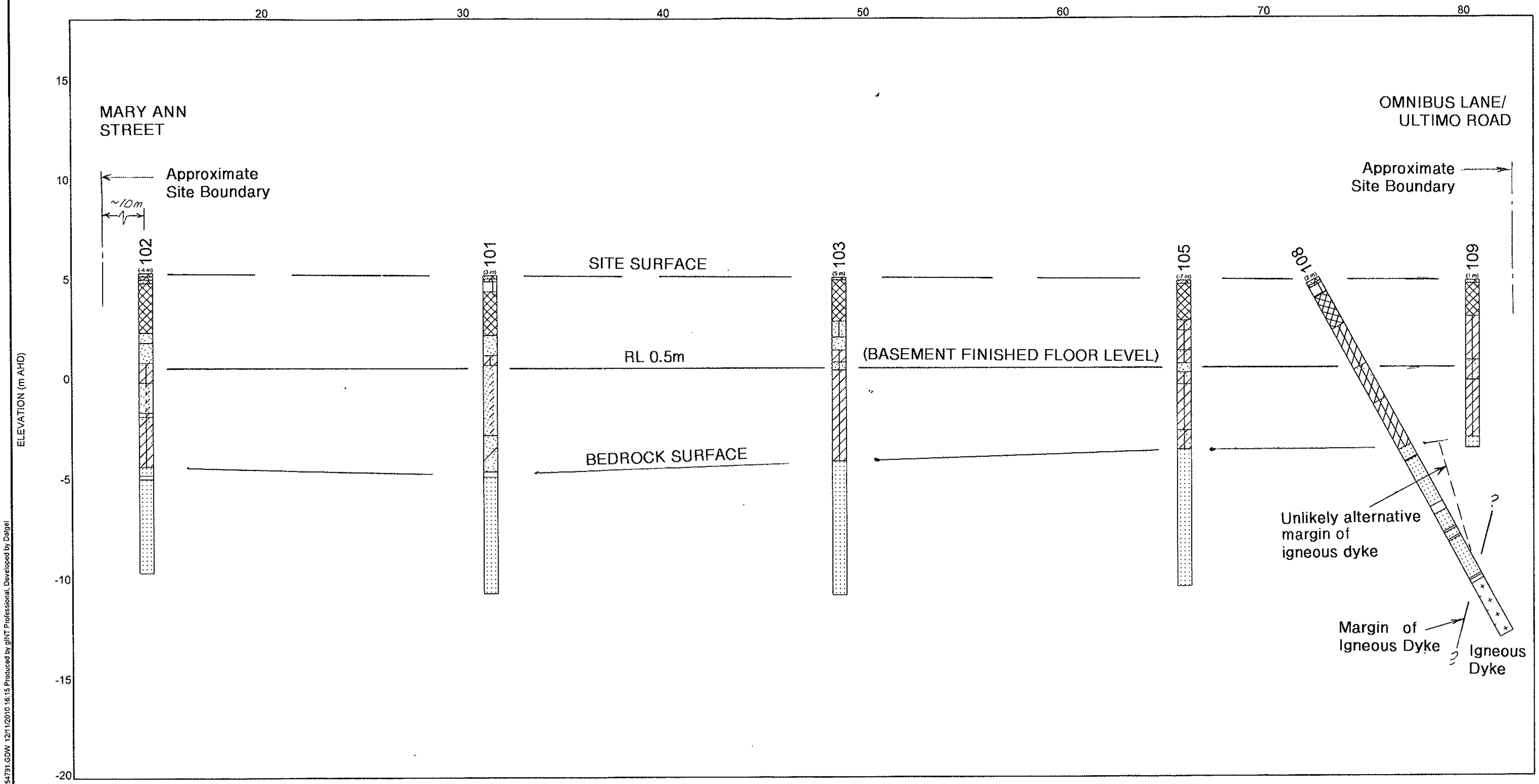


| | | | | |
|---|---|---|---|---|
| BRICK | CORE LOSS | SILTY CLAY | CONCRETE | SANDSTONE |
| CLAYEY SAND | SAND | SILTY SAND | DOLERITE, DIORITE | |
| CLAYEY SILTY SAND | SANDY CLAY | SILTY SANDY CLAY | FILL | |

# GRAPHICAL BOREHOLE SUMMARY: SECTION LINE D–D

| No. | Amendment Description | Initials | Date |
|---|---|---|---|
| | | | |

A3 Original | This sheet may be prepared using colour and may be incomplete if copied | Coordinate System: MGA94 Zone 56 | Height Datum: AHD

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DESIGNED: P.R.

REVIEWED: B.F.W.

**Jeffery and Katauskas Pty Ltd**

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NEW FACULTY OF BUSINESS BUILDING
14-18 ULTIMO ROAD, SYDNEY, NSW
GRAPHICAL BOREHOLE SUMMARY SECTION D-D

**SCALES**

H 1:200

V 1:200

**JOB NUMBER**

**24387WR**

**FIGURE NUMBER**

**5**

# 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.

**Jeffery and Katauskas Pty Ltd**
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

# 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

# UNIFIED SOIL CLASSIFICATION TABLE

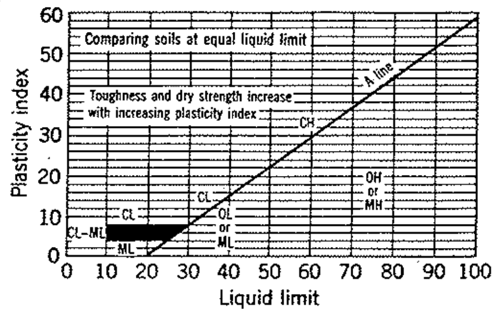
| Major division | Subdivision | Fines | Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights) | Group Symbols[a] | Typical Names | Information Required for Describing Soils | Laboratory Classification Criteria |
|---|---|---|---|---|---|---|---|
| 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 |
| | | | 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 | For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics | 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 |
| | | | Plastic fines (for identification procedures, see *CL* below) | *GC* | Clayey gravels, poorly graded gravel-sand-clay mixtures | | 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 |
| | | | 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 |
| | | | 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 for 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

Identification Procedures on Fraction Smaller than 380 μm Sieve Size

| Major division | Subdivision | Dry Strength (crushing characteristics) | Dilatancy (reaction to shaking) | Toughness (consistency near plastic limit) | Group Symbols[a] | Typical Names | Information Required for Describing Soils |
|---|---|---|---|---|---|---|---|
| Fine-grained soils: More than half of material is *smaller* than 75 μm sieve size | 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 | For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions |
| | | Slight to medium | Slow | Slight | *OL* | Organic silts and organic silt-clays of low plasticity | |
| | 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 | Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*) |
| | | High to very high | None | High | *CH* | Inorganic clays of high plasticity, fat clays | |
| | | 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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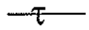
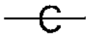
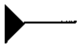
## LOG SYMBOLS

| 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<br>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<br>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. |
| | T 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
November 2007

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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 | |