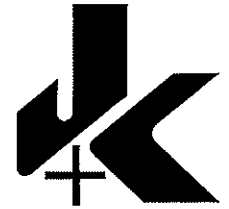




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9.13 Geotechnical Investigation Report



REPORT

TO

GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED GLOBAL SWITCH 2 BUILDING

AT

CORNER PYRMONT AND QUARRY STREETS, ULTIMO

23 April 2009

Ref: 22706VTrpt

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TABLE OF CONTENTS

1	INTRODUCTION	1
2	INVESTIGATION PROCEDURE	2
3	RESULTS OF INVESTIGATIONS	4
3.1	Site Description	4
3.2	Subsurface Conditions	6
3.2.1	Regional Geology	6
3.2.2	Previous Boreholes	7
3.2.3	Recent Boreholes and Test Pits	8
3.3	Laboratory Test Results	11
3.4	Rock Face Mapping	11
4	COMMENTS AND RECOMMENDATIONS	13
4.1	Summary of Principal Geotechnical Design Issues	13
4.2	Excavation	15
4.2.1	Potential Vibration Risks	16
4.3	Groundwater and Drainage	17
4.4	Rock Batter Slopes and Treatment	20
4.4.1	Existing Rock Face	20
4.4.2	Proposed Rock Faces	22
4.5	Retaining Walls	25
4.6	Building Footings	26
4.6.1	Footings at Crest of Proposed Cuts	28
4.7	Basement Floor Slabs	31
4.8	Pavements	31
4.8.1	Subgrade Treatment	32
4.8.2	Engineered Fill	32
5	SUMMARY OF FURTHER GEOTECHNICAL WORK	33
6	GENERAL COMMENTS	34

TABLE A: SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

TABLE B: ROCK UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS



**BOREHOLE LOGS 1 TO 4 INCLUSIVE WITH ROCK CORE PHOTOGRAPHS
BOREHOLE PUMP-OUT TEST RESULTS FOR BH1 AND BH3**

FIGURE 1: GEOTECHNICAL SITE PLAN

FIGURE 2: GRAPHICAL BOREHOLE SUMMARY

FIGURE 3: TEST PIT 5 CROSS SECTIONAL SKETCH

FIGURE 4: TEST PIT 6 CROSS SECTIONAL SKETCH

FIGURE 5: TEST PIT 7 CROSS SECTIONAL SKETCH

PLATE 1: VIEW OF WESTERN ROCK FACE – CH9.5M TO CH42M

PLATE 2: VIEW OF WESTERN ROCK FACE – CH40M TO CH77M

PLATE 3: VIEW OF WESTERN ROCK FACE – CH75M TO CH107M

PLATE 4: VIEW OF SOUTH-WESTERN ROCK FACE – CH115M TO CH138M

PLATE 5: VIEW OF SOUTHERN ROCK FACE – CH135M TO CH160M

FIGURE 7: OVERHANG A CROSS SECTIONAL SKETCH

FIGURE 8: OVERHANG B CROSS SECTIONAL SKETCH

FIGURE 9: OVERHANG C CROSS SECTIONAL SKETCH

FIGURE 10: OVERHANG D CROSS SECTIONAL SKETCH

FIGURE 11: OVERHANG E CROSS SECTIONAL SKETCH

FIGURE 12: OVERHANG F CROSS SECTIONAL SKETCH

FIGURE 13: MAPPING SYMBOLS

FIGURE 14: INFERRED GEOTECHNICAL SECTION A-A

VIBRATION EMISSION DESIGN GOALS SHEET

REPORT EXPLANATION NOTES

**APPENDIX A – FACTUAL INFORMATION FROM PREVIOUS 2001 GEOTECHNICAL INVESTIGATION
APPENDIX B – EARTH TECHNOLOGY SOLUTIONS PTY LTD REPORT (PROJECT NO. ET337 DATED
FEBRUARY 2009)**



1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Global Switch 2 Building in Pymont Street at its intersection with Quarry Street, Ultimo, NSW. Mr David Guth of Global Switch Property Australia Pty Ltd commissioned the investigation, on the basis of the scope of work offered in our proposal, Ref: P30450VTProp, and our subsequent email to Connell Wagner Pty Ltd (now known as Aurecon Australia Pty Ltd). The report was first issued in draft form for comment on 24 March 2009. This report supercedes the draft report.

We understand that it is proposed to construct a multi-level building with two and a part third (basement) level below ground. The new building is also to have a link to the existing Global Switch Building. The basement wall will need to be dry and watertight due to the proposed sensitive use of these areas. The construction of the basement would probably require bulk excavation generally of the order of 12m to 17m below existing levels. There is to be an at-grade car park, dock and turntable adjacent to the northern end of the building. Column loads of 15,000kN to 25,000kN have been advised by Aurecon.

Jeffery and Katauskas Pty Ltd completed in 2001 subsurface explorations at the site of the existing Global Switch Building and for cranes which were to be located in its adjacent at-grade car park (the subject site); refer to Report Ref. 15637VTrpt, issued 18 January 2001. This previous investigation comprised six boreholes (BH1 to BH6 inclusive), four from within the basement of the building and two at the western side of the at-grade car park. At the time of that investigation, that site contained the Government Printing Office Building. This building has subsequently been extensively refurbished, with some existing columns strengthened and new building columns constructed to accommodate increased building loadings. We subsequently provided advice and inspected some footing excavations during the refurbishment



works. No footings along the line of columns in the general vicinity of the existing rock face were inspected.

The borehole location plan and the borehole logs from the 2001 investigation are attached in the Appendix A to this report.

The scope of the current investigation was limited to obtaining information on subsurface conditions at a further seven locations as a basis for comments and geotechnical recommendations to assist the structural engineers and builders with the design and construction of the proposed development, including excavation, retention, groundwater issues, footing, floor slab and pavement design.

Electrical earth resistance testing has also been undertaken by a specialist consultant, Earth Technology Solutions Pty Ltd and a copy of their report is attached in the Appendix B.

A summary of the principal geotechnical issues for the proposed development are presented in Section 4.1.

2 INVESTIGATION PROCEDURE

The investigation comprised the drilling of four boreholes (BHs 1 to 4) using our JK500 truck mounted drill rig and three, hand excavated test pits. The boreholes were initially drilled using spiral flight augers to depths between 0.24m and 0.7m below existing surface levels. The boreholes were then extended by rotary diamond core drilling techniques, using an NMLC triple tube core barrel with water flush, to final depths ranging from 14.0m to 20.5m. Prior to drilling, each test location was checked by a specialist sub-contractor for buried services using electronic detection equipment, after referring to 'Dial Before You Dig' services drawings. The test pits



were excavated adjacent to the eastern perimeter wall of the existing Global Switch Building in an attempt to expose its footing and its foundation.

The test locations, as shown on Figure 1, which is based on the supplied survey plan of the site, were set out by taped measurements from existing site features and inferred site boundaries. The approximate location of two of the previously drilled boreholes is also indicated on Figure 1.

The state of compaction of the fill and strength of the subsurface soils were assessed from our observations during excavation of the test pits and of drilling resistance. The strength of the sandstone bedrock within the augered lengths of the boreholes was assessed from observation of drilling resistance when using a tungsten carbide ('TC') auger bit and examination of the recovered rock cuttings. The strength of the sandstone bedrock within the cored length of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with the results of rock strength testing.

Groundwater observations were made both during and on completion of borehole drilling. Further longer term groundwater monitoring has been facilitated by installation of slotted PVC standpipes in BH1 and BH3. Inclinometer casing has also been installed in BH4. Groundwater levels were monitored in these standpipes during borehole pump-out tests on 25 February 2009 and were re-measured on 9 April 2009.

The fieldwork was carried out under the direction of our engineering geologist, Mr Janak Patel, who was present on a full-time basis and set out the test locations, directed the buried service scans, nominated sampling, logged the encountered subsurface profile in the boreholes and test pits, and mapped the existing rock faces.



The surface reduced levels shown on the attached borehole logs were estimated by interpolating between spot levels indicated on the provided survey plan (Dwg No 113185001 Sheet 1, Rev 01, dated 9 January 2009) prepared by Hard and Forester Consulting Surveyors. The site datum is Australian Height Datum (AHD).

The borehole logs and test pit sections are included with this report, together with a Standard Set of Notes, which describes the methods and procedures employed in the investigation and their limitations and the logging terms and symbols used.

The recovered rock core was returned to Soil Test Services Pty Ltd (STS), a NATA registered laboratory, where it was photographed and selected sections of core subjected to Point Load Strength Index Tests ($I_{s(50)}$). The core photographs are attached opposite the relevant borehole log and the Point Load Strength Index tests are indicated on the borehole logs and are summarised in Table A. UCS tests were also carried out on four selected rock core samples and the results are given in Table B of this report.

Environmental screening of the site soils and groundwater was outside the agreed scope of this investigation.

3 RESULTS OF INVESTIGATIONS

3.1 Site Description

The site is located mid-slope on the north-east side of the Pymont ridge which falls from Harris Street to Darling Harbour. Pymont Street forms the eastern boundary beyond, which is the light railway, Darling Drive, then The Exhibition Centre in Darling Harbour. The light rail is around 5m lower than the footpath in Pymont Street. The footpath was retained by a two tiered, mortared sandstone block wall, which appeared to be in a reasonably good condition given its age. The lower



section of the wall appeared to be founded on a near vertical sandstone rock face estimated to be approximately 1m high when viewed from Darling Drive.

An elevated roadway forming part of the Western Distributor lies to the north. Quarry Street forms the southern boundary while further south, there is the low rise Bullecourt Development.

The site area covers an overall plan area of approximately 160m by 70m and contains the existing multi-storey "Global Switch" brick building on its western side with a lower level open car park on its eastern side. The Global Switch site is terraced down the hillside and cut and fill earthworks appear to have been undertaken to form the basement floor level of the building. The basement floor level is around 2m below the level of Harris Street at the western side of the building, indicating some excavation into the slope was required, while the eastern side of the basement appears to be over retained fill and is elevated above surrounding ground levels.

Offset some 1m to 2m from the eastern perimeter of the Global Switch building is a sandstone block, brick or concrete retaining wall, which has an approximate north-west/south-east alignment. This retaining wall is generally above a sub-vertical rock face previously excavated into the sandstone. The retaining wall and the underlying rock face generally have an overall height of approximately 3.7m.

The proposed Global Switch 2 Building site consists of the terraced area at the toe of the rock face. Adjacent to the rock face, the ground surface is relatively flat, then further to the east steps up a few hundred millimetres. This vacant site is covered with asphaltic concrete (AC) and concrete surfacing, and was being used as a car park. The detail survey generally indicates that surface levels are generally between RL 9.0m and RL 9.1m on the eastern side and between RL 8.7m and RL 8.9m on the western side of the car park terrace. The proposed development site also extends



below the Western Distributor and consists of an AC car park with an entry in its north-east corner. The north-east end of the site is retained by a brick wall which runs along the eastern boundary. This wall varies in height up to about 0.6m.

The footpath along Pymont Street is paved with AC and contains a line of medium sized trees. The Quarry Street footpath is located above a rock face and is also AC paved. Both roads are surfaced with AC, with concrete kerbing.

3.2 Subsurface Conditions

3.2.1 Regional Geology

The Sydney Geological Map shows the area to be underlain by sandstone bedrock belonging to the Hawkesbury Sandstone Formation (coarse to medium grained quartz sandstone with very minor shale and laminate lenses). This map profile does not take into account the soils derived from in-situ weathering of the sandstone or earthworks (eg. cutting and filling) that have previously been undertaken at the site and its surrounds. Note that based on our observations, the surface of the sandstone appears to step down from Harris Street towards the east to the light rail tracks. Fill and alluvial/estuarine deposits are indicated in the flat Darling Harbour area to the east.

The map also indicates that a dyke runs across the general area from the south-east towards the north-west. In June 2003, we observed the nearby excavations at the Bullecourt site from Pymont Street. These excavations had exposed sandstone bedrock apart from the north-west corner which had been covered by an anchored concrete wall. The sandstone adjacent to the Pymont Street frontage of that site appeared to be exposed from road level; however, a small section towards its southern end was also covered with concrete. We understand that the dyke crossed this neighbouring Bullecourt site.



3.2.2 Previous Boreholes

The boreholes drilled in 2000 by Jeffery and Katauskas Pty Ltd inside the existing basement area of the Global Switch Building (BH1 to BH4 inclusive) penetrated concrete pavements, varying in thickness from 380mm to 450mm, which were underlain by silty sand fill. The fill contained varying amounts of sandstone and igneous gravel and building rubble (ceramic pieces and glass fragments) and was assessed to be in a poorly to moderately compacted condition. Sandstone bedrock was encountered below the fill at depths ranging from 0.58m to 2.11m below basement floor level.

The sandstone was distinctly to slightly weathered and of medium to high strength with estimated UCS values generally in the range of 10MPa to 40MPa. This medium to high strength sandstone was generally bedded at 0° to 10° and contained occasional minor clay seams, extremely weathered seams or bedding partings. A lesser quality, low strength zone was intersected from 1.3m to 1.5m depths in BH1.

Two of the Boreholes, BH5 and BH6, were drilled on the western side of the at-grade car park (and to the east of the existing building). These boreholes intersected asphaltic concrete, 80mm to 100mm thick, over road base to depths of 0.35m and 0.18m, respectively. The road base covered the underlying distinctly to slightly weathered sandstone bedrock. This sandstone was predominantly of medium to high strength (with estimated UCS values generally between 12MPa and 54MPa) and contained some minor clay seams. However, the core in BH5, from 4.49m to 5.38m showed a series of minor clay or extremely weathered seams and an extremely low to very low strength layer.

The boreholes were 'dry' during and on completion of auger drilling in BH1, BH5 and BH6 and on completion of rotary wash boring in BH2, BH3 and BH4. For specific



details of the conditions encountered, reference should be made to the borehole logs in Appendix A.

3.2.3 Recent Boreholes and Test Pits

In general terms, the recent boreholes have disclosed a subsurface consisting of existing pavements and fill over weathered sandstone bedrock at depths between 0.24m and 0.7m below existing levels at the time of drilling. Groundwater was not encountered in the boreholes, prior to water being added during the coring process. For a more detailed description of the subsurface profile and groundwater levels encountered at each borehole location reference should be made to the attached borehole logs. A graphical summary of the borehole information is presented as Figure 2. The more pertinent details of the encountered variable subsurface conditions are presented in the following.

- **Existing Pavements** covered the ground surface at the boreholes and extends down to the underlying fill. In BHs 1, 2 and 3, these pavements comprised asphaltic concrete, 30mm or 40mm in thickness over concrete, 150mm or 200mm thick. Whilst in BH4, the pavement consisted of AC, 100mm thick.
- **Fill** comprising sandy gravel, which contained varying proportions of sand, igneous and sandstone gravel and sandstone cobbles. No fill was encountered in BH2. Assessing the degree of fill compaction is difficult, when gravel or larger size materials are present. Nevertheless, the fill was generally assessed to be generally in the moderately compacted range. The fill extended down to the weathered sandstone bedrock.
- **Weathered Sandstone Bedrock** was found below the pavements or fill profile at 0.7m in BH1, at 0.24m in BH2, at 0.62m in BH3, and at 0.53m in BH4 below existing levels. The sandstone revealed in the boreholes was



predominantly distinctly to slightly weathered and of medium to high strength. Iron indurated bands and cross bedded laminae were generally distributed through the rock profile. In BH3, the sandstone contained a reduced strength zone from 3.7m to 4.4m depth. Defects within the weathered sandstone comprised, extremely weathered seams and clay seams (between 3mm and 100mm thick), occasional extremely low strength bands, horizontal or sub-horizontal bedding partings, cross bedding and occasional (45° to 90°) joints. The core loss zones are inferred to be extremely weathered seams or fractured bands. Note that very high strength sandstone was intersected from 10.8m to 11.8m in BH2.

- **Groundwater:** The boreholes were 'dry' during and on completion of auger drilling. Immediately following coring, groundwater was measured at depths of 5.5m in BH2 and 5.0m in BH3. These water levels would most likely not have stabilised after the diamond core drilling as the introduction of water during coring obscures groundwater measurements. The groundwater levels after completion of drilling and prior to the pump-out tests are expected to be artificially high. We note that substantial water loss occurred during drilling of BH4 (on 25 February 2009). The lost drilling water flowed from the borehole almost certainly through open defects in the sandstone to surrounding areas; it is possible that the water migrated towards BH3. There was full return of drill water during coring of BH1, BH2 and BH3.

Subsequent monitoring of the groundwater was carried out in slotted PVC standpipes that had previously been installed in BH1 and BH3 on drilling completion. The groundwater levels in these boreholes were measured prior to and on completion of pump-out out tests on 25 February 2009 and again on 9 April 2009 following a wet period. A summary of the measured groundwater levels are presented in Table 1.



Table 1 – Groundwater Measurements

Borehole No	Screen Depths below Ground Surface (m)	Depths to Groundwater below Ground Surface (m)			
		On Completion of Coring	Prior to Pumping Out on 25/02/09	On Completion of Pump-Out Test on 25/02/09	On 9/0/4/09
1	8.17 – 14.17	-	6.1	9.0	6.0
2	Not Installed	5.5 (23/02/09)	-	-	-
3	1.5 – 10.2	5.0 (24/02/09)	5.2	5.4	8.3
4	Not Installed	-	-	-	-

The groundwater inflow rates during pumping are summarised in the borehole pump-out test result sheets. The initial inflow rates were relatively high in BH3 (about 39 litres/hour), possibly drill water migrating from BH4. The inflow rates, in both BH1 and BH3, were between 2 litres/hour and 3 litres/hour at completion of testing, indicating the short-term pump-out test had locally drained the nearby areas.

Based on our observations during drilling, the pump-out tests, and the subsequent groundwater level measurements, the groundwater appears to be flowing locally, in places, through defects within the weathered sandstone bedrock.

- **Test Pit Exposure of Existing Footings.** The test pits exposed the concrete footings below the perimeter wall of the Global Switch Building. The exposed edges of the footings were 160mm, 150mm or 350mm thick below the adjacent brick wall; refer to Figures 3, 4 and 5. The exposed concrete edges were in a good condition with no obvious damage or cracks. There was no plastic vapour barrier below the concrete. The footings, as exposed were



bearing on distinctly weathered sandstone of at least low strength, at depths of 0.57m in TP5, and at 0.65m in TP6 and TP7. The area adjacent to the wall had been backfilled with sand or silty sand fill, assessed to be in a poorly compacted condition.

3.3 Laboratory Test Results

The laboratory Point Load Strength Index Test showed reasonable correlation with our field assessments of rock strength. The approximate Unconfined Compressive Strengths (UCS) of the rock core, as shown on Table A, were estimated from correlations with the Point Load Strength Index Tests. The UCS values varied in the sandstone from 2MPa to 74MPa, with an average of about 29MPa.

Laboratory UCS tests were also carried out on four test specimens taken from the recovered rock cores. The ends of each specimen were trimmed prior to testing. The UCS results were between 17MPa and 21MPa; these results were less than expected. We note that two of these results were about 7% and 21% less than the UCS values estimated from Point Load Strength Index tests carried out on off-cuts retrieved during trimming of the cores.

3.4 Rock Face Mapping

Our engineering geologist, Mr Janak Patel has inspected the retaining walls and rock faces on the western and southern perimeters of the site. Our observations were made from safe vantage points at the top of the walls and from the existing car park at the toe of the rock faces. Some photographs of the retaining walls and rock face are presented in Plates 1 to 5 inclusive. Our observations have been referenced to chainages that have been set out by taped measurements along the toe of the faces. CH 0m is located at the metal fence, which runs to the east, at the northern end of the rock face. The rock face continues to the south-east to CH 137m, turns to the



south, and then to the east-north-east. We make the following generalised observations of the walls and rock faces:

- The rock faces varies in height from about 1m to 3m and run for approximately 137m adjacent to the existing building perimeter in a north-west to south-east direction, then turn to the east-north-east. Their eastern end is at CH 160m.
- The rock face exposes sandstone of assessed medium to high strength. The face is sub-vertical and appears to have been formed by pre-split blasting methods, as evidenced by numerous half round blast holes on the rock face.
- Mortared sandstone block, brick or concrete (in places) retaining walls run along the crest of the majority of the rock faces. These walls are between 0.3m and 2m in height and generally appeared in a good condition. The top of the walls are up to 3.7m above the car park platform.
- There are some wall sections, which are undercut to form overhangs above the underlying rock face (refer to Figures 7 and 10). In some areas, the walls extend down to car park level. In places (between CH 45m and CH 50m), the mortar between the sandstone blocks is cracked and the blocks may become dislodged from the face of the wall.
- Rock defects were exposed within the sandstone rock face and comprised horizontal to sub-horizontal (0° to 10°) bedding planes, spaced at approximately 0.1m to 2m intervals down the rock faces, and steeply inclined (60° to 90°) joints. Some bedding planes contain bedding partings (clay seams or extremely weathered seams), 50mm to 100mm thick.
- The joints exposed in the face generally strike either north-north-east/south-south-west or east-south-east/west-north-west (see Figure 1). Note mapping symbols are presented on Figure 13. These joints are spaced at approximately 1m to 5m intervals along the rock face. Some joints are open (2mm to 20mm gaps between the joint edges). There are occasional joints which are roughly parallel to the existing rock face.



- Some overhanging sections were observed either within the rock face or below the base of the existing retaining walls (see Figures 7, 8, 10 to 12 inclusive).

An inferred geotechnical model of the subsurface profile is illustrated in Section A-A of Figure 14; this is based on the rock face mapping, our site observations, the previous and current borehole information.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Design Issues

Based on the results of the boreholes, the test pits and rock face mapping and our understanding of the proposed development, we have summarised the following principal geotechnical issues to be considered in the planning, design and construction of the development:

- The proposed development will involve excavations of substantial volumes of soil and rock. Good engineering design, construction and maintenance practices should be adopted to maintain stability to adjoining sites and structures during excavation and in the long term, as well as reducing the risk of vibration damage to adjoining structures during excavation.
- Groundwater was not encountered during auger drilling. The use of water flush techniques during coring precluded further meaningful groundwater observations. Subsequent monitoring indicates that where the basement is proposed, localised groundwater inflow may occur through defects in the sandstone exposed in the sandstone cut faces and the sandstone floor. Further and longer term monitoring of groundwater levels in the standpipes should be carried out.
- Given the likely moderate to heavy building loads, footings should be founded on the underlying sandstone bedrock. Where bedrock is exposed or at shallow



depth after site earthworks, pad or strip footings may be used, but piles will be required where the depth to rock is deeper than about 1.5m.

- The proposed pavements may be constructed on the existing fill subgrade, provided it is prepared and proof rolled as detailed in Section 4.8.1. However, even following proof rolling, and treatment as required, of the fill there will still be a risk of poor pavement performance due to the underlying untreated fill. The only way to reduce such risks would be to excavate and replace the existing fill below the pavement area unless available records or additional testing confirms that the fill is sufficiently well-compacted.

Further comments on the above and other issues are provided within the following sections of this report. A summary of additional geotechnical work recommended are provided in Section 5.

Although only a limited subsurface investigation was completed, we believe sufficient information has been gained to be reasonably confident as to subsurface conditions. However, it will be essential during excavation and construction works that regular geotechnical inspections be commissioned to check initial assumptions about excavation and foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for excavation, shoring and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections.

We believe that a meeting of the design team would be fruitful, once the concept design is further advanced, in order to discuss geotechnical problems and solutions and further investigation in more detail.



4.2 Excavation

The building, which is to have two and three basement levels, will have perimeter and internal walls and columns supported off the sandstone bedrock. Construction of the basement will require substantial excavation rock for the bulk excavation and locally deeper for the lift wells. The sides of the excavation, which are to be in good quality sandstone, would be cut vertically and rock faces treated as discussed in Section 4.4.2.

The materials to be excavated will comprise existing pavements, fill, and in the most part the underlying sandstone bedrock. The fill and soils should be readily excavated by conventional earthworks plant such as large hydraulic excavators.

We expect that excavation of the medium strength or stronger sandstone will present hard ripping or "hard rock" excavation conditions. Ripping may only just be possible with a Caterpillar D10 or D11 dozer and a very generous allowance should be made for rock hammer assistance to the ripping, especially where rock defect spacing (bedding and cross bedding, joints, etc) is greater than about 0.5m and/or is heavily iron indurated. The use of an impact ripper is recommended. Excavation production rates are likely to be very low and shoe wear rates high.

A ripping hook on a heavy excavator (> 30 tonnes) working in tandem with a large rock saw would probably be an effective low vibration method for at least some of the work. Alternatively, hydraulic rock breaking equipment will be required for productive effective excavation. This equipment would also be required for detailed excavations such as footings or services, for which rotary grinders would also be useful.

The ease with which excavation of rock is achieved depends upon the equipment used, the skill and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.



Excavation and retention recommendations provided in this report should be complemented by reference to the Code of Practice Excavation Work, Cat. No. 312 dated 31 March, 2000 by WorkCover NSW.

4.2.1 Potential Vibration Risks

If a hydraulic impact hammer is used, considerable caution should be taken during rock excavation, as there will likely be direct transmission of ground vibrations to adjoining structures and buildings.

Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. This limit of vibrations should be reviewed once more definite details of the excavation and development staging are known to confirm that they are still suitable. Given the sensitive nature of the facilities in the existing Global Switch Building and the proximity of the freeway columns, a vibration monitoring management plan should be prepared for the site; the RTA may impose their own or additional requirements.

If large rock hammers are to be used, we recommend that the initial excavation in rock should preferably be commenced away from likely critical areas and instrument vibration monitoring undertaken to confirm whether the vibration limits are likely to be exceeded and to provide guidance on how far the rock hammer should be kept away from the site boundaries and boundary structures. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

If it is found that transmitted vibrations are unacceptable, which may occur with a 1000kg hammer (Krupp 960) within about 15m of existing structures, it may be



necessary to change to a small to medium sized excavator fitted with a light rock hammer no larger than 600kg (e.g. Krupp 350 size), or to a rotary grinder or jackhammers.

Vibrations induced by excavations can be reduced by alternative methods such as the following.

- Start the rock excavation away from likely critical areas.
- Maintain rock hammer orientation into the face and enlarge excavation by breaking small wedges off faces.
- Operate hammers in short bursts only, to prevent amplification of vibrations.
- Use smaller equipment (offset by a loss in productivity and economy and greater duration of the nuisance).
- Use line drilling, especially along excavation boundaries, to aid breaking and trimming.

Another means of reducing vibrations would be to cut or excavate a trench (with low energy equipment) along the sides of the excavation using a rock saw and then carry out bulk excavation with a very large (Caterpillar D11) dozer.

In addition, we recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. The contractor should also be provided with a copy of this report to make his own judgement on the most appropriate excavation equipment.

4.3 Groundwater and Drainage

The RL of the site is currently around +9m AHD and the proposed (basement) excavations will therefore extend well below sea level. Limited information is available from our boreholes drilled during the previous and recent investigations, however, the rock excavation will expose joints, bedding partings and other defects,



both in the rock faces and across the basement floor. Groundwater was measured as high as 5.2m depth (at about RL 3.9m) in BH3 and at 6.1m (about RL 2.9m) in BH1 immediately prior to the pump-out tests or more recently at depths of 8.3m (RL 0.7m) and 6.0m (RL 3.0m), respectively. We note that earth resistivity testing has identified a relatively conductive groundwater table within the sandstone bedrock at about 7m depth (ie. at about RL 2m).

We understand that a semi-tanked structure is proposed with walls, propped by the floor slab, retaining any overburden and sealed into the sandstone at the crest of the proposed vertical rock walls. Some groundwater inflows into the proposed excavations will occur as local seepage flows at the soil/rock interface and through joints and bedding planes within the sandstone bedrock. Where a suitable drainage void is to be provided behind the permanent basement walls, open perimeter drains should be provided to collect seepage flowing down the vertical rock faces for diversion to a permanent dewatering pump. The extent of groundwater flow through defects cannot be accurately predicted from boreholes, which only represent a very limited portion of the site. An assessment of likely seepage and required pumping capacity would best be made following completion of the bulk excavation, when seepage could be observed. Our experience in Darling Harbour, where several basements have been constructed well below sea level, indicates that a direct hydraulic connection between the open joints and Cockle Creek is unlikely and that much of groundwater flow is usually relatively minor and comes from a few discrete locations. Locally higher flows tend to occur at dykes or major defects in the rock but these do not appear to exist at this site. Pump-out tests in the standpipes indicate groundwater seepage should decrease substantially when excavations have initially drained the local area. Initially, the seepage inflows into the boreholes were measured to be as high as 39 litres/hour. The final inflow rates in the boreholes decreased to about 2-3 litres/hour; these rates are applicable to the borehole geometry. Note that these inflow rates may be artificially high due to the use and, in BH4, the loss of drill water during coring. Seepage volumes into the excavations are



expected to be controlled by the use of surface drains and sump and pump systems at basement level during construction.

We recommend that complete and permanent drainage be provided behind the basement walls. In addition, a free draining, drainage layer should be provided below the basement floor slabs to safeguard against the possibility of groundwater pressure causing an uplift pressure and to drain water charged rock defects. The relatively minor to modest groundwater flows would be able to drain through a free draining gravel bed below the floor slabs. If locally high or persistent inflows occur from defects below sea level, which would be unusual, curtain grouting techniques may be used to control the groundwater flowing through open or infilled defects in the rock.

For preliminary budgeting purposes, allowance should be made for a free draining gravel bed, 300mm thick, with 100mm diameter slotted pipes installed in 'rock-saw' slots cut into the sandstone floor on say a 4m grid. The piped drains below the gravel bed should be graded to sumps with an automatic fail-safe pump system for discharge of collected seepage to the stormwater system. Appropriate waterproofing will also be required for the permanent walls in contact with the excavated areas.

Further and longer term monitoring of groundwater levels in the standpipes should be carried out. It is recommended that the excavation also be carefully monitored in conjunction with the groundwater monitoring in the borehole standpipes, to confirm site drainage requirements.



4.4 Rock Batter Slopes and Treatment

4.4.1 Existing Rock Face

If the existing rock face is to be left at its present location, several existing features will require stabilisation and/or treatment. Note that some of the overhangs would be removed if the existing cut is trimmed to a line closer to the existing building. These areas in the rock face requiring treatment should be clearly identified with paint markings so that proposed remedial work can be easily communicated to, and undertaken by the contractor.

- The undercut sections of the rock faces and retaining walls, as shown in Figures 7 to 12, should be underpinned down to competent sandstone (slightly below the car park level). This work will require careful stripping of vegetation to allow closer geotechnical inspection. Once these areas have been cleared, further geotechnical inspection will be required to confirm the final extent of the proposed stabilisation measures, in conjunction with the contractor, as some new defects or features may be exposed.
- We recommend that reinforced shotcrete walls be constructed to provide the underpinning support to these undercut rock faces and retaining walls. The shotcrete walls would consist of SL61 or equivalent reinforcing mesh attached, and the face protected with shotcrete, at least 125mm thick, but with at least 50mm cover on either side of the mesh. The rock dowels may comprise N20 hot dipped galvanised steel bars, 1m to 1.5m long, installed and fully grouted into 65mm diameter holes drilled at about 10°-15° below horizontal. The dowels should be installed at 1m vertical and horizontal spacing. The exposed ends of the bars should be threaded to allow fitting of hot dipped galvanised steel face plates and nuts. Alternatively, the ends of the dowels may be cogged to allow attachment of the reinforcing mesh. Note that shotcrete may be treated to produce a sandstone-like appearance.



- Selective scaling down and removal of occasional small blocks of sandstone which may be dislodged from the rock face. This work should be carried out under geotechnical supervision.

The rock dowels are permanent, long term features and hence, must be of good quality materials, which are fully protected against corrosion.

We recommend that only experienced contractors, with appropriate insurances, be considered for these face treatment works. We can provide a list of contractors, if you wish.

Allowance must be made for groundwater seepage flowing from the rock faces and permanent and effective drainage provisions should be incorporated into all areas to be covered with shotcrete to discharge the seepage (eg. geotextile enclosed strip drains and weepholes). Weep holes should be provided through the shotcrete at say 1m horizontal spacing and should target defects from which seepage could flow (eg. joints or bedding separations). A nominal 20mm diameter PVC pipe should be inserted a minimum length of 0.5m into the ground behind the shotcrete/pitching. The outside end of the PVC should be plugged with rags during shotcrete application. Some pre-drilling may be required for the weep holes. The embedded end of the PVC in the rock face should be slotted. Geofabric should be wrapped around the embedded end to entirely cover the slots.

The existing sandstone, concrete and brick retaining walls appear generally to be performing satisfactorily at present. Details of their construction are unknown to us and should be checked from available records. If records are available, we could carry out calculations to assess the strength and stability of the walls. Mortar is missing from joints between the blocks in the sandstone walls in places. Some blocks are also deteriorating. Where the walls are to be retained, we recommend



that at some stage allowance be made for re-pointing of the mortared joints and for replacing some of the blocks. The open joints (weepholes) at the base of the walls should not be filled but should be flushed out to enhance their long term performance.

4.4.2 Proposed Rock Faces

It is proposed to re-trim the existing rock faces and to excavate a further 12m to 17m depth into the rock to form the stepped basement. We anticipate that the good quality, medium strength or stronger, sandstone bedrock, without unfavourable defects, may be cut vertically and the face left unsupported. However, some temporary and permanent treatment for localised support of these rock cuts may be required. Such treatment can be necessary due to the presence of joints or other defects, which may form continuous planes of weakness affecting the stability of unsupported rock faces. Identification of specific zones requiring stabilisation is rarely possible, even when numerous boreholes have been drilled. The stability of battered cuts or near vertical cuts, even in good quality, low to medium strength or stronger rock, must therefore be subject to confirmation by an inspection by a geotechnical engineer. In addition to inspection, allowance should be made for spoon testing in jackhammer holes taken about 1.5m horizontally into the rock faces to confirm the presence of steeply inclined joints.

No excavation face should be allowed to advance more than 1.5m vertically between inspections and the excavation should be staged or stepped so that a whole face is not excavated 1.5m vertically between visits. The staging of the excavation would also allow assessment of the spacing and inclination of joints and other defects in the rock adjacent to critical structures adjacent to the excavation.



Care must be taken during rock excavation not to destabilise the rock foundation supporting the existing Global Switch Building on the western side of the site and the freeway footings to the north. It is highly desirable for there to be an offset, preferably 0.3-0.5m, to the cut face below the perimeter wall of the Global Switch Building to give some "construction tolerance".

If adverse defects are identified by the geotechnical engineer during the inspections, then stabilisation or flatter batters will be required. If there are only occasional bedding and joint defects in the rock, the face may only require protection by dowels, mesh and shotcrete or the permanent basement walls. The extent of shotcrete to temporarily protect the rock faces prior to construction of the permanent walls should be confirmed during the geotechnical inspections. Stabilisation may also require the use of rock bolts, mesh and/or shotcrete protection to support the large blocks or other rock face areas. It would be unusual to complete such an excavation without some form of support being required to the rock faces, though this may take forms other than rock bolting. The excavated rock faces may also contain pockets or seams of weathered shale, which require treatment.

A moderate provision for rock bolting and shotcrete and mesh should be included in the Contract Documents for works nominated following the geotechnical inspections. For preliminary budgeting purposes, our best guess would be to allow for installing rock bolts, on say a 2m grid, to support around 10-15% of the exposed rock faces. The rock bolts would possibly be 3m to 5m long galvanised N24mm diameter bars (with an Ultimate Tensile Strength of 220kN), threaded at the heads and fitted with galvanised square anchor plates, spherical washers and nuts. In places, SL81 mesh and 100mm thick shotcrete may also be needed to protect exposed shale bands or seams or 'fractured' zones over possibly 5-10% of the sandstone cut face. Rock-bolts may be designed for an allowable bond stress of 400kPa for sandstone bedrock of at least medium strength. Where deemed



necessary by the structural and/or geotechnical engineer, rock-bolts should be proof tested to 1.3 times the working load under the supervision of an experienced engineer independent of the bolting contractor. Rock-bolt group interaction must also be taken into account. Permanent bolts should have appropriate corrosion provisions.

An excavation methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation techniques, batter profiles, the proposed excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types, and contingency plans in case of exceedances. The excavation methodology must be reviewed and approved by the geotechnical engineer.

It is likely that the excavation will induce movements of adjacent ground that falls within the area of influence of the excavation. As excavation of the sandstone occurs, the rock mass will tend to move inwards towards the excavation along bedding planes, clay seams, etc. as it is stress relieved. With increasing depth of excavation, the bed undergoing excavation will also drag overlying beds with it as the lower bed moves towards the excavation. The extent of movement will depend on the strength of the rock between the bedding planes and the spacing of joints. As the beds move inwards, joints, etc. will start opening behind the excavated face and any structures on or in the rock also move. These stress-relief movements will decrease away from the excavated face, however, their magnitude will increase as the depth of excavation increases.

The major in-situ stress in the rock in the Hawkesbury Sandstone tends to be slightly east or west of north, and in this region, is possibly parallel to the Ultimo ridgeline. The sandstone rock face below the Global Switch building contained sub-horizontal bedding planes and sub-vertical joints that generally strike either north-east/south-



west or east-south-east/west-north-west. In view of the above, we would anticipate that higher relief movements would occur at the north and south ends of the proposed basement excavation.

Experience with excavations in sandstone indicates that lateral ground movements of around 0.5mm/m to 1mm/m of excavation depth may occur as a result of stress relief. The extent of influence may be defined as extending a horizontal distance from the excavation equal to at least the excavation depth. Vertical movements would be expected to be minor below adjacent structures. Hence, any existing adjoining building or structures which fall within this area of influence of the excavation should be assessed for risks of architectural or structural damage due to excavation induced movements. This risk of damage will depend on their sensitivity to horizontal and vertical deformations, structural loading, type and founding elevations of footings and their foundation conditions. All these factors should be carefully investigated and evaluated.

The final offset of the freeway column footing from the rock face will require detailed assessment following consultation with the Roads and Traffic Authority, who may impose some site specific design, construction or monitoring procedures. For example, they may require numerical modelling to confirm the impact of the proposed excavation, monitoring of excavation induced movements using an inclinometer during excavation (inclinometer casing has been installed in BH4 for this purpose), and surveying of surface monuments.

4.5 Retaining Walls

The existing and proposed retaining walls may be designed or checked on the basis of an 'active' lateral pressure coefficient, K_a , of at least 0.35 for the existing fill, provided some deflection is tolerable. The K value may be reduced to about 0.25 for sandstone of extremely low to very low strength rock, if any. Subject to



geotechnical inspection, no K values need to be taken into account for the sandstone of at least medium strength. Approximate bulk unit weights of 20kN/m^3 for the soils and 22kN/m^3 for extremely low to very low strength rock may be adopted. Walls which are to be subsequently propped by the permanent structure (e.g. by the upper ground floor slab) should be designed based on a higher lateral pressure coefficient, K, of at least 0.6 (or about 0.4 for extremely low to very low strength sandstone) using a trapezoidal earth pressure distribution.

The recommended lateral earth pressure coefficients assume almost horizontal ground surfaces behind the crest of the walls. If inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill should be taken as a surcharge load in the design.

Applicable hydrostatic pressures should be added to the lateral earth pressures, unless specific measures are taken to introduce complete and permanent drainage of the ground behind the walls. Any surcharge affecting the walls (e.g. footings, retaining walls and their backfill, the ground slope behind the wall, etc.) should also be taken into account in design.

4.6 Building Footings

It is recommended that all proposed building footings be founded within the sandstone to provide uniform support and reduce the potential for differential footing settlements. Strip and pad footings formed at basement level and founded in sandstone of medium to high strength may be designed for an allowable end bearing pressure of 6.0MPa. A safe shaft adhesion of 10% of the nominated safe bearing pressures may also be adopted for rock sockets in the medium to high strength sandstone under compressive vertical loading (ie provided excavation is not carried out within the zone of influence of the footing). Two-thirds of these adhesion values may be adopted in uplift. These adhesion values assume excavation is not carried



out within the zone of influence of the footing. The bearing and adhesion values assume footing bases have been cleaned of loosened or softened materials and sockets are free of smeared material (a special roughening tool is normally required to achieve this in bored piers).

For footings fully embedded into the underlying bedrock below the lowest building floor levels, an allowable lateral stress in the rock socket equal to one third of the allowable bearing pressure may be adopted. These passive resistance values assume excavation is not carried 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 socket should not be taken into account to allow for disturbance effects during excavation.

If the designer wishes to adopt the limit state design methods of the Piling Code, AS2159-1995, then the ultimate values of end bearing pressure may be estimated by multiplying the above recommended allowable values by Factors of Safety of 3. A Factor of Safety of 2 should be applied to the shaft adhesion values. We recommend that the ultimate values be multiplied by a geotechnical strength reduction factor, Φ_g , of 0.5. Higher reduction factors may be adopted but these will depend on the intensity and type of proving of the piles and their foundation. An appropriate load factor should also be applied to the proposed pile loadings.

The nominated rock bearing pressure is based on a serviceability criteria of deflections at the footing base/pile toe of less than or equal to 1% of the least footing dimension (or pile diameter). Footing settlements may also be estimated using the Elastic Moduli given in Table 2.

Footings on rock can also be designed using 'Limit State Design' principles as detailed in the paper "Foundation on Sandstone and Shale in the Sydney Region' by Pells, Mostyn and Walker, Australian Geomechanics, Number 33, Part 3, December 1998 (Pages 17-29). It must be emphasised that the use of limit state design to



adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate footing performance.

Table 2 – Elastic Moduli for Footings in Rock

Strata	Bulk Unit Weight (kN/m³)	Poisson's Ratio	Elastic Modulus (MPa)
Sandstone – extremely low strength	22	0.25	50 – 100
Sandstone – very low strength	22	0.25	100 – 200
Sandstone – low strength	23	0.25	400 – 500
Sandstone – low to medium strength	23	0.2	500 – 700
Sandstone – medium strength	24	0.2	1000 – 2000
Sandstone – high strength	24	0.2	2000

We recommend that all footing excavations be checked and approved prior to concrete being poured. In addition to inspection, the sandstone foundation should be spoon tested in jackhammer holes taken to 1.5 times the least footing width under a proportion of strip and pad footings designed using a safe bearing pressure of 6.0MPa. This testing is to confirm that seams or defects present below the founding levels are within tolerable limits. The presence of such seams would require a reduction in allowable bearing capacity or an increase in footing depth. Excavation of rock sockets will be difficult in the medium strength or stronger sandstone, requiring the use of hydraulic hammers or rotary grinders.

4.6.1 Footings at Crest of Proposed Cuts

The load carrying capacity of existing footings or proposed footings that are to be founded close to the top of a vertical rock face (eg the existing building or freeway pier footings or proposed slab down-turn footings) will need to be carefully assessed.



The safe bearing pressure would need to take into account rock strength, jointing and the influence of clay seams in the sandstone foundation materials as well as the magnitude and inclination of the applied loadings. A reduction of the previously nominated bearing pressure is likely to be necessary.

An allowable end bearing pressure of 2500kPa may tentatively be adopted for checking the load carrying capacity of existing pad footings that are founded on sandstone of at least medium strength and are located within 2.5m to 3m of the rock face along the eastern side of the existing Global Switch Building subject to vertical (concentric) loading. Jeffery and Katauskas Pty Ltd provided similar advice during the refurbishment of the Global Switch Building in February and August 2001. Only the lift well footing excavation adjacent to Column D10 was inspected by us at that time. We have not inspected any footing strengthening works which were carried out at pad footings located in the general vicinity of the existing rock face adjacent to the eastern side of the building. The perimeter wall of the building is supported on a concrete strip footing founded on sandstone initially of low, then medium strength; the bearing value below the wall footing should be reduced to 1500kPa, provided that there is an offset of at least 0.3-0.5m between the eastern edge footing and the proposed perimeter of the excavation.

We have been supplied with a Department of Main Roads drawing (Regn. No. 6003 412ER0049 Sheet No. SP-20). The drawing, which is stamped "work-as-executed", indicates that the nearby freeway column (adjacent to BH4) is supported on a pad footing founded at about RL 6.1m on sandstone bedrock. Plan dimensions of the footing are not shown on the drawing. The sandstone below this footing (at BH4) is predominantly distinctly to slightly weathered and of medium or high strength. Rock defects within the sandstone profile comprised occasional sub-horizontal (10°) bedding planes, clay seams or extremely weathered seams (5mm to 140mm thick) and a very low strength band. When assessed using "Foundation on Sandstone and Shale in the Sydney Region" by Pells, Mostyn and Walker, Australian Geomechanics,



Number 33, Part 3, December 1998 (Pages 17-29), the sandstone foundation is deemed suitable for the design minimum safe bearing capacity of 10 tons per square foot (about 1.1MPa) nominated on the drawing. It would appear that the proposed basement excavation would be as close as 2m from the edge of the existing footing. The excavated (vertical) rock face would therefore be located within the zone of stress influence of this freeway footing. Assuming that the column is subject to vertical (concentric) loading and the applied stress at foundation level is dispersed through the underlying sandstone foundation at 2V in 1H, the stress envelope would daylight at about 6.9m depth on good quality (high strength) sandstone with occasional, minor defects. Given this, no reduction to the 1.1MPa design bearing capacity would be required. It is anticipated that the rock face will expose clay and extremely weathered seams (between 4.95m and 6.25m in BH4), these features when expose will deteriorate. We recommend that these weak zones be dug out to a width of 350mm and backfilled with concrete or alternatively, the face may be supported by the permanent construction. Galvanised dowels installed and grouted into the medium or high strength sandstone and reinforcing mesh should be used to hold the concrete in place. Weepholes at regular 2m intervals or a strip drain should be provided at the rear of trimmed slot to allow potential seepage to drain from behind the concrete or the permanent construction.

The sandstone below proposed ground floor slab down-turn footing will generally be of medium strength and the 2500kPa bearing pressure may (subject to geotechnical inspection) be adopted for design. Preferably, an offset of at least 0.3-0.5m should be between the edge of the down-turn footing and the proposed perimeter of the excavation.

Further investigations (such as inclined boreholes) and/or inspections by a geotechnical engineer of rock faces near existing **or proposed** footings may be required once details of the footings and loadings have been determined by the structural engineer.



4.7 Basement Floor Slabs

On-ground basement floor slabs, which are constructed over the sandstone, will require no special treatment other than the removal of loose and softened material. Areas, which have to be built-up to infill low points in the excavations should be filled with properly compacted sub-base material.

Although we expect that some under-floor drainage will be required, this should be reviewed following further monitoring of groundwater seepage during and on completion of the excavations. The under-floor drainage (such as perimeter drains and/or a free draining gravel bed) should be installed with sumps for gravity or automatic pumped discharge of groundwater. If under-floor drainage is not installed, then the on-ground floor slabs will be subjected to uplift pressures from the groundwater; this may require additional mass or ground anchors.

4.8 Pavements

The design of pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of new fill imported to the site, as well as vehicle loadings and use. Lightly loaded pavements may tentatively be designed using a lower bound characteristic CBR value of 3% or a coefficient of subgrade reaction of 30kPa/mm (750mm plate) or a long term Young's modulus of 15MPa for the proof rolled and treated fill subgrade. These preliminary design values should be confirmed by CBR tests once initial earthworks design is complete and by inspection and testing during construction.

Concrete pavements and on-ground floor slabs subject to traffic loadings should be supported on a sub-base layer of RTA Specification 3051 unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD.



Concrete pavements should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements should preferentially be used in areas where heavy vehicles manoeuvre such as garbage bin and truck unloading areas.

4.8.1 Subgrade Treatment

Any remaining existing fill may be left in place below proposed pavements on the condition that the subgrade is proof rolled and appropriately treated. However, there is a chance that some settlement may still occur under pavements bearing on the existing fill, even after it is treated by proof rolling.

Following excavation to the proposed design levels, the exposed fill subgrade should be proof rolled using a 5 tonne dead weight smooth drum vibratory roller under the supervision of an experienced earthworks superintendent, geotechnician or geotechnical engineer to check for any unstable areas. During proof-rolling care should be taken to avoid vibration damage to any neighbouring structures or services or improvements. The vibrations should be monitored and the vibrations may need to be reduced or ceased if there is a risk of damage. Where unstable areas are encountered the area should be locally excavated down to a sound base and replaced with engineered fill as detailed in Section 4.8.2.

4.8.2 Engineered Fill

Engineered fill should preferably comprise well-graded granular material (ripped or crushed sandstone), free of deleterious substances and having a maximum particle size of 75mm. The sandy fill materials may be re-used provided unsuitable ('over-wet' and 'over-size') material and any deleterious material is excluded. However, we do not recommend the re-use of any clay materials that may be encountered. The well-graded granular fill for backfilling excavations or for raising site levels should be



compacted in layers of not greater than 150mm loose thickness, to a density between 98% and 102% of Standard Maximum Dry Density (SMDD).

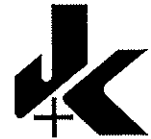
All fill should either be retained or battered to a slope of compacted fill of no steeper than 1V in 2H to prevent instability. All engineered fill areas should be over-filled and compacted and then the loose outer face of the fill should be cut back so that only well-compacted fill remains. We recommend a horizontal compacted fill platform extend beyond the pavement periphery by at least 2m, where possible. All exposed fill should be protected from erosion by quickly establishing a grass cover.

Density testing should be carried out at not less than the frequencies given in AS3798. At least Level 2 testing (but Level 1 where fill is to support movement-sensitive floor slabs/pavements) of earthworks should be carried out in accordance with AS3798. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not as part of the earthworks contract. We can complete these tests if you wish to commission us. The earthworks recommendations provided here should be complemented by reference to AS3798.

5 SUMMARY OF FURTHER GEOTECHNICAL WORK

As detailed in the previous sections of this report, further geotechnical work is recommended as follows:

- Establish dimensions, depth of embedment and material upon which the adjoining building pad footings are founded. The dimensions of the existing freeway pier footings should also be confirmed.
- Complete dilapidation surveys of the adjoining buildings and structures and assess the vulnerability of the structures to excavation-induced movement or vibration.



- Prior to design completion, it would be advisable to carry out a detailed assessment of the impact of the excavation on the existing building and freeway piers; the latter should be undertaken following consultation with the Roads and Traffic Authority, who may impose some site specific design, construction or monitoring procedures.
- Quantitative monitoring of transmitted vibrations during rock excavation using rock hammers.
- Monitoring of excavation induced movements using borehole inclinometers and conventional surveying techniques.
- Progressively inspect rock faces during excavation to confirm areas requiring treatment with rock bolts, mesh or shotcrete, etc. and the extent and volume of groundwater seepage.
- Inspect footing excavations to ascertain that the recommended foundation has been reached and to check initial assumptions regarding foundation conditions and possible variations that may occur. If higher bearing pressures are required core drilled boreholes so that the quality of the sandstone bedrock can be assessed and testing of the sandstone bedrock can be completed.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and 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.



The long-term successful performance of pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgement from an experienced engineer. Such judgement often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

The offsite disposal of soil will most likely require classification in accordance with the Department of Environment & Climate Change (NSW) guidelines as Virgin Un-Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous



waste. We can complete the necessary classification and testing if you wish to commission us. As testing requires about seven days to complete, allowance should be made for such testing in the construction program unless testing is completed prior to construction. If contamination is found to be present then substantial further testing and delays should be expected. We strongly recommend this issue be addressed prior to commencement of excavation on site.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. 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.

Tony Walker
Associate

Fernando Vega
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

Ref No: 22706VT
 Table A: Page 1 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH	$I_{S(50)}$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
	m	MPa	(MPa)
1	1.28-1.30	0.9	18
	2.22-2.25	0.9	18
	3.58-3.61	1.2	24
	4.45-4.47	1.1	22
	5.35-5.37	0.5	10
	6.28-6.30	0.7	14
	7.50-7.52	1.0	20
	8.18-8.20	1.2	24
	9.12-9.14	1.5	30
	10.53-10.55	2.2	44
	11.18-11.21	1.7	34
	12.52-12.55	0.7	14
	13.68-13.71	0.5	10
	2	0.74-0.77	1.5
1.47-1.50		1.7	34
2.54-2.57		1.3	26
3.46-3.49		2.2	44
4.39-4.42		2.0	40
5.30-5.33		2.0	40
6.30-6.33		2.6	52
7.32-7.35		1.6	32
8.29-8.32		1.3	26
9.41-9.44		1.6	32
10.75-10.78	2.4	48	
11.73-11.76	3.7	74	

NOTES: See page 3 of 3

Ref No: 22706VT
 Table A: Page 2 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{s(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	12.46-12.49	2.5	50
	13.54-13.57	0.5	10
3	0.91-0.94	0.7	14
	1.29-1.32	0.9	18
	2.62-2.65	1.1	22
	3.69-3.72	0.6	12
	4.46-4.49	0.1	2
	5.52-5.55	1.5	30
	6.45-6.48	1.3	26
	7.69-7.72	0.6	12
	8.68-8.71	2.0	40
	9.62-9.65	1.4	28
	10.36-10.39	1.6	32
	11.54-11.57	2.4	48
	12.14-12.17	1.4	28
	13.55-13.58	1.7	34
	14.66-14.69	2.6	52
	15.45-15.48	1.2	24
16.37-16.40	1.2	24	
17.42-17.45	1.2	24	
18.22-18.25	1.2	24	
19.40-19.43	1.3	26	
20.15-20.18	0.3	6	

NOTES: See page 3 of 3

Ref No: 22706VT
 Table A: Page 3 of 3

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH	$I_{s(50)}$	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH
	m	MPa	(MPa)
4	0.70-0.73	1.4	28
	1.66-1.69	1.0	20
	2.63-2.66	1.3	26
	3.53-3.56	1.3	26
	4.46-4.49	1.4	28
	5.51-5.54	0.8	16
	6.77-6.80	1.2	24
	7.57-7.60	2.1	42
	8.53-8.57	1.1	22
	9.57-9.60	2.5	50
	10.34-10.37	2.7	54
	11.47-11.50	2.5	50
	12.69-12.72	1.5	30
	13.65-13.68	2.2	44
	14.56-14.59	1.6	32
	15.51-15.54	1.5	30
	16.43-16.46	1.8	36
17.56-17.59	1.6	32	
18.64-18.67	1.5	30	
19.56-19.59	1.7	34	

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :

$$U.C.S. = 20 I_{s(50)}$$

Ref No:22706VT
 Table B: Page 1 of 1

TABLE B
ROCK UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS

Borehole Number	Depth m	Sample Diameter cm	Sample Length cm	L/d Ratio	Sample Mass g	Area cm ²	Volume cm ³	Wet Density t/m ³	Force N	Calculated UCS MPa
1	8.35-8.50	5.18	8.52	1.64	423.1	21.074	179.55	2.356	36000	17
2	9.20-9.30	5.18	9.27	1.79	443.3	21.074	195.36	2.269	43200	20
3	5.60-5.70	5.18	8.73	1.69	408.1	21.074	183.98	2.218	45250	21
4	12.38-12.69	6.08	10.27	1.69	715.7	29.033	298.17	2.400	48400	17

Note: Point Load Strength Index Tests were carried out on off-cuts retrieved during trimming of the specimens from BH1 and BH2; these indicated common UCS values of 22MPa.



Borehole No.

1

1/4

BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 9.0m
Date: 23-2-09 JK500 **Datum:** AHD
Logged/Checked by: J.P./Q

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION					0		-	ASPHALTIC CONCRETE: 30mm.t over CONCRETE: 150mm.t FILL: Sandy gravel, fine to coarse grained, light brown, with cobbles.	M	-	-	ON OBSERVED REINFORCEMENT APPEARS MODERATELY COMPACTED
					1			REFER TO CORED BOREHOLE LOG				PVC STANDPIPE INSTALLED, SLOTTED BETWEEN 8.17m AND 14.17m
					2							
					3							
					4							
					5							
					6							
					7							



Borehole No.

1

2/4

CORED BOREHOLE LOG

Client:	GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project:	PROPOSED GLOBAL SWITCH 2 BUILDING
Location:	CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT	Core Size: NMLC	R.L. Surface: ≈ 9.0m
Date: 23-2-09	Inclination: VERTICAL	Datum: AHD
Drill Type: JK500	Bearing: -	Logged/Checked by: J.P./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$										DEFECT DETAILS				
																	DEFECT SPACING (mm)	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.			
							EL	VL	L	M	H	VH	EH	500	300	100		50	30	10	Specific
		0		START CORING AT 0.70m																	
FULL RETURN		1		SANDSTONE: fine to medium grained, grey and brown, with iron indurated bands, bedded at 0-20°.	DW	M															
		2			SW																
		3																			
		4				H															
		5				M															
		6		as above, but with coarse grained sandstone bands.																	
		7																			

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JOB NO: 22706 VT

BH: 1

START CORING @ 0.70_m



1



2



3



4



5



6



7



8



9



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10

11

12

13

14

END OF BOREHOLE @ 14.17m



Borehole No.


2

1/3

BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 9.0m
Date: 23-2-09 JK500 **Datum:** AHD
Logged/Checked by: J.P./R

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	USO	DB DS									
DRY ON COMPLETION					0			ASPHALTIC CONCRETE: 40mm.t over CONCRETE: 200mm.t REFER TO CORED BOREHOLE LOG				NO OBSERVED REINFORCEMENT
					1							
					2							
					3							
					4							
					5							
					6							
					7							

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Borehole No.
2
2/3

CORED BOREHOLE LOG

Client:	GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD	
Project:	PROPOSED GLOBAL SWITCH 2 BUILDING	
Location:	CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW	
Job No. 22706VT	Core Size: NMLC	R.L. Surface: ≈ 9.0m
Date: 23-2-09	Inclination: VERTICAL	Datum: AHD
Drill Type: JK500	Bearing: -	Logged/Checked by: J.P./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX I _s (50)	DEFECT DETAILS			
								DEFECT SPACING (mm)		DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.	
								500	300	100	50
		0		START CORING AT 0.24m							
		1		SANDSTONE: fine to medium grained, grey, with iron indurated bands, bedded at 0-20°.	DW	H					
		2									
		3									
		4		as above, but grey and orange brown.					- XWS, 100mm.t		
		5							- Be, 0-5°, P, S, IS		
		6							- XWS, 20mm.t		
		7		as above, but grey.	XW	EL			- CS, 20mm.t		

FULL RETURN

ON COMPLETION OF CORING

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JOB NO: 22706VT

BH: 2

START CORING @ 0.24m

0

0.24_m

1

2

3

4

5

6

7

8



Borehole No.
2
3/3

CORED BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Core Size:** NMLC **R.L. Surface:** ≈ 9.0m
Date: 23-2-09 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** J.P. [Signature]

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX I _s (50)	DEFECT DETAILS															
								DEFECT SPACING (mm)	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.														
							EL	VL	L	M	H	VH	EH	500	300	100	50	30	10	Specific	General		
FULL RET- URN		8		SANDSTONE: fine to medium grained, grey, with iron indurated bands and coarse grained sandstone bands, bedded at 0-10°.	DW	H	X														- Be, 10°, P, S, IS		
		9					X															- Be, 0-5°, P, S, IS	
		10		as above, but grey and orange brown.				X														- XWS, 10mm.t	
		11		as above, but grey, with dark grey laminae.	SW			X														- Be, 20°, P, S, IS	
		12																				- Be, 5°, P, S, IS	
		12		SHALE: dark grey.																		- Be, 0°, P, S	
		12		SANDSTONE: fine to medium grained, grey and orange brown, with iron indurated bands.																		- CS, 40mm.t	
		13		as above, but with dark grey laminae.																		- XWS, 5mm.t	
		13																				- XWS, 3mm.t	
		14		END OF BOREHOLE AT 14.0m																		- XWS, 5mm.t	

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12

13

END of BOREHOLE @ 14.00m



Borehole No.

3

1/4

BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 9.1m
Date: 24-2-09 **Datum:** AHD
Logged/Checked by: J.P./~~JK~~

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION					0		-	ASPHALTIC CONCRETE: 40mm.t over CONCRETE: 200mm.t FILL: Sandy gravel, fine to coarse grained, light brown, with cobbles.	M	-	-	NO OBSERVED REINFORCEMENT APPEARS MODERATELY COMPACTED
					1			REFER TO CORED BOREHOLE LOG				PVC STANDPIPE INSTALLED, SLOTTED BETWEEN 11.41m AND 20.41m
					2							
					3							
					4							
					5							
					6							
					7							



Borehole No.
3
 2/4

CORED BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Core Size:** NMLC **R.L. Surface:** ≈ 9.1m
Date: 24-2-09 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** J.P./R

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$										DEFECT DETAILS			
																	DEFECT SPACING (mm)		DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.	
							EL	VL	L	M	H	VH	EH	500	300	100	50	30	10	Specific
		0		START CORING AT 0.62m																
		1		SANDSTONE: fine to medium grained, grey and orange brown, with iron indurated bands, cross bedded at 0-10°.	DW	M														
		2																		
		3																		
		4				L-M														
		5		as above, but grey, with occasional orange brown seams.																
		6				H														
		7		as above, but grey and orange brown.		M														

FULL RETURN
 ON COMPLETION OF CORING

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JOB NO: 22706VT

BOREHOLE NO: 3

START CORING @ 0.62m

0

0.62
m

1

2

3

4

5

6

7

8



Borehole No.
3
3/4

CORED BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Core Size:** NMLC **R.L. Surface:** ≈ 9.1m
Date: 24-2-09 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** J.P./g

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX I _s (50)	DEFECT DETAILS	
								DEFECT SPACING (mm)	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.
		8		SANDSTONE: fine to medium grained, grey and brown, with iron indurated bands and XW bands.	SW	M	VL L M H VH EH	500 300 100 50 30 10	- XWS, 15mm.t - XWS, 5mm.t - XWS, 25mm.t - XWS, 8mm.t
		9				H			
		10							- XWS, 10mm.t
		11							- XWS, 70mm.t
		12		as above, but grey, brown and red brown.					
		13							- Be, 15-20°, P, S
		14							

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FULL RETURN

9

10

11

12

13

14

15

16

17

18





Borehole No.

4

1/4

BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 9.0m
Date: 25-2-09 JK500 **Datum:** AHD
Logged/Checked by: J.P. / [Signature]

Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	USO	DB									
DRY ON COMPLETION					0		-	ASPHALTIC CONCRETE: 100mm.t	M	-	-	NO OBSERVED REINFORCEMENT
							-	FILL: Gravelly silty sand, fine to coarse grained, grey, with igneous gravel and sandstone cobbles.	DW	-	-	
					1		-	SANDSTONE: fine to medium grained, light grey and brown. REFER TO CORED BOREHOLE LOG				INCLINOMETER CASING INSTALLED FROM GROUND LEVEL TO 20.5m
					2							
					3							
					4							
					5							
					6							
					7							



Borehole No.
4
2/4

CORED BOREHOLE LOG

Client: GLOBAL SWITCH PROPERTY (AUSTRALIA) PTY LTD
Project: PROPOSED GLOBAL SWITCH 2 BUILDING
Location: CNR. QUARRY STREET AND PYRMONT STREET, ULTIMO, NSW

Job No. 22706VT **Core Size:** NMLC **R.L. Surface:** ≈ 9.0m
Date: 25-2-09 **Inclination:** VERTICAL **Datum:** AHD
Drill Type: JK500 **Bearing:** - **Logged/Checked by:** J.P./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX I _s (50)	DEFECT DETAILS	
								DEFECT SPACING (mm)	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.
		0		START CORING AT 0.53m					
		1		SANDSTONE: fine to medium grained, grey, with iron indurated bands, bedded at 0-10°.	DW	H	X		- Be, 0-5°, P, S
		2					X		- Be, 0-5°, P, S - Be, P, S - Be, 10°, P, S, CLAY COATED
		3					X		- Be, 10°, P, S
		4					X		- Be, 10°, P, S - XWS, 5mm.t
		5				M	X		- CS, 140mm.t
		5.5				VL			- XWS, 40mm.t - CS, 40mm.t
		6			DW	M	X		- J, 45°, P, S, CLAY COATED - Be, 10°, P, S, CLAY COATED
		6.13		CORE LOSS 0.13m					
		6.23		SANDSTONE: fine to medium grained, grey.	DW	M			
		6.33		CORE LOSS 0.10m					
		6.43		SANDSTONE: fine to medium grained, grey.	SW	H	X		- XWS, 20mm.t
		7							

NO RETURN

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JOB NO: 22706 VT

BOREHOLE NO: 4

START CORING @ 0.53m

0

0.53m

1

2

3

4

5

CORE LOSS
0.13m

6

CORE LOSS
0.10m

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15

16

17

18

19

20

CORE LOSS 0.2m

END OF BH @ 20.5m

