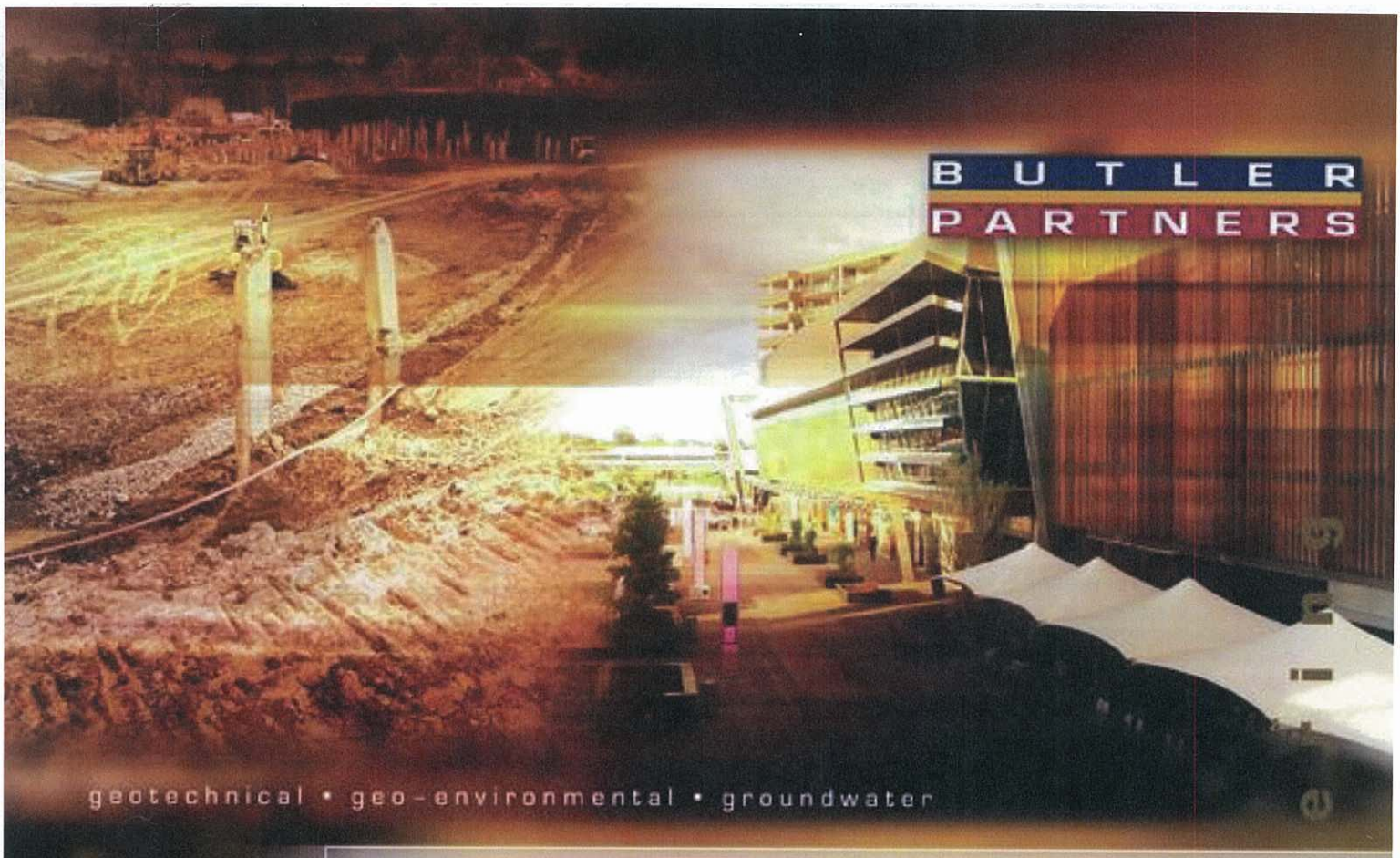

GEOTECHNICAL INVESTIGATION
HIGH RISE RESIDENTIAL DEVELOPMENT
29 - 35 CAMPBELL STREET
BOWEN HILLS

Prepared for: Metro Property Development Pty Ltd
Project No. 010-203A, 18 February 2011

High Rise Residential Development
29 - 35 Campbell Street, Bowen Hills
Project No. 010-203A
18 February 2011

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SECTION 1 - INTRODUCTION

1.1 Project

It is understood the proposed development is to consist of basement car parking levels a common podium and two residential towers approximately twenty seven and thirty storeys in height. It is anticipated that column loads will be in the order of 20 000k . Current planning envisages three basement car parking levels but site constraints and statutory re uirements may necessitate four basement levels. The approximate location and extent of the development site are shown on Drawing o. 1 attached.

1.2 Proposed Scope of Work

Based on prior knowledge of the immediate area including other major adjacent sites located in Campbell Street it was anticipated that the site ground conditions may comprise fill and soil to approximately 1m depth overlying generally low to medium strength tuff that contains numerous clay seams. A shallow groundwater table was not envisaged.

Based on the scope of the proposed development the anticipated ground conditions and the relatively limited current site access it was proposed to undertake geotechnical investigation by

- drilling and sampling six bores to 15m depth at accessible locations and
- installing two standpipe piezometers to enable groundwater level monitoring over time.

A preliminary contamination assessment and an acid sulfate soil ASS assessment of the site were undertaken at the same time as the geotechnical investigation. The results of the preliminary contamination assessment are reported separately and advice regarding the presence of ASS is contained herein.

sing the results of the fieldwork and laboratory testing outcomes it was proposed that a report would be produced that would provide geotechnical design information on each of the following topics

- Relevant subsurface information for nearby sites.
- Subsurface conditions and laboratory testing data.
- Earthworks and rock excavatability.
- Erosion and sediment control guidance.
- Shrink swell characteristics and moisture reactivity classification of soils.
- Groundwater conditions variability initial estimates of the rates of inflow.
- Temporary and long term dewatering options.
- Suitable foundation options as appropriate and design parameters – footings bored piers precast piles tension piles and recommendations on suitability of various options.
- Settlement characteristics of proposed foundation options and differential settlement estimates.
- Permanent and temporary retention options and design parameters.
- Soil rock anchor design parameters.
- Impacts of excavation dewatering and shoring on adjoining structures.
- Stability criteria for open excavations and advice regarding excavation staging including bench heights and the like.

- Advice on site traffickability after disturbance of the site at excavated levels.
- Advice on construction difficulties likely to be encountered.
- Pavement subgrade design parameters.
- □ptions for slab on ground components including the basement and external slabs – floating or suspended□hydrostatic or non hydrostatic.
- Settlement characteristics for slabs on grade.
- Advice on effects of proposed development on settlements of existing structures□both long term and short term □due to construction activities such as piling and compaction□
- Construction related advice.
- Pavement and slab subgrade parameters.

1.3 Commission

Based on the proposed nature of the development□the anticipated subsurface conditions and the scope of work□ a fee to undertake the investigation was presented in a proposal of 6 August 2010. Butler Partners Pty Ltd □Butler Partners□was subse□uently commissioned by Metro Property Development Pty Ltd□to conduct the geotechnical investigation as proposed□which was undertaken in consultation with the Robert Bird Group Pty Ltd□consulting engineers for the development. This report was first issued in draft for comment on 10 □ctober 2011.

SECTION 2 - THE SITE

2.1 Site Description

The site occupies an irregular shaped parcel of land covering an area of approximately 2□600m^2 bounded to the west by Mayne Road and to the south by Campbell Street. The east of the site is bounded partially by Hazlemount Street and partially by residential property. To the north of the site lie commercial premises. At the time of the investigation the site was occupied by a now vacant car sales yard predominately comprising car parking areas with a single storey sales office. The ground surface level generally falls from east to west between approximate RL19.9m and RL17.7m. A relatively recent aerial view of the site is shown in Photograph 1 and three general views of the site at the time of investigation are given in Photograph 2 to Photograph 4.

Photograph 1: Aerial view of site

Photograph 2: Panoramic view of the site looking north east to south west from the western side of Mayne Road

Photograph 3: Panoramic view of the site looking east to south from the vicinity of Bore 2

Photograph 4: Panoramic view of the site looking north-west to north-east from the southern side of Campbell Street

2.2 Geology

Reference to the Geological Survey of Queensland's 1:31,680 Brisbane geological sheets indicates that the site is located in an area mapped close to the boundary between the Brisbane Tuff comprising rhyolitic tuff, conglomerate and breccia with minor sandstone and shale and the Teneranleigh Fernvale Group comprising greywacke, siltstone, shale, chert, jasper and basic volcanics.

SECTION 3 - FIELDWORK

3.1 Drilling and Sampling Methods

The investigation comprised the drilling and sampling of five bores (Bores 1 to 5) using either a truck mounted Mobile B40L or a Dacro 350 drilling rig. The bores were initially drilled to between approximately 0.3m and 2.0m using solid flight augers. Casing was then installed and the bores were advanced until refusal or 'practical' refusal (in rock) using a 'TC' blade bit and wash boring techniques. The bores were then extended using (MLC coring techniques) to recover 50mm diameter core samples of rock, utilising 'mud' circulation for cuttings removal. Strata identification was based on inspection of cuttings recovered in the 'mud' circulation, supplemented with inspection of Standard penetration test (SPT) and rock core samples recovered. Pocket penetrometer tests were carried out within select cored sections of cohesive strata. On completion of drilling, Bores 2, 4 and 5 were backfilled with drill spoil and surface plugged with bituminous concrete. A standpipe piezometer was installed in Bore 1 and in Bore 3 to enable groundwater level monitoring over time. Of the original six bores proposed one bore was unable to be completed due to existing buildings in the south eastern corner of the site.

3.2 Test Locations and Supervision

Bore locations were set out by direct measurement from existing site features and are shown approximately on Drawing No. 1, attached. Ground surface levels at the bore locations were determined by interpolation from RPS Australia East Pty Ltd's *Detail Survey,* Drawing No. 103356-1, dated 30 July 2010.

A senior geotechnician set out the test locations, logged the subsurface soil profiles, determined the in-situ sampling and testing program and supervised the fieldwork.

SECTION 4 - INVESTIGATION RESULTS

4.1 Results

The subsurface conditions encountered at the bore locations are given on Bore Report sheets included in Appendix A using classification and descriptive terms defined in accompanying notes. Laboratory test result sheets are included in Appendix B.

4.2 Subsurface Conditions

4.2.1 Stratigraphy

For a description of the stratigraphy encountered in the bores the Bore Report sheets should be referred to. The stratigraphy encountered in the bores was variable however in broad summary the subsurface conditions encountered in the bores comprised a surface layer of between 0.4m and 1.7m depth approximately of pavement construction predominately granular fill and or isolated natural silty clay soil Bore 2 underlain by extremely low to medium strength tuff that graded with depth to very high strength rock in all bores with the exception of Bores 3 and 5.

In Bore 2 a zone of silty clay was encountered between 2.95m and 3.95m and was underlying a medium to high strength tuff. Pocket penetrometer readings undertaken within core samples of clay recovered ranged between 150kPa and 450kPa uncorrected.

In Bore 5 a zone of brecciated rock fault breccia was encountered between 11.7m and 15.0m depth. The fault breccia contained several clayey zones and an uncorrected pocket penetrometer test undertaken in a core sample of clay recovered indicated a reading of 350kPa. It is considered possible that the presence of the fault breccia may indicate that the base of the tuff layer had been encountered in Bore 5 or that the base of the tuff layer may not be far below the termination depth of Bore 5.

4.2.2 Rock Defects

In general two sets of rock joints discontinuities were intersected in Bores 1 to 5 which dipped at approximately 10° to 25° and 40° to 90° with isolated joints at 30° to 35° and were generally clean tight and 'healed' at approximately 1mm thickness. However crushed sheared zones with zones of clay were encountered in Bore 5 between 12.4m and 15.0m depth.

4.2.3 Rock Strength Inversions

It should be carefully noted that 'strength inversions' (i.e. 'lower' strength rock and or soils underlying 'higher' strength rock) were encountered in some bores. For example very high strength rock was underlain by high strength rock in Bore 1 at approximately 11.9m to 13.3m depth i.e. approximately RL7.9m to RL6.5m and high strength rock with medium strength zones was encountered underlying very high strength rock below 12.4m in Bore 4 i.e. below approximate RL5.8m.

4.3 Groundwater

Free groundwater was observed in the Bore 3 at 0.7m depth during initial auger drilling. No free groundwater was observed in Bores 1, 2, 4 or 5 during initial auger drilling. Beyond the depth of auger drilling, the use of water circulation for cuttings removal precluded further groundwater observations during drilling. The depth/RL of groundwater measured in the standpipe piezometers installed in Bores 1 and 3 are summarised in Table 1.

Table 1: Groundwater Measurements in Standpipe Piezometers

B	G O			
	12 O 2 1		18 F 2 11	
	D	R L	D	R L
1	0.82	RL19.0m	I ACCESSIBLE	
3	1.17	RL17.0m	1.32	RL16.9m

Possibly not fully recovered following drilling

It is considered possible that groundwater levels may be influenced by existing/ nearby development/services. It is also considered that the groundwater level beneath the site could vary with season/prevailing weather and that local 'perched' water tables may occur over the site.

4.4 Laboratory Testing

Selected soil samples were tested in a geotechnical testing laboratory to determine erosion and sediment control parameters/particle size distribution/plasticity and acid sulfate soil /ASS/potential and selected rock core samples were tested for measurement of rock strength.

4.4.1 Erosion and Sediment Control

Three selected samples recovered from the bores were tested to determine Emerson Class /umber/pH and electrical conductivity and a summary of the reported test results is presented in Table 2.

Table 2: Summary of Reported Emerson Class, pH and Conductivity Test Results

B	D	S D	E C N .	H	E C S
1	0.5 – 0.95	Fill	3	7.5	0.12
3	0.5 – 0.95	Fill	2	6.3	0.04
5	0.25 – 0.59	Fill	2	5.0	0.03

4.4.2 Particle Size Distribution

Three selected samples recovered from the bores were tested to determine particle size distribution using wash sieve grading techni/ues and the reported results are summarised in Table 3.

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Table 3: Summary of Reported Particle Size Distribution Test Results

B	D	D	S M C	G F [1]	S F [2]	S C F [3]
2	0.5 – 0.95	Fill	17.8	21	27	42
3	0.5 – 0.95	Fill	24.6	53	23	14
5	0.25 – 0.59	Fill	18.7	16	54	38

[1] Particle size 60 mm 2 mm [2] Particle size approximately 2 mm 0.75 mm [3] Particle size approximately 0.75 mm

4.4.3 *Plasticity*

One sample recovered from Bore 2 was tested for measurement of plasticity using Atterberg limit and linear shrinkage test methods. The reported test results are summarised in Table 4 and indicate that the sample tested is of medium plasticity.

Table 4: Summary of Reported Plasticity Results

L	D	S D	S M C	L L	L	I	L S	I D F ϕ' [1]	C
2	3.5 – 3.6	Silty Clay	27.5	47	29	18	8.5	28	CI

[1] Australian Standard AS1726 1993 *Geotechnical site investigation*

4.4.4 *Acid Sulfate Soils*

Two samples of the fill encountered in Bores 2 and 3 were tested for the presence of acid sulfate soils (ASS) using Chromium Suite analysis. The samples were kept on ice until transported to Australian Laboratory Services Pty Ltd (ALS) under standard chain of custody protocol and a summary of the test results is given in Table 5.

Table 5: Summary of Reported Chromium Suite Test Results

B	D	S D	H_{KCL}	T A A S	C R S – S_{CR} S	N A S
2	0.5 – 0.95	Fill	6.4	0.02	0.005	0.02
5	0.0 – 0.25	Fill	4.1	0.08	0.005	0.08

4.4.5 *Rock Strength*

Selected rock core samples recovered from the bores were tested for measurement of rock strength using Point Load Test ($I_s(50)$) methods and the test results are given on the relevant Bore Report sheets and a table of results is included in Appendix B.

SECTION 5 - COMMENTS

5.1 Ground Model

The results of the investigation indicate that the site is covered at the bore locations by a surface layer up to approximately 1.7m depth comprising pavement construction materials predominantly granular fill and isolated natural silty clay soils underlain by extremely low to low to medium strength tuff that generally graded with depth to very high strength rock and contained 'strength inversions'. In these ground conditions geotechnical design will need to consider at least but not exclusively the following key issues

- conditions and compaction status of the existing fill
- shrink swell characteristics of soils and rock
- earthworks and excavatability
- movement control to site boundaries and adjacent services etc.
- groundwater conditions and control construction and permanent within the proposed basement
- temporary and permanent excavation support
- effects on excavation stability of the presence of 'faults'/continuous clay coated joints etc. in the rock;
- potential settlement of adjacent services roads due to excavation support movement
- retaining wall pressures
- suitable foundation types
- settlement characteristics of proposed foundation options
- appropriate founding depths and bearing pressures
- variations in footing founding depths and founding conditions across the site
- presence of strength 'inversions' and zones of 'clayey' material possible brecciated 'contact' zone with depth in the rock
- slab and pavement subgrade properties
- possible construction difficulties
- possible settlements of existing adjacent structures
- settlement characteristics for slab on ground and
- site traffickability

5.2 Earthworks

5.2.1 Existing Fill

It is not known whether the existing fill material at the site is 'controlled' (i.e. it is not known whether the fill has been placed and uniformly compacted to an appropriate engineering specification. If any of the existing fill is required to support settlement sensitive elements and compaction control documentation does not exist or the specification used for filling is not appropriate for the proposed development then it is suggested that the existing fill be assumed to be uncontrolled.

If the fill cannot be shown to be controlled then consideration should be given to the potential for adverse variation to exist in both the composition and degree of compaction of the fill. The presence of voids within uncontrolled fill as well as potential soft loose zones or inclusions of deleterious materials may lead to potentially significant future total and differential settlements occurring possibly over relatively short distances.

5.2.2 Contamination

At the time of preparing this report the site was listed on the Department of Environment and Resource Management's (DERM Environmental Management Register EMR. Earthworks design planning and costing as well as groundwater control works must take into account the site's contamination status.

5.2.3 Excavatability

5.2.3.1 Bulk and Confined Excavation

Based on the results of the fieldwork excavation to the depths proposed would be expected to encounter some surface fill and soils overlying weathered tuff rock. The rock encountered in the bores ranged from extremely low to very high strength, and it is considered possible that zones of 'stronger' and/or 'less jointed' rock may also exist within the proposed excavation depth.

Excavation of fill soils and extremely low to low strength rock should be readily achieved in bulk excavation using a large hydraulic excavator. A large tractor D11 should be able to economically remove low strength and medium strength rock however supplementary rock breaking may be re uired to aid production in medium strength rock subject to existing rock fracturing. It should be noted that successful ripping would re uire the use of a large tractor and freedom to rip in all directions. If this is not possible it is likely that rock breakers will be re uired in lieu of ripping.

Bulk excavation of medium strength rock will require relatively major use of 'rock breaker' equipment unless joint spacing is moderately close less than 0.3m. In high strength or stronger rock with relatively few discontinuities it is likely that the most economic method of excavation will involve blasting followed by either ripping with a heavy tractor or use of a large rock breaker. Without blasting rock breaker excavation methods only would be expected to be very slow and potentially severely damaging to e uipment.

In confined trench footing etc. excavations in medium strength rock heavy rock breaker e uipment and slow excavation rates should be allowed for. In high strength or stronger rock blasting may be re uired in order to minimise e uipment damage and to increase production.

Due to the inherent jointing and bedding planes contained in the rock over break should be allowed for in pricing.

Based on past experience in similar conditions elsewhere in Brisbane the maximum economic depths of bulk excavation using various methods has been estimated at the location of Bores 1 2 3 4 and 5 and the estimates are tabulated in Table 6. The depths given in the table must be read in conjunction with the Bore Report sheets. The excavatability of rock is critically dependent upon the depth of weathering nature and distribution of rock defects i.e. persistent joints sheared extremely to highly weathered or highly fractured zones as well as strength, the 'size' of the excavation, the availability of plant and any site specific limitations.

Table 6: Preliminary Estimated Bulk Excavation Method Transition Depths

E M	E T D B G S B O Excavation Method and Another for 'Economic Bulk Excavation'				
	B				
	1	2	3	4	5
H E ▼ D11	1	1 – 2	5 – 6	1	3 – 4
R ▼ H R B ▼ B	1 – 2	1 – 2	5 – 6	1	4 – 5
	5 – 6	5 – 6	9 – 10	2 – 3	–

Possibly supplemented by heavy rock breaker work

5.2.3.2 Bored Piles

Heavy drilling e uipment will be re uired to drill medium strength or stronger rock. In high strength and stronger rock low to very low production should be allowed for in pricing combined with high bit wear. se of coring bucket pilot hole drilling methods will be re uired in high strength or stronger rock.

5.2.4 Traffickability

Following excavation to basement level traffickability for plant may be very difficult in general and particularly in the 'wet') in any areas of soil or extremely to very low strength rock and consideration should be given to the placement of a coarse granular working surface (e.g. 'crushed concrete') over any such exposed subgrade. The re uired layer thickness will depend on the type of plant proposed to traffic the site and should be specifically assessed on a case by case basis. However a minimum layer thickness of approximately 0.1m may be required for 'light' equipment.

In addition and depending on final basement level the 'clayey' rock encountered in Bore 5 below approximately 12m depth may be exposed in the base of the excavation and traffickability is likely to be difficult to impossible for most construction plant particularly in the 'wet'.

5.2.5 *Acid Sulfate Soils*

The very limited extent of clay soils encountered in the bores are considered to be residual soils derived from the in place weathering of underlying rocks of the Brisbane Tuff. It is considered that whilst site soils may be acidic because of the site geology they will not produce any additional acidity and are therefore not acid sulfate soils. An Acid Sulfate Soil Management Plan will therefore not be required for site development. However as a general precaution all collected stormwater should be checked for pH prior to off site discharge if the pH is outside the range 6.5pH to 8.5pH units it should be treated to bring it within this range prior to discharge.

5.2.6 *Erosion and Sediment Control*

Earthworks at the site is to comprise excavation to depths below street level of up to approximately 10m to 12m across the site area. Based on the results of the bores the stratigraphy that will be encountered by basement excavation will consist of variable strength tuff rock overlain by some thin fill and soil.

Assessment of erodibility dispersability of fill and soils on the site could initially be carried out using Brisbane City Council's Erosion Hazard Assessment Form (EHAF), which adopts 'scores' and 'trigger scores' to identify the erosion risk of sites due to the soils present site characteristics and disturbance proposed. Only sites with a high risk ranking would require a full Erosion and Sediment Control (ESC) Program.

The site will have fill and soil exposed during excavation until material with a very low erosion potential is intersected which varies greatly between approximately 1.0m (Bore 1) and 12.0m (Bore 5) below existing site levels. In addition the entire site is to be excavated below road level such that no direct run off will occur to stormwater but rather water will accumulate in depressions within the excavation. Therefore it is considered that the published BCC Erosion Hazard Form is not applicable to the site and a detailed ESC Program is not warranted. However stormwater will collect in the excavation during rain events and it is therefore considered that a stormwater management program will be required. A stormwater management program will need to address the collection testing and discharge requirements for stormwater and groundwater entering the site.

Although it is considered that whilst an ESC Program is probably not required, 'good construction practices' will still need to be adopted to control potential sediment movement from the site particularly when site levels are at or near ground level during the initial stages of construction and where excavated fill soils and weathered rock are being removed from site.

5.2.7 *Reactivity*

It is anticipated that extremely low to very low strength rock (if encountered) is likely to have the potential to display reactive behaviour (i.e. have a potential for shrink and swell movements associated with wetting up and drying back) and would therefore be expected to readily change volume with change in moisture content especially if disturbed and recompacted.

5.3 Anticipated Rock Structure

Positive excavation support will be required to maintain excavation stability and to control movement of any adjoining roads, services etc. In general, all excavations on site should be assessed by suitably qualified personnel as variations in geological structure and rock strength can occur over short distances, and these variations must be assessed as excavation proceeds.

Joint dip angles obtained from the bores and previous projects undertaken in the tuff rocks of Brisbane are considered to indicate that the rock jointing at the site probably mainly comprises a set of 'low angle' (i.e. 0° to 20°) and a set of 'high angle' (i.e. 50° to 80°) joints that are probably tight. ***This geological structure (and resulting potential rock failure mode) should be considered as preliminary only and must be subject to confirmation and adjustment based on the results of additional investigation as well as by inspection and mapping as the excavation proceeds.***

5.3.1 Continuity of Major Rock Discontinuities

A sheared/brecciated zone was intersected in Bore 5 below approximately 12m depth. The continuity of such discontinuities (and their dip/strike) cannot be accurately determined from the current bores and mapping during excavation will be required for confirmation. Consideration should be given to the possibility that the sheared/brecciated zones may lie close to or directly underlie the maximum depth of excavation or at shallower depths elsewhere within the proposed excavation.

Butler Partner's experience with the geotechnical design and construction of ten basements/excavations to depths between 18m and 40m in Brisbane is that 40% of the excavations encountered significantly inclined, low strength, continuous discontinuities (that generally were not intersected in the investigation bores), some of which did not become evident until the excavations were almost completed.

Considering that large scale, continuous discontinuities have been found in the Brisbane Tuff rock and that discontinuities in the form of sheared/brecciated zones have been intersected within Bore 5 at the site, it is strongly recommended that, until site mapping is undertaken during the actual excavation, large scale, continuous planar discontinuities should be assumed to exist at the site. To neglect their potential presence would require the acceptance of a significant risk that calculated excavation support loads could be significantly underestimated.

5.4 Temporary Batter Slopes

If movement sensitive services/buildings etc. are <u>not</u> located 'close' to excavation crests and geometry permits, temporary batter slopes may be adopted for temporary slopes to approximately 4m height. Provided slopes are protected from groundwater or surface water effects, the maximum preliminary cut slope angles given in Table 7 may be used with a relatively low risk of instability for <u>unsurcharged</u> batters. However, detailed inspection by an experienced geotechnical engineer will be required at the time of construction to confirm the stability of batter faces.

Table 7: Maximum Temporary Unsurcharged Cut Batter Slopes – 4m Maximum

M	S	M S
Fill uncontrolled	–	1 2.5H[1]
Silty Clay	stiff	1 2.0H[2]
Tuff	extremely low very low to low medium to high or stronger	1 1.5H 1 1.0H[3] 1 0.5H[3]

[1] Subject to inspection, uncontrolled fill may include 'less stable' zones that may require flatter batter angles and or positive support

[2] Not underlain by 'softer' soils

[3] Subject to jointing and confirmation by inspection and analysis possibly steeper

At the batter angles nominated in Table 7 there may be some localised slumping of batter slopes and it will be necessary to ensure that any such slumping will not adversely affect any services footpaths etc. located behind the slopes.

If insufficient space exists for the construction of cut batters at the slopes given in Table 7 or risk of some slumping cannot be accepted the excavation sides will need to be continuously supported in order to prevent instability.

5.5 Temporary Excavation Support Methods

It is considered that ground support for excavated faces could be provided by one or a combination of methods outlined below.

In areas of excavation not continuously supported it may be necessary to use steel mesh or heavy gauge galvanised wire as appropriate strung between points of rock anchor support to provide protection to personnel from isolated pieces of rock falling from faces.

Surface filling soils and extremely low to low strength rock will re uire continuous support to prevent instability. As a preliminary guide for costing the minimum depth of surface material potentially re uiring continuous support if cut vertical or near vertical for each bore has been estimated and is given in Table 8 below. These depths may need to be adjusted for the location depth and backfill uality of any adjacent service trenches etc.

Table 8: Estimated Minimum Continuous Support Depth

B	E M D M R C S
1	1
2	5
3	6
4	1
5	10 to 13

5.5.1 Soldier Pile and Shotcrete Lagging

Surface fill soils and extremely low to low strength rock will require continuous support to prevent instability and a system that has been used successfully in Brisbane comprises anchored cantilever solider piles installed in predrilled holes which in turn retain shotcrete lagging. Due allowance should be made in pricing to remove remnant building structure along site boundaries prior to commencement of soldier pile installation.

As a preliminary guide for costing the depth of surface material requiring continuous support if cut vertical or near vertical has been estimated at the current bore locations and is given in Table 8. These depths are based on the stratigraphy revealed in the bores only and may need to be adjusted for the location depth and backfill quality of any service trenches located close behind excavation boundaries.

Where soldier piles do not extend the full depth of excavation the toes of the piles will require substantial restraint to prevent excessive vertical and or horizontal deflection. A stiff, anchored 'toe beam' has previously been used to successfully restrain soldier pile toes on similar projects.

5.5.2 Hit and Miss Construction

Subject to ground conditions actually encountered it will probably be necessary to adopt a 'hit and miss' near surface excavation approach along most of the site boundaries. n site supervision during excavation may enable the hit and miss approach to be deleted however this cannot be determined in advance of excavation. Hit and miss excavation and shoring may also be required immediately below the founding level of soldier piles.

It is recommended that 'lift heights' for hit and miss panel construction be limited to 1.2m, with subsequent panels not excavated until hit panels are supported. Adoption of an increased panel height would involve HS issues for personnel working in close proximity to the face during wall construction.

When undertaking a 'hit and miss' approach, the stable slopes of any temporarily unsupported face will primarily depend on the composition and strength of buttress materials the orientation of rock discontinuities and the presence of groundwater. Careful assessment and confirmation of individual faces must be made during construction to assess panel design.

5.5.3 Reinforced Panels

Lateral support to near surface materials located on the site boundary could be provided by a reinforced continuous concrete panels that are constructed on a 'hit and miss' basis and supported by prestressed rock anchors.

Subject to the design of the facing it may prove more cost effective to install additional rock anchors dowels to reduce the span of the facing and hence it's required thickness/reinforcement. An optimisation process is suggested between anchor spacing and required facing structural properties.

5.5.4 Rock Dowels

For excavation faces unsurcharged by building loads and or where some movement tolerance exists rock dowels and reinforced shotcrete facing could be adopted for face support. If rock dowel and shotcrete support is to be adopted for permanent works dowels and shotcrete must be fully corrosion protected for at least the life of the structure with dowels being P C sheathed and galvanised. Galvanised dowels are considered necessary for temporary works.

5.5.5 Prestressed Rock Anchors

In 'good' quality rock e.g. medium strength or stronger tuff with limited discontinuities excavation support could be provided by use of a regular grid of prestressed rock anchors. The exact location of anchors would need to be determined by inspection during excavation however an anchor spacing of 3m to 4m is suggested for initial costing. In near surface zones anchor depths and or inclinations will be controlled by the location and depth of existing services and the re uirement to minimise total anchor length.

5.5.6 Shotcrete

In order to stabilise any 'loose' and/or 'low strength' rock zones between rock anchors, reinforced shotcrete is suggested primarily as a personal safety measure.

5.5.7 External Corners

It will be necessary to allow for continuous anchored reinforced concrete support facing over the full excavation depth for any external corners. It is suggested that the facing extend not less than 3m to 5m from each corner along each face with confirmation during construction.

5.5.8 Intermediate Face Support

5.5.8.1 Temporary

In order to stabilise any 'loose' and/or 'low strength' rock zones between rock anchors, shotcrete incorporating mesh welded to anchor plates is suggested primarily as a personal safety measure. This support system could also be adopted to stabilise surface materials with the shotcrete being cantilevered off underlying rock using rock anchors for 'toe' support.

For the stabilisation of local zones of 'unstable rock' between anchors, 100mm thickness of reinforced shotcrete could be adopted for an anchor spacing of not more than 2.5m.

5.5.8.2 Permanent

Permanent support to medium strength or stronger rock located between floor slabs permanent anchors etc. should be designed for a uniform working pressure of not less than given in Table 15 to be confirmed by inspection during construction provided that the rock is not surcharged by adjacent foundation or building loads.

5.6 Temporary Excavation Support Loads

Due allowance must be included in retention support load analysis for groundwater pressure surcharge effects from construction loading the effects of sloping retained materials buildings structures on adjacent sites etc each of which may introduce loads into retaining walls.

5.6.1 Earth Pressures

Where soil fill and or extremely low strength rock is retained near surface temporary wall pressures may be preliminarily calculated using the parameters given in Table 9 where the boundary is not surcharged by adjacent buildings and some movement tolerance exists. A triangular pressure distribution is suggested for a flexible cantilever wall section.

Table 9: Lateral Earth Pressure Coefficient – Near Surface

M	U W $_3$	L E C K	
		C	
Soil Fill	1.8	0.4	0.6
Extremely low strength rock	2.4	0.3	0.5

5.6.2 Boundary Loadings

5.6.2.1 Street Loadings

An average street loading of 16 kPa has been adopted for footpath and road pavement. Detailed consideration must also be given to the location and loading from any proposed 'hard stand', material load out areas that may be located around the excavation perimeter. The analyses reported herein do not include any allowance for load out points as it is currently not known if they will be re uired and if so where they are to be located.

5.6.2.2 Adjacent Building Loadings

Details of adjacent building loads foundation levels along the site boundaries are not currently known and have been assumed for preliminary analysis purposes to be as indicated in Table 10 the loads levels must be confirmed as part of design finalisation so that the relevant estimated support loads given in Table 12 can be revised as necessary.

Table 10: Adjacent Building Loads (Assumed)

F	F S	A F F D	A E F L DL LL	A E B F L DL LL
Adjacent Residential	1m	1m		25
Adjacent Commercial	0m	1m	150	30

To be confirmed preliminary only