It will be necessary to confirm the founding levels and loads for all adjacent building foundations and undertake reanalysis of excavation retention loads as required.

5.6.3 Rock Anchor Excavation Support

5.6.3.1 Design Philosophy

Butler Partner's recommended approach to deep excavation support is to budget for, and to begin, the installation of upper level anchors as if adverse□continuous□planar discontinuities are present and to drop out any unnecessary anchors□if favourable rock structure□jointing becomes obvious as the excavation deepens.

If it is 'optimistically' assumed that continuous, planar discontinuities do not exist at the site, calculated anchor restraint loads will be significantly lower□than if continuous discontinuities are assumed. However□ the scale and continuity of discontinuities in the rock may not become known until the excavation is close to the bottom□in which case the installation of additional anchors would become a major issue □especially if not enough site material remains to build a ramp back to the re□uired anchor locations□□potentially resulting in a significant over□run in excavation program and budget □i.e. the time and cost impost may be greater than the money apparently saved by the proposed reduction in anchor capacity□

5.6.3.2 Analysis

Primary rock anchor loads re□uired to maintain temporary excavation stability have been <u>preliminarily</u> estimated using a 'sliding wedge' analysis for a maximum ('drained') excavation depth of 10m.

The base of the sliding wedge has been considered to comprise a continuous□planar joint filled with some clay □refer Section 5.3.1□ Whilst various alternative rock structure 'failure' modes are considered possible, until site mapping during excavation confirms any alternative modes□it is considered prudent to adopt the scenario of a planar sliding wedge.

The clay coatings on joints encountered in the bores could not be sampled □which is usually the case□and the strength properties used for the stability analysis □given in Table 11□have been estimated based on past experience. It will be necessary to confirm the strength values by laboratory testing□if suitable samples can be recovered during excavation.

Table 11: Rock Discontinuity Strength Parameters

[illegible]	A□□□□□ V□□□
Effective cohesion (c')	5kPa
Effective friction angle (ø')	35 degrees

I[illegible] T[illegible] 11 are 'best estimates', which have [illegible] H[illegible] W[illegible] occurrence was a 'once-off' (to date) it is possible that lower parameters may be encountered in a joint. If such a joint is 'continuous' and adversely oriented with respect to the e[illegible] S[illegible] 5.[illegible]3.3. C[illegible] T[illegible]

5.6.3.3 Stressed Anchor Loads

Estimates of the required temporary rock anchor loads per metre length of excavation face are given in Table 12 for drained conditions, for rock anchor inclinations to the horizontal of 10°, for a factor of safety (F.S.) of 1.5, and no allowance for seismic activity. The anchor loads tabulated are for anchors bonded behind a 45° ('influence') plane rising up from the toe of the deepest point of the excavation face (primary anchors). Anchors that do not bond behind the 45° plane do not contribute to overall wedge stability and must not be included as part of the retention load requirements given in Table 12. Where building foundations are present on adjacent sites, the 'influence' angle may be significantly flatter than 45° and must be assessed on a location by location basis (refer Section 5.13.1).

Table 12: Temporary Rock Excavation Support Loads (10° Anchor Inclination) - Preliminary

F[illegible]	F[illegible] H[illegible]	E[illegible] S[illegible]		R[illegible] W[illegible] L[illegible] N[illegible]	A[illegible] I[illegible]
		S[illegible] L[illegible]	A[illegible] B[illegible] F[illegible] L[illegible]		
Residential Buildings	10	[illegible]	25kPa	750	10
Commercial Building	10	[illegible]	150k[illegible]m[illegible]30kPa	1[illegible]200	10
Street Frontages	10	16	[illegible]	650	10

[illegible]Factor of Safety (F.S.) 1.5 (refer Section 5.6.3.4) [illegible] To be confirmed

It should be noted that the required primary anchor loads given in Table 12 only apply for the nominated anchor inclinations downwards to the horizontal. If the inclination is increased, a substantial increase in required anchor capacity will result.

It should be noted that additional anchor capacity to that given in Table 12, may be required directly below the footings to existing retaining walls located on the site boundary. The magnitude of the additional capacity cannot be determined until full details of the wall footings and loadings are known.

Detailed consideration must be given to the number of anchors employed and the consequences for overall excavation stability should one anchor fail. It is considered essential that anchor numbers and sizing be selected such that if any particular anchor fails, then only a local lack of support results and that an 'unzipping' of the remaining anchors does not occur due to overstressing from load shedding from a failed anchor.

Actual rock anchor spacing will depend on excavation geometry and rock discontinuities and must be finally determined by detailed inspection during excavation.

□nless rock excavation faces are continuously supported□it is considered likely that local 'wedges' of rock will be encountered that will re□uire either removal or stabilisation. It is suggested that single strand prestressed anchors □possibly combined with reinforced shotcrete□would be the most suitable to stabilise wedges with locations being determined on site as re□uired by suitably □ualified personnel. If unstable pieces of rock are too small to anchor or too small to be economically supported by shotcrete□they should be 'barred down' to prevent danger to personnel working below. Safety mesh may be re□uired over faces □where shotcrete is not employed□as a personal protection measure. The need for the mesh would be assessed□based on rock □uality as exposed during excavation.

5.6.3.4 Factor of Safety

To allow for uncertainties in the geological model □particularly□but not only defect strength□□it is considered necessary to apply a F□S of not less than 1.5 to the calculated anchor loads for temporary loading□a higher F□S would be applicable for use for permanent load application.

5.6.4 Rock Dowels

As a preliminary guide to potential rock dowel excavation face support re□uirements□preliminary calculations have been undertaken for a street boundary excavation face supported by □26□corrosion protected dowels□ and the results are given in Table 13. The calculations were based on drained conditions□a F□S of 1.5 for temporary works□and no allowance for loading due to seismic activity. The preliminary dowel details tabulated are for dowels bonded behind a 45□plane.

Table 13: Summary of Preliminary Excavation Support Analysis Results – Rock Dowels

F□□	A□□□ F□ H□□	F□ S□ U□ F A□	R□ S□ FOS 1.5
Street Frontage	10m	16kPa	4 rows of □26 dowels at 1.3m horizontal spacing

□10° Dowel inclination maximum

5.7 Temporary Anchors/Dowels

For the purpose of preliminarily estimating temporary anchor□dowel bond lengths□the maximum working stresses given in Table 14 may be used for sizing. In□situ proof testing of anchors□dowels should be undertaken to confirm design capacity. Bond for 'primary' anchors should not be considered above a plane rising up from the toe of the excavation at 45□to the horizontal. This angle should be confirmed by monitoring as the excavation proceeds.

Table 14: Rock Anchor Working Bond Stress

R□□□ S□□□□□□	W□□□□□□ B□□□ S□□□□□□ M□□□	W□□□□□ B□□□□□□ □□□□□□□□□□□□□□□□
very low	0.1	400 – 500
low	0.2	1□000 – 1□500
medium	0.5	2□000 – 3□000
high □or stronger□	1.0□	4□000 – 5□000

□Subject to grout strength
□Subject to rock discontinuities

Care should be taken with anchor stressing in fill□soils and extremely low to low strength rock to prevent 'outward' movement of temporary walls.

The maximum allowable working bearing pressures given in Table 14 may be used as an initial guide for sizing of bearing plates□the actual size of plates will be influenced by the degree of rock fracturing and rock strength at actual anchor locations and should be confirmed on a plate by plate basis.

5.8 Shotcrete Design Pressures

Working shotcrete face pressure will be dependant on the □uality and strength of retained materials □and horizontal shotcrete span□□which will vary from location to location and can only be accurately determined by inspection of the cut excavation face. However□for initial design□the working pressures given in Table 15 could be adopted for design in the site rock. Pressures for intermediate span lengths can be determined pro□rata.

Table 15: Working Shotcrete Pressure

S□□□□□□□ S□□□ H□□□□□□□□□	M□□□□□□ W□□□□□□ □□□□□□□□□□□□□□
1.2m	15
2.7m	25

5.9 Adjacent Building Underpinning

It will be necessary to confirm prior to commencement of bulk excavation□the founding levels and founding materials of adjacent building foundations so that□if re□uired□underpinning may be designed□undertaken to minimise the risk of excessive foundation movement occurring during basement excavation works.

A 'hit one □miss two' approach may be required for any underpinning of existing foundations, combined with close monitoring for movement. Careful assessment of the stability of each underpin and buttress face must be made during construction.

An underpin panel width of the order of 1.5m may be feasible□with individual panels possibly re□uiring temporary anchoring to maintain stability and to minimise lateral movement. Detailed geotechnical inspection will be re□uired for confirmation at the time of excavation.

5.10 Boundary Movement

Boundary movement will occur as a conse□uence of the basement excavation process□such movements are not possible to prevent and are difficult to accurately predict □accurate predictions are difficult mostly due the significant difficulties in □uantifying the spatial variability in the subsurface conditions and their engineering properties□ As a preliminary guide□and subject to detailed analysis of the retention system proposed□it is generally expected that excavation induced horizontal deflections in areas where fill and soils are retained should be able to limited to something of the order of 10mm to 20mm where an appropriately designed□ anchored soldier pile wall is constructed□depending on the pile section used and the number and capacity of ground anchors installed. In areas where 'good quality' rock is retained by an appropriately designed system of stressed rock anchors□it is generally expected that excavation induced horizontal deflections would be expected to be generally less than approximately 20mm to 30mm □but will re□uire analysis and monitoring for confirmation□

5.11 Approval and Services Checks

It is strongly recommended that approvals □from adjacent property owners□DERM and□or Brisbane City Council□for any anchoring beneath adjoining properties and streets□be obtained □as far in advance of construction as possible□to enable finalisation of excavation support design. Also□underground services that could be potentially affected by anchor installation should be clearly identified well in advance of construction and well before finalisation of excavation support design.

5.12 Groundwater Control

Groundwater control works will be re□uired to be installed to enable economic sizing of temporary and permanent excavation support works and to enable the collection and disposal of groundwater inflow to an 'untanked' basement.

It will be essential to ensure that all services connecting into the site that are to be terminated are properly sealed off □including trench backfill for all services□to prevent them becoming water conduits to the site and causing water pressure surcharging of excavation support works.

5.12.1 Wall Drainage

As part of the calculation of re□uired temporary anchor loads□assumptions as to groundwater level were re□uired to be made. In order to ensure that the actual groundwater level behind each face is controlled □and does not exceed the design assumptions□□it will be necessary to install a regular grid of rock drainage holes to the following preliminary re□uirements□

- 2.5m x 5m □vertical x horizontal□rectangular grid□
- incline upwards to horizontal at 5□□
- installed to 3m minimum behind the 'presumed' 45° 'failure' plane; and
- 50mm minimum diameter.

In addition to the drainage holes□temporary drainage is re□uired behind shotcrete facing□comprising not less than, full height strips of 'Core Drain' (or equivalent) of 20mm minimum thickness, 150mm wide, spaced at not more than approximately 1.8m centres. The drains should be connected into a collection system at the base of the retention system.

5.12.2 Under Slab Drainage

Groundwater seepage will occur through the floor of the excavation that could adversely affect floor slab performance and building amenity□unless a fully tanked basement is adopted. In order to control seepage flows it will be necessary for a 'non□tanked' floor slab to be cast over a free draining layer (incorporating agricultural drains□graded to sumps so that groundwater is removed and pressure does not build□up under the slabs.

Groundwater inflow quantities (and 'design' groundwater level) should be determined by detailed groundwater modelling □incorporating both normal seasonal and extreme weather condition inputs□using computer packages such as M□DFL□W□SEEPW etc.□so that the drainage system can be properly designed. However□for initial costing purposes only □subject to confirmation by detailed analysis and design□a 150mm thick layer of single sized aggregate should be allowed for□together with 50mm diameter geotextile wrapped agricultural drains laid one way at 6m to 8m centres and connected to sumps. Subject to assessment of rock □uality following excavation□it may be necessary to place the drainage layer over a geotextile to prevent clogging due to 'fine' particle ingress.

Groundwater chemistry testing should be undertaken to confirm groundwater □uality□as it will potentially effect discharge options □i.e. stormwater□sewer□on□site treatment etc.) and/or whether a 'closed' collection and disposal system is re□uired.

5.13 Construction Monitoring

Because a number of assumptions have to be made □in advance of construction□to enable geotechnical design for the project □e.g. basement retention and foundation design□□it is essential to implement a detailed construction monitoring programme during inground construction work.

5.13.1 Rock Strength and Structure

It is considered essential that all excavation and support works at the site be conducted under the supervision of an experienced geotechnical engineer□engineering geologist so that ground conditions as exposed can be compared to the ground conditions assumed for excavation support analysis etc.□enabling design modifications □e.g. the addition or deletion of anchors and□or anchor repositioning□to be made□if re□uired.

All foundation excavations should be inspected by a □ualified engineer□geologist prior to casting□to confirm rock quality and final foundation dimensions. 'Cored' inspection bores should be located beneath all 'major' foundations to a depth not less than 2.0 times footing width □or a minimum of 5m□whichever is greater□to ensure that clay seams□'fragmented zones' etc. □e.g. fault breccia encountered below 12m in Bore 5□do not exist within the zone of foundation stress influence□and to enable confirmation□modification of working bearing pressures on a footing by footing basis as re□uired by local rock conditions.

5.13.2 Vibration

□ibration will be caused by excavation □and demolition□work at the site. There is significant debate as to the maximum amount of vibration that buildings □and services□can accommodate□however□vibration restrictions should be set with a realistic appreciation for the normal operational environment of the site and the type of plant□methods to be used for excavation.

Development of a peak particle velocity□fre□uency criteria is considered necessary to enable ade□uate monitoring and control. Whilst vibration assessment was outside the scope of the investigation□the preliminary threshold values given in Table 16 may be suitable as a guide □subject to confirmation□

Table 16: Preliminary Vibration Limits

D□□□□□□□	M□□□ □□ V□□□□□ □□ □ □□□	F□□□□□□ R□□□□ H□□
Buildings with existing defects□having visible cracks in brickwork.	6	10
□ndamaged buildings in technically good condition□apart from minor defects such as cracks in plaster.	10	10 – 50

□nless very heavy and sustained hydraulic rock breaker excavation is undertaken in close proximity to site boundaries□it is considered that excavation induced vibration may not pose a significant threat to adjacent buildings. However□a dilapidation survey of adjacent buildings □and services□and a detailed vibration assessment are strongly recommended prior to commencement of site work and finalisation of excavation methodology□pricing.

5.13.3 Boundary and Adjacent Building Movement

Boundary movements will occur as a conse□uence of the excavation process□such movements are not possible to prevent or to accurately predict. It is considered essential than an accurate survey monitoring program of the excavation boundary and adjacent buildings be put in place for the duration of the excavation works□so that if untoward movement does occur □e.g. due to latent□adverse ground conditions□excavation face support □anchor□loads can be upgraded□modified as re□uired □refer Section 5.6.3.2□

Movement monitoring should be to not less than 1mm □horizontal and vertical□accuracy□so that any movement trends can be readily identified.

5.13.4 Rock Anchor and Dowel Installation

□nly fully experienced□pre□□ualified contractors should be used to install rock anchors □and rock dowels□□ due to the potential to cause damage to off□site properties if incorrect anchoring installation methods are employed□and to ensure that anchors □and dowels□achieve their full design capacity with all necessary load factors.

Detailed anchor□rock dowel construction and stressing□testing records must be kept by the anchoring contractor□which must be continuously available for review and checking. □nder no circumstances should excavation proceed below any row of anchors□until the installation and stressing records are reviewed and accepted by the Superintendent.

5.13.4.1 Rock Anchors

It is considered essential that anchor lift□off tests be conducted on all anchors to confirm that creep is not occurring. □ot less than 1.2m of strand should be left protruding from anchor heads in order to allow for lift off testing and re□stressing □if re□uired□. Protruding strand must be protected from damage.

5.13.4.2 Rock Dowels

A detailed program of proof load testing of specially installed 'test' dowels should be undertaken, prior to the installation of production dowels□in order to confirm design assumptions regarding dowel working capacity etc. The program of testing should also be carried out throughout the process of production dowel installation, especially (but not exclusively) targeted to 'weaker' and/or more 'weathered' zones, encountered during production dowel hole drilling.

□on destructive □proof load□testing□should also be undertaken on all permanent dowels □to confirm dowel bond□□prior to final dowel grouting□which will result in a two stage dowel grouting process□that should be allowed for in construction time estimation.

It will also be necessary to monitor the result of dowel hole drilling (to 'log' soft seams etc.) and to accurately record grout 'takes' to assist with the confirmation of acceptable dowel installation and overall capacity.

5.14 Foundations

During drilling of Bore 5 the base of the tuff was penetrated at a depth of approximately 12m. The underlying strata contained significant 'stiff' clay zones. The occurrence of this 'weak' zone underlying very high strength tuff in Bore 5 and possibly elsewhere underlying the site must be considered in foundation design and pricing.

Use of bored pile foundations for support of major building loads would potentially be required if the base of the excavation is close to the base of the Tuff, as 'soft' seams and strength inversions are more easily dealt with during construction by bored piles, than large pad footings.

The 'strength inversions' (i.e. 'lower' strength rock, underlying 'higher' strength rock) identified in some bores, will require careful consideration in foundation design.

All foundation excavations should be inspected by a qualified engineer/geologist prior to casting, to confirm rock quality. 'Cored' inspection bores should be located beneath all 'major' foundations to a depth not less than 1.5 times minimum footing width to ensure that clay seams, 'fragmented zones', 'lower' strength rock bands etc. do not exist within the zone of foundation stress influence.

5.14.1 Maximum Bearing Pressure

Maximum allowable working bearing pressure values are given in Table 17, together with 'settlement' modulus values for four typical rock strengths anticipated at or close to lowest basement level.

Table 17: Maximum Working Bearing Pressures

Rock Strength	Maximum Allowable Working Bearing Pressure (kPa)*			Settlement Modulus E (MPa)
	Pad Footings⁺	Bored Piles		
		Shaft	Base⁺⁺	
□ow□	1,□□□	12□	1,2□□	2□□ – 1,2□□
Medium□	□,□□□	□□□	□,□□□	□□□ – □,□□□
□igh□	□,□□□	□□□	□,□□□	□□□ – □,□□□
□ery □igh□	□,□□□	1,□□□	1□,□□□	1,□□□ – 15,□□□

□ Maximum allowable working base bearing pressures should be limited to not greater than 25□□k□a if core drilling of footing bases □to confirm the quality of founding strata in the critical zone of stress influence□is not to be carried out

□ Not underlain by 'softer' material

□□ Minimum embedment of four times diameter into founding materials and no underlying 'softer' rock

Due regard must be given in foundation design to the potential for 'strength reversals' and clay seams to be present in the rock below founding level at the site. □n particular, careful consideration must be given in design to the potential to encounter 'weak' zones below footing founding level □e.g. the fault breccia encountered below 12m depth in Bore 5□

□t is considered that local variations in rock strength could be expected to occur over the site and it is suggested that a 'flexible' approach be adopted to the foundation design, construction methodology and costing, so that footing sizes/founding depths can be readily adjusted as required during construction, without cost/time penalties being incurred.

5.14.2 Settlement

Detailed foundation settlement analysis must be undertaken on a foundation by foundation basis, as part of detailed foundation design. Group effects must be considered where 'interaction' of footings can occur.

5.1□.2.1 □ad □ootings

As a broad guide, preliminary estimates of pad footing settlement have been made for a □□,□□□k□ structural load, for two footing sizes/rock strengths, and the results are given in Table 1□ for a single footing.

Table 18: Estimated Isolated Pad Footing Settlement

Footing (m)	Rock⁺ Strength	Maximum Allowable Working Pressure (kPa)	Estimated Settlement (mm)
□2 x □2	medium	□,□□□	5 – 1□
2.2 x 2.2	high	□,□□□	□ – 7

□Uniform with depth below founding level, with no 'softer' zones

The estimated settlements given in Table 1□ are based on the assumption that there are no lower strength materials or clay seams etc. located below founding level and this will need to be confirmed by core drilling at the footing location.

5.1□2.2 Bored □iles

□ingle bored pile settlements for properly constructed piles are typically found to be in the range of □5□ to □7□ of pile diameter at working load, provided that 'softer' seams/bands of rock are not located within the zone of base stress influence.

5.15 Floor Slab Subgrade Properties

□ubgrade properties may vary significantly over the site following excavation and testing will be required at the time of construction in order to confirm design values. □or the purposes of initial costing and preliminary design, the values given in Table 1□ may be adopted.

Table 19: Floor Slab Subgrade Properties

Subgrade □□□e	Strength	□BR (□)	Modulus of Subgrade Reaction (kPa mm)
□ut □ock	very low to low□ medium and stronger	5 – 1□ □□ – □□	□□ – □□ 11□ – 1□□

□Undisturbed and not moisture affected

The weathered tuff rock at the site would be expected to breakdown significantly under the action of excavation equipment and tracked plant. As a result, where very low strength □or weaker□ rock is exposed at subgrade level, it is likely that if the rock is disturbed, recompaction will result in a 'clayey gravel' type subgrade with properties significantly degraded below those of undisturbed rock. □n addition the clayey zones encountered within Bore 5 and underlying the tuff will possess significantly lower subgrade properties than that of the overlying tuff which will need to be considered in design if this material is, or has the potential to be, exposed near or close to the base of the excavation.

5.16 Additional Investigation

□t is recommended that following demolition of the existing buildings in the south eastern corner of the site, additional drilling is undertaken to confirm ground conditions in this area, particularly in view of the 'clayey'/sheared material encountered within Bore 5 between approximately 12m and 15m.

B□□□ER PAR□□ERS P□□ □□□

□eviewed by□

BR□□E B□□□ER
□enior □rincipal

M□□E □E□□□B□□R
□rincipal

Important Information about Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

geotechnical • geo-environmental • groundwater

BUTLER PARTNERS

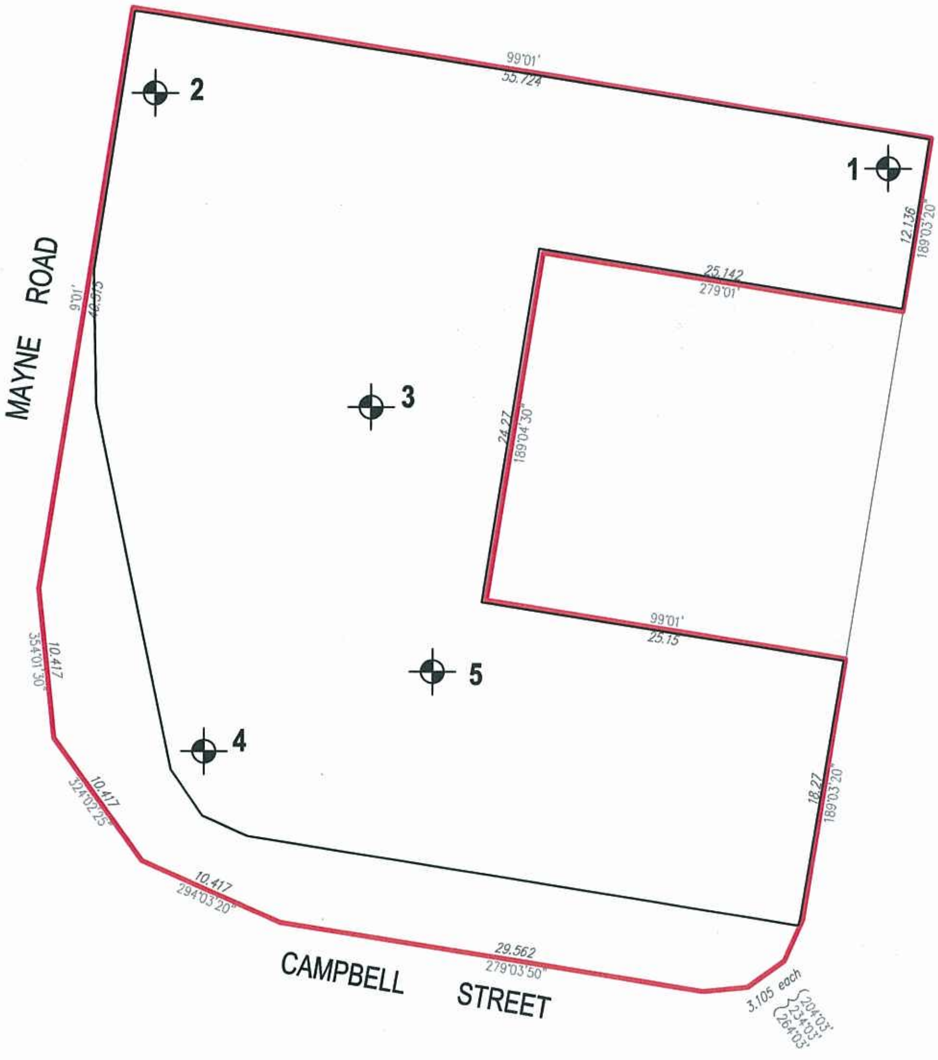
Butler Partners Pty Ltd
ABN 14 816 574 868
79 Doggett Street
Newstead, Queensland 4006
P.O. Box 2267
Fortitude Valley BC
Queensland 4006

www.butlerpartners.com.au

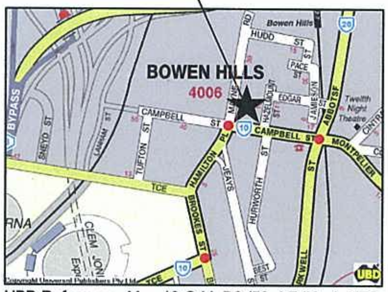
Ph: 61 7 3852 3800
Fx: 61 7 3852 3808
enquiries@butlerpartners.com.au

CONSULT AUSTRALIA Member Firm

Certified System Quality ISO 9001 SAI GLOBAL



Site



UBD Reference: Map 19 Grid D2 (53rd Edition) NTS

HIGH RISE RESIDENTIAL DEVELOPMENT

29-35 CAMPBELL STREET, BOWEN HILLS

LOCALITY PLAN AND TEST LOCATIONS

CLIENT: METRO PROPERTY DEVELOPMENT PTY LTD

LEGEND

- 1 Bore
- Site Boundary
- Basement Outline

SCALE AT A4:

0 5 10 15 20m

1:500

DATE:	FEBRUARY 2011
DRAWN:	RL
APPROVED:	
PROJECT No:	010-203A
DRAWING No: 1	REV: A

Reference: RPS Australia East Pty Ltd's 'Site Dimensions', Drawing No. 103356, undated and ML Designs' 'Option 12 Basement 1 & 2', Drawing Nos. METR0004_SK058-F and SK059-F, dated 7/10/10.

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

APPENDIX A

BORE REPORT SHEETS WITH EXPLANATORY NOTES

BORE REPORT

BUTLER PARTNERS

BORE 1

Client: Metro Property Development Pty Ltd

Project: High Rise Residential Development

Location: 29-35 Campbell Street, Bowen Hills

Project No: 010-203A

Page No: 1 of 2

Date: 30 September 2010

Ground Surface Level: RL19.8m*

Description

Depth (m)	Description
0.0	***CONCRETE SLAB***
	FILL - dark brown, silty clay, trace fine to medium grained sand, trace fine to medium subrounded to subangular gravel
	TUFF (XW) - extremely low strength, yellow, pale grey-white mottled orange
	TUFF (MW) - low to medium strength, pale grey-white mottled orange
	TUFF (SW) - high to very high strength, pale purple and orange
	- pale red-pink mottled pale orange
	- very high strength

RL (m): 19.8 (0 m), 19.0, 18.0, 17.0, 16.0, 15.0, 14.0, 13.0, 12.0, 11.0, 10.0

Structures

- 0.95m,defects typically J,0-45,iv-vii, Cl-Fe-XW,1-5mm
- 2.5m,J,45,viii,Fe,1-2mm
- 3.67m,J,0,vii,XW,2-7mm
- 4.87m,J,0-20,iv,Ca,1-8mm
- 5.05m,J,20,vi,Ca,1-3mm
- 5.2m,J,15,vi,Ca,1-4mm
- 9.16m,J,65,vii,Fe,1mm
- 10.0m,J,70,vii,Fe,3-4mm

Samples and Coring

Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	D/E	0.1					
	D	0.25					
	S/E	0.5					15 for 120mm HB
		0.62					
15	C	0.95 – 2.45 (1.4, 1.42)	4.3(d), 3.9(a)	100	50		
5	C	2.45 – 3.95 (2.9, 2.95; 3.6, 3.65)	2.9(d), 4.3(a); 3.7(d), 2.7(a)	100	71		
5	C	3.95 – 5.45 (4.9, 4.95)	2.0(d), 1.5(a)	100	77		
2	C	5.45 – 6.95 (5.95, 5.97)	4.7(d), 5.0(a)	100	100		
1	C	6.95 – 8.45 (7.8, 7.85)	5.7(d), 5.6(a)	100	90		
5	C	8.45 – 9.95 (8.55, 8.6)	3.6(d), 3.3(a)	100	70		
		9.95 (9.95, 9.97)	3.6(d), 5.5(a)				

Groundwater Monitoring Welll Details: Backfill; Casing

D	Disturbed Sample	E	Environmental Sample	C	NMLC Coring
B	Bulk Sample	S	Standard Penetrometer Test (SPT)	Is(50)	Point Load Test Result (MPa)
U	Undisturbed Tube (50mm diameter)	HB	SPT Hammer Bouncing	(d)	Diametral Point Load Strength Test
pp	Pocket Penetrometer Test (kPa)	( )	No Sample Recovery	(a)	Axial Point Load Strength Test

Rig: Jacro 350

Drilling Method: Auger to 0.75m, casing to 0.8m, wash bore to 0.95m, then NMLC core to 15.0m

Groundwater: No free groundwater encountered during auger drilling

Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010

BORE REPORT

BUTLER PARTNERS

BORE 1

Client: Metro Property Development Pty Ltd
Project: High Rise Residential Development
Location: 29-35 Campbell Street, Bowen Hills
Project No: 010-203A

Page No: 2 of 2
Date: 30 September 2010
Ground Surface Level: RL19.8m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
	TUFF (SW) - very high strength, pale red-pink with pale mottled orange	9.0			5	C	10.8 10.85 10.95	3.1(d) 4.2(a)	100	68			Bentonite
11							11.03	2.7(a)					Sand
		8.0		11.5m,J,60,vii,Fe,4mm 11.89m,J,0,vii,XW-Fe,2-5mm	4	C	11.9	2.8(d)	100	57			
12	- high strength			12.53m,J,45,vii,Fe,4mm			12.69	2.1(d) 2.1(a)					Slotted
13		7.0		13.0m,J,45,vii,Fe,2-3mm									
	- very high strength	6.0			2	C	13.3	4.5(d) 4.5(a)	100	80			
14							14.15 14.28	3.3(d) 3.5(a)					
		5.0			1	C			100	100			
15	End of Bore at 15 m						15.0						
16		4.0											
17		3.0											
18		2.0											
19		1.0											
20		0.0											

D Disturbed Sample
B Bulk Sample
U Undisturbed Tube (50mm diameter)
pp Pocket Penetrometer Test (kPa)
E Environmental Sample
S Standard Penetrometer Test (SPT)
HB SPT Hammer Bouncing
( ) No Sample Recovery
C NMLC Coring
Is(50) Point Load Test Result (MPa)
(d) Diametral Point Load Strength Test
(a) Axial Point Load Strength Test

Rig: Jacro 350
Drilling Method: Auger to 0.75m, casing to 0.8m, wash bore to 0.95m, then NMLC core to 15.0m
Groundwater: No free groundwater encountered during auger drilling
Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010

BORE REPORT

BUTLER PARTNERS

Client: Metro Property Development Pty Ltd

Project: High Rise Residential Development

Location: 29-35 Campbell Street, Bowen Hills

Project No: 010-203A

BORE 2

Date: 6 October 2010

Ground Surface Level: RL18.9m

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
0 – 1.3	***FILL*** - brown-black, sandy clayey gravel, fine to medium grained gravel, fine to medium grained sand	18.9, 18.0				S/E	0.5, 0.95					1,1,5 N=6	
1.3 – 2.0	***SILTY CLAY (CI/CH)*** - stiff, pale orange-grey, trace fine grained, subangular gravel	17.0		1.65m,Auger Refusal 1.65m to 2.91m,B,0-10,iv,Cl,<1mm with J,45-90, iv,cln,Fe, <1mm	>10	C	1.65, 1.75, 2.1, 2.65	0.5(d), 1.9(a)	100	0			
2.0 – 2.9	***TUFF (MW)*** - medium to high strength, pale grey-red and pale orange				>10		2.91			0			
2.9 – 3.9	***SILTY CLAY (CI)*** - stiff grading very stiff, pale grey and pale orange, with fine subangular gravel and zones of tuff	16.0, 15.0			-	C	3.93		100	-		pp=450, pp=150	
3.9 – 4.9	***TUFF (XW)*** - extremely low strength, pale grey mixed with clayey zones	14.0			>10	C			100	0			
4.9 – 5.4	***TUFF (MW)*** - high strength, pale red and pale orange			4.9m to 5.07m,J,75-90,iv,Fe, healed 5.05m to 5.1m,J,45,iv,cln,<1mm	1-2		4.9, 4.95, 5.37	1.2(i)		100			
5.4 – 6.9	***TUFF (SW)*** - very high strength, pale orange and pale purple	13.0, 12.0		6.0m,B,0-10,iv,Fe,<1mm	1	C	5.9, 5.92	3.0(d), 4.4(a)	100	100			
6.9 – 8.4		11.0		7.01m,B,0-10,iv,Fe,<2mm 7.19m to 7.31m,J,45,iv,Fe,<1mm 7.74m to 7.79m,J,45,iv,Fe, healed 7.75m to 7.8m,J,45,iv,Fe,<1mm 7.95m,B,0,iv,Fe,<1mm 8.01m,B,0,iv,Fe,<1mm	2-3	C	6.91, 6.92, 6.97, 7.9	4.0(d), 4.6(a), 3.0(d), 4.2(a)	100	100			
8.4 – 9.8		10.0, 9.0		8.58m to 8.59m,J,35,iv,Fe,<1mm 9.02m,B,0,iv,Fe,<1mm 9.08m to 9.15m,J,45-80,iv,Fe,<1mm - healed 9.17m,B,0-10,iv,Fe,<1mm 9.25m to 9.41m,J,45-90,iv,Fe,<1mm - healed 9.58m to 9.77m,J,25-90,iv,Fe,<1mm - healed	4-5	C	8.42, 8.5, 9.5, 9.52, 9.84	5.3(d), 5.6(a), 3.0(d), 4.1(a)	100	71			
9.8 – 10				9.94m to 10.02m,J,0-45,iv,Fe,healed									

D	**Disturbed Sample**	**E**	**Environmental Sample**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**HB**	**SPT Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**

Rig: Jacro 350

Drilling Method: Auger to 1.65m, casing to 2.0m, wash bore to 1.65m, then NMLC core to 15.0m

Groundwater: No free groundwater encountered during auger drilling

Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010

BORE REPORT

BUTLER PARTNERS

Client: Metro Property Development Pty Ltd
Project: High Rise Residential Development
Location: 29-35 Campbell Street, Bowen Hills
Project No: 010-203A

BORE 2

Date: 6 October 2010
Ground Surface Level: RL18.9m

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
10–11.21	**TUFF (SW)** - very high strength, pale orange and pale purple	8.0		10.12m to 10.14m,J,45,iv,Fe,<1mm 10.47m to 11.11m,J, 50-90,iv, Fe,<1mm 10.57m to 10.61m,J,45,iv,Fe,healed 10.66m,B,0,iv,Fe,<1mm 10.82m to 10.9m,J,10-90, iv, Fe, healed 10.91m to 11.08m,J,45-90,i v, Fe, healed	3-4	C	11.1 11.21	3.4(d)	100	41			
11.21–12.49		7.0		11.17m to 11.24m,J,45,iv,Fe,healed 11.51m,B,0,iv,Fe,<1mm 11.52m,B,0,iv,Fe,<1mm 11.87m,B,0,iv,Fe,<1mm 11.88m,B,0,iv,Fe,<1mm	2	C	11.9 11.92 12.49	4.1(d) 5.8(a)	100	98			
12.49–13.68		6.0		13.13m to 13.16m,J,45,iv,Fe,<1mm 13.175m to 13.2m,J,35,iv,Fe,<1mm 13.395m to 13.45m,J,45,iv,Fe,<1mm	2	C	13.1 13.13 13.68	4.6(d) 6.0(a)	100	96			
13.68–14.71		5.0		14.3m to 14.34m,J,45,iv,Fe,<1mm 14.69m to 14.73m,J,45,iv,Fe,<1mm	2	C	14.25 14.28 14.71	5.3(d) 5.9(a)	100	100			
14.71–15.0		4.0			2	C	15.0		100	100			
15	End of Bore at 15 m												
16		3.0											
17		2.0											
18		1.0											
19		0.0											
20		-1.0											

D	Disturbed Sample	E	Environmental Sample	C	NMLC Coring
B	Bulk Sample	S	Standard Penetrometer Test (SPT)	Is(50)	Point Load Test Result (MPa)
U	Undisturbed Tube (50mm diameter)	HB	SPT Hammer Bouncing	(d)	Diametral Point Load Strength Test
pp	Pocket Penetrometer Test (kPa)	( )	No Sample Recovery	(a)	Axial Point Load Strength Test

Rig: Jacro 350

Drilling Method: Auger to 1.65m, casing to 2.0m, wash bore to 1.65m, then NMLC core to 15.0m

Groundwater: No free groundwater encountered during auger drilling

Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010

BORE REPORT

BUTLER PARTNERS

Client: Metro Property Development Pty Ltd
Project: High Rise Residential Development
Location: 29-35 Campbell Street, Bowen Hills
Project No: 010-203A

BORE 3

Page No: 1 of 2
Date: 29 September 2010
Ground Surface Level: RL18.2m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
0	***CONCRETE SLAB***	18.2				D/E	0.0 0.25						
	FILL - grey-brown orange, clayey sandy gravel, fine to coarse subangular to subrounded gravel, fine to medium grained sand, strong petrol smell					D/E S	0.5 0.95					6,8,18 N=26	Backfill
1	- light grey	17.0					1.4						
	TUFF (XW/HW) - very low strength, light grey			1.44m,crush zone,30mm 1.5m,crush zone,20mm 1.54m,clay zone,20mm 1.8m to 2.25m,Core Loss	10	C	1.7		100	0			Casing
2	***TUFF (HW)*** - low strength, pale white-grey and mottled orange, very low strength zones	16.0			>15	C			20	0			
				2.29m,B,0,iv,Cl,1mm defects typically B-J, 0-15,iv, vii,Cl, clean,Fe,1mm 2.52m to 3.83m,Core Loss	7	C	2.25 2.5 2.52	0.1(i)	100	0			
3		15.0			-	C			0	0			
4	***TUFF (MW)*** - low strength, mottled orange and pale white-grey, with bands of (XW) and high strength bands	14.0		3.92m,J,45,viii,Cl,1-2mm 4.07m,J,45,viii,Cl,1-2mm 4.24m,J,45,viii,Cl,1-2mm 4.5m to 4.9m,Fe veins 4.83m,J,45,vi,Cl,1mm 4.9m,crush zone,15mm	3	C	3.83 3.93 4.35	0.1(i) 0.2(d) 0.2(a)	100	71			
5		13.0		5.29m,J,45,iv,Cl,1mm	2	C	5.07 5.29		100	0			
				5.58m,crush zone,20mm	>15	C			63	0			
					1	C	5.7 5.89 5.85	0.1(d) 0.2(a)	100	0			
6	- medium to high strength	12.0		6.07m,J,0,iv,Cl,4mm 6.17m,J,45,iv,Cl,5mm 6.19m,J,45,iv,Cl,4mm 6.97m,J,20,vii,Cl,1mm	16	C	6.37 7.28 7.35	0.4(d) 0.8(a) 0.7(d) 1.2(a)	100	60			
7		11.0											
8		10.0		7.77m,J,45,vii,Cl,1mm	12	C	8.2 8.22	0.7(d) 0.7(a)	100	28			
				8.4m,crush zone,100mm 8.5m to 8.7m,J,40,vi,Cl,1mm 9.0m to 9.12m,Core Loss	5	C			71	37			
9		9.0					9.12 9.47	2.1(d) 3.4(a)					
10				10.0m,B,45,viii,Fe,6mm	3	C			100	84			

D	Disturbed Sample	E	Environmental Sample	C	NMLC Coring
B	Bulk Sample	S	Standard Penetrometer Test (SPT)	Is(50)	Point Load Test Result (MPa)
U	Undisturbed Tube (50mm diameter)	HB	SPT Hammer Bouncing	(d)	Diametral Point Load Strength Test
pp	Pocket Penetrometer Test (kPa)	( )	No Sample Recovery	(a)	Axial Point Load Strength Test

Rig: Jacro 350

Drilling Method: Auger to 1.0m, casing to 1.2m, wash bore to 1.4m, then NMLC core to 15.0m

Groundwater: Free groundwater encountered at 0.7m depth during auger drilling

Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010

BORE REPORT

BUTLER PARTNERS

Client: Metro Property Development Pty Ltd
Project: High Rise Residential Development
Location: 29-35 Campbell Street, Bowen Hills
Project No: 010-203A

BORE 3

Date: 29 September 2010
Ground Surface Level: RL18.2m*

Description	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
TUFF (MW) - high strength				10.38 10.67	0.5(d) 2.0(a)					Bentonite
		8	C	11.25 11.27	1.0(d) 0.7(a)	100	0			Sand
	11.6m,crush zone,5mm 12.14m,J,45,vii,Cl-XW,2mm 12.34m,J,45,vii,Cl-XW,3mm	12	C			100	22			
	12.51m,J,0,iv,Cl,4mm	0	C	12.37 12.46 12.52	0.8(d) 1.5(a)	100	0			Slotted
	12.86m,J,45,viii,Cl,2mm 12.92m,crush zone (XW),20mm 13.33m,J,45,vii,Cl,3mm	8	C	13.35		100	37			
		1	C	13.6 13.63	1.1(d) 1.1(a)	100	0			
		2	C	14.25		100	80			
- band of (DW) rock with clay	14.4m,clay zone,40mm 14.52m,crush and clay zone,40mm 14.68m,crush zone	10	C	14.86 15.0	0.8(d) 0.8(a)	100	30			
End of Bore at 15 m										

Depth (m): 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

RL (m): 7.0, 6.0, 5.0, 4.0, 3.0, 2.0, 1.0, 0.0, -1.0

Lithology

D	Disturbed Sample	E	Environmental Sample
B	Bulk Sample	S	Standard Penetrometer Test (SPT)
U	Undisturbed Tube (50mm diameter)	HB	SPT Hammer Bouncing
pp	Pocket Penetrometer Test (kPa)	( )	No Sample Recovery

C NMLC Coring
Is(50) Point Load Test Result (MPa)
(d) Diametral Point Load Strength Test
(a) Axial Point Load Strength Test

Rig: Jacro 350

Drilling Method: Auger to 1.0m, casing to 1.2m, wash bore to 1.4m, then NMLC core to 15.0m

Groundwater: Free groundwater encountered at 0.7m depth during auger drilling

Remarks: *Approximate ground surface level interpolated from RPS Australia East Pty Ltd's, 'Detail Survey' Dwg. No. 103356-1, dated 30 July 2010