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Excavations In The Brisbane CBD

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Douglas.

Significant geological non-conformities (ie. shear zones or contact zones) exist within the geological formations that underlie the Brisbane CBD. Where deep excavation is proposed geotechnical modeling of these excavations must be undertaken to fully understand the possible slope failure mechanisms and to design appropriate positive support requirements. Site specific geotechnical models must be verified by inspection and monitoring of the respective excavations.

This paper highlights the geological and geotechnical engineering aspects of some recently completed deep excavations in the Brisbane CBD.

The projects discussed include the Brisbane Casino Carpark which comprised a 22m deep by 80m square excavation where full-time slope stability assessment and inclinometer monitoring were undertaken. The excavation was adjacent to heritage-listed buildings and the project won an ACEA 'Award of Excellence' for Douglas Partners Pty Ltd.

1.0 INTRODUCTION

Deep excavations (ie. deeper than 6m) in soil and rock generally require battering or where space does not permit, some form of temporary support during construction. Some excavations require permanent support if the structure is not designed to support the excavation earth pressures.

Douglas Partners Pty Ltd (DP) Brisbane office have designed excavations on a number of significant projects within the Brisbane CBD in the past seven years. The work included design of support measures and monitoring of the excavation during construction.

Significant shear and contact zones were encountered within these excavations which required detailed stability assessment and ongoing monitoring to ensure that significant failures did not occur. This paper presents an overview of two of these projects namely the Brisbane Casino Carpark and the Central Energy Building at the Royal Brisbane Hospital .

2.0 BRISBANE CASINO CARPARK

2.1 Project and Geotechnical Investigation

The Brisbane Casino underground carpark excavation was 22m deep by 80m square. The site is bounded between George Street, Elizabeth Street, William Street and the Lands Administration Building and is located one city block east of the Brisbane River in the Brisbane CBD.

Field investigations comprised the following:-

- drilling and sampling of nine (9) test bores
- excavation and sampling of five (5) test pits
- pressuremeter testing
- water pressure tests
- rock stress measurements in one bore
- seismic traverses
- mapping of rock exposures

A site plan showing the location of the excavation and the test locations is given in Figure 1.

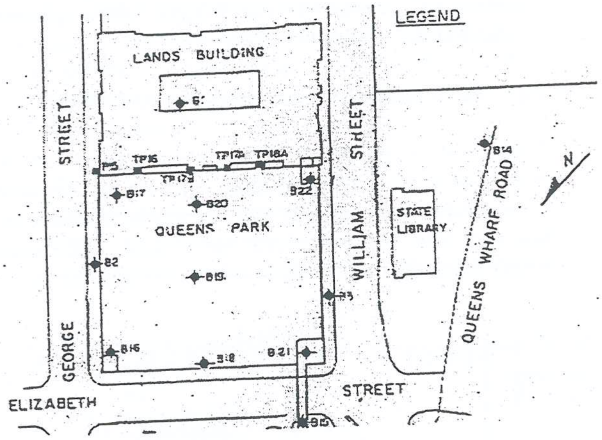

Figure 1 Site Plan

2.2 Subsurface Profile and Regional Geology

The generalised subsurface profile on this site comprised a mantle of variable filling materials and natural very stiff to hard silty clays typically 0m to 5m thick. These soils were overlying bedrock.

Bedrock comprised phyllite, argillite (cleaved shale), and greywacke of the Palaeozoic Age Neranleigh-Fernvale Group which is in close contact with the Pre-Cambrian Bunya Phyllite Group which comprises phyllite, chert and greywacke. Documented information and the results of previous work in the CBD indicated that the "Normanby Fault" was situated close to the site.

The sediments from which the rocks were derived were originally deposited in a deep sea marine environment and have been regionally metamorphosed, folded, tilted, and uplifted. During the deformation and folding processes, the various interbedded rock types have been moved differentially to produce zones of shearing which appear mostly along contacts between essentially 'competent' sandy materials (greywackes) and 'non-competent' muddy materials (argillites, and phyllites).

The original bedding is generally preserved as a strong, parallel and/or slatey cleavage, which can be either straight or tightly folded (crenulated). These rocks are commonly interbedded, steeply (40° to 60°) dipping, and well jointed.

Most joints form as a result of tensile stresses imposed within the rock mass. Such stresses may develop during lithification of strata or during subsequent phases of deformation and folding as the rocks become lithified. Joints often occur along the crests of folds and/or other areas of stress relaxation. Faults, shears and other related geologic structures develop as a result of compressive stresses in the rock. Complex structural patterns may exist in rock strata which have been subjected to multiple phase of tectonic deformation and metamorphism. On this site, jointed rock mass strength and the superimposed surficial weathering processes controlled the geotechnical characteristics of the rock mass.

2.3 Rock Slope Stability and Support Design

The dominant joint and cleavage directions were derived from measurement of rock core samples and site mapping of outcrops near the site and are given in Table 1. All bearings and dip directions are staked with respect to AMG North (True North).

Table 1

Joint Set	**Dip (°)**	**Dip Direction (°)**
1	50-80	235-245
2a	80-88	115-145
2b	70-80	295-325
3	45-65	170-180
4	35-45	270-285
Cleavage	**Dip (°)**	**Dip Direction (°)**
C1	30-50	060
C2	55-65	075

The potential failure modes predicted prior to excavation for each face are detailed below:

William Street (Trend 131^0)

Planar failures along cleavage planes which dip out of this face at 40-65^0 to the north-east and wedge failures caused by the intersection of Joint Set 4 and the cleavage plunge out of this face at 40-45^0. Shears zones that parallel the cleavage, also cut this face and were possible causes of planar failures.

Elizabeth Street (Trend 049^0)

Planar failures along Joint Set 2 which dips out of this face at 30-55^0 and which are truncated by the cleavage were considered to be the most likely mode of failure on this face. Wedge failures resulting from the intersection of Joint Sets 2 and 4, which plunge out of this face at 40-45^0. Wedges caused by the intersection of Joint Set 2 and cleavage, which plunges out of this face at approximately 40-65^0.

George Street (Trend 131°)

Wedge failures caused by the intersection of either Joint Sets 1 and 2 which plunge to the south-west at 50-70^0 or Joint Sets 2 and 3 which plunge to the south-west at 55-65^0 and are truncated by the cleavage.

Lands Administration Building (Trend 049°)

Wedge failures caused by the intersection of Joint Sets 1 and 3 which plunge at about 50^0 to the south-west and are truncated by the cleavage.

Geotechnical modelling of the proposed excavation was undertaken. The rock anchor loads required to maintain excavation stability (and hence basement wall loads) were estimated using a 'sliding wedge' analysis for a range of unsurcharged, 'drained', excavation depths to 23m.

The base of the sliding wedge analysis was considered to comprise a continuous, planar joint filled with clay. This was considered to be an upper bound value as it was considered that continuous clay filled joints existed only in the upper portions of deep excavations.

The clay coatings on joints encountered in the bores could not be sampled and the properties used in the stability analysis were estimated based on correlation with Atterberg limit results. The properties used are given in Table 2.

Table 2

Rock Joint Infill Property	**Excavation Face**	
	George & Elizabeth Sts, Land Administration Building	**William Street**
Effective cohesion (c') kPa	5	2
Effective friction angle (Ø') degrees	28	22

The maximum estimated anchor loads per metre length of excavation face, calculated for a factor of safety (FOS) of 1.0, are given in Table 3.

Table 3

Excav -ation Depth (m)	Anchor Inclination (°)	Required Anchor Load (kN/m of face)	
		George & Elizabeth Sts, Lands Administration Building Faces	William Street Face
6	10	190	N/C
8	10	360	N/C
1	10	560	N/C
12	10	800	1050
17	10	1900	2300
23	10	4000	4500

N/C Not calculated

Support for the straight sided excavation faces was then designed. The top 2m to 4m depth comprising surface filling, soils, and extremely low strength rock which required continuous support to prevent instability, was supported by anchored, cantilever soldier piles socketed into rock, with timber lagging.

The ground below the soldier pile and lagging comprised competent rock (ie slightly fractured, medium strength phyllite, argillite or greywacke) and was supported by a regular grid of prestressed rock anchors. An anchor spacing of 2m to 3m horizontally was suggested for initial design.

Loose rock zones between the anchors were to be supported by shotcreted and meshed panels.

A diagrammatic representation of the general support details for the excavation faces is given in Figure 2.

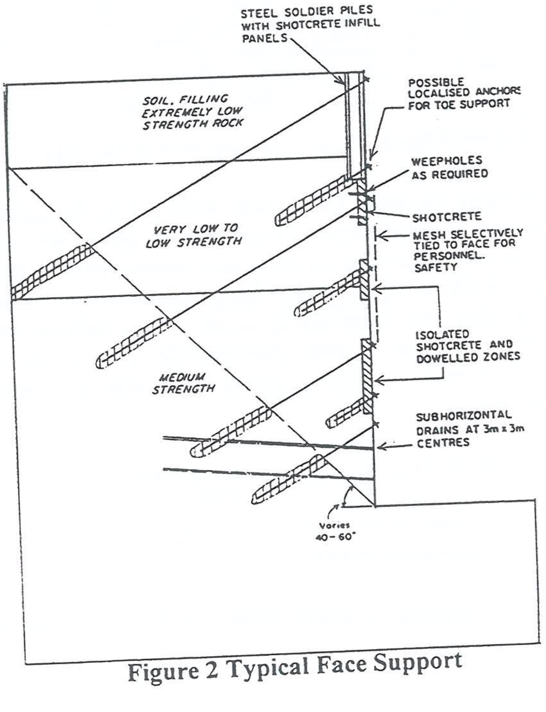

Figure 2 Typical Face Support

Estimated settlement of the ground behind the crest of the excavation due to relaxation of the upper soils and filling materials for a 3m deep vertical cantilevered retaining wall are given in Table 4.

Table 4

Distance from wall (m)	Estimated Settlement (mm)
1	7-9
6	4-5
12	3-4
18	1-2
24	0

The insitu rock stress measurements undertaken in Bore 20 indicated that to the depth tested (ie 14m) no significant stress exists in the rock in excess of geostatic stresses. It was predicted that significant lateral movement of the excavation faces would not occur due to release of 'locked in' tectonic stresses.

3.0 CONSTRUCTION MONITORING

DP monitored the excavation as the work proceeded. This involved progressive mapping of the excavation faces, in particular the locations of support measures, and monitoring lateral movement via inclinometers. Surveying of the condition of the adjacent heritage listed buildings was part of the program. Monitoring results were compared with original predictions - failure mechanisms and excavation face displacements.

About 440 geological discontinuities were mapped during the course of the works. Mapping data were assessed using the 'DIPS' computer software. Figure 3 shows the resulting lower hemisphere stereographic projection contoured plot.

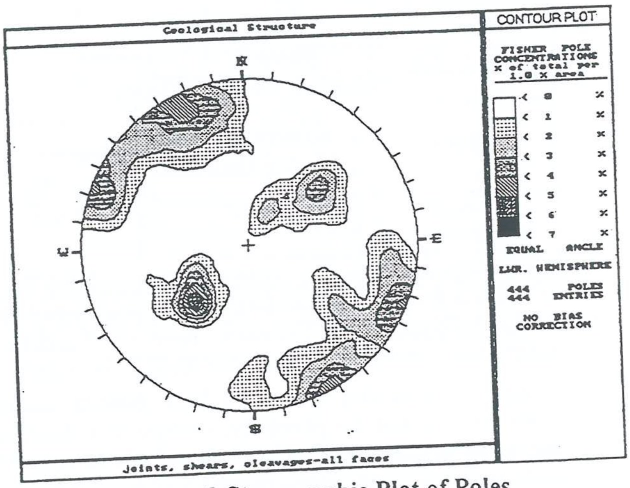

Figure 3 Sterographic Plot of Poles

Two persistent cleavage directions C1 and C2 were identified, one superimposed on the other (i.e. both sets are present within the strata). These are summarised in Table 5.

Table 5

Cleavage Orientations		
Cleavage	Dip	Dip Direction
C1	35-50°	035-060°
C2	55-65°	055-070°

The face mapping also revealed that sheared zones were generally parallel to the cleavage, but often dipped more steeply. Conjugate shear zones were also noted on the George Street face and dip at 20^0 to 45^0 opposite to the cleavage.

Mapped cleavage orientations compared well with the predicted ones.

Five joint sets were interpreted, including two steeply dipping, conjugate sets (1a and 1b) trending at 20^0 to 30^0 and 55^0 to 65^0. These steep joints strike approximately parallel and up to 15° on either side of the cleavage dip direction. Set J2 almost parallels the cleavage but dips 90° in the opposite direction. Joint sets 3 and 4 are minor sets. Table 6 lists the joint sets.

Table 6

Joint Set Orientations			
Joint Set	Measured		Corresponding Predicted Joint Set
	Dip	Dip Direction	
1a	80-90	325-335 145-155	Joint Set 2b
1b	80-90	110-120 290-300	Joint Set 2a
2	15-25	210-220	Joint Set 3
3	50-60	230-240	Joint Set 1
4	60-70	302-312	Joint Set 2b

Potential wedge failures identified by mapping are summarised in Table 7

Table 7

Potential Wedge Failures		
Face	Excavated Wall Facing Direction	Joint Combinations (°M)
Lands Administration	319°	2 and 1
Elizabeth Street	139°	C2 and 1
William Street	041°	C2 and 1a
George Street	221°	3 and 1a

Samples of silty clay recovered from sheared zones and joints were tested for plasticity, particle size distribution and shear strength to check the initial predictions. This testing indicated an effective friction angle (Ø') in the range of 23^0 to 36^0 and an effective cohesion c' of 4kPa and 22kPa. A comparison of the initial failure plane joint properties used in the design and those used on the basis of the laboratory testing are presented in Table 8

Table 8

Rock Joint Infill Property	Predicted Design		Measured
Effective cohesion (c') kPa	5	2	4
Effective friction angle (Ø') degrees	28	22	26

Monitoring of the excavation faces, including mapping and laboratory testing of joint infill materials, verified the initial geotechnical assumptions. The dominant failure mechanisms identified during excavation were planar wedges.

The distribution of the principal rock types ie argillite, phyllite, and greywacke, is shown on Figure 4. A number of shear zones are also shown.

Zones identified in the investigation were mapped as continuous structures across the excavation. These sheared zones were generally less than 200mm wide and comprised firm clays grading to extremely low strength rock as infill materials. Some clay seams within the crush and shear zones extended below full excavation depth.

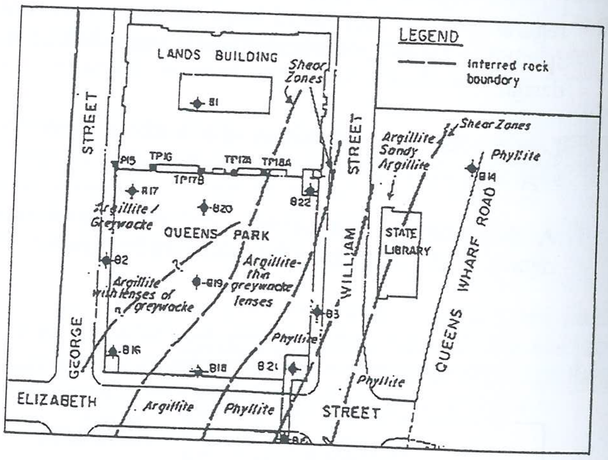

Figure 4 Rock Types Encountered in the Excavation

Generally the excavation faces were stable throughout the duration of the project, however a minor instability occurred in the corner bounded by the Lands Administration Building and William Street. A 6mm displacement into the excavation was noted along a rock wedge defined by Joint Set 4, Joint Set 1a and Cleavage C2. Part of the wedge failed during excavation and the resulting overhang was removed.

The contributing factors to this failure were:

- water from services entering the face along joints and shear zones.
- omission to install anchors in this area of the face.

The encountered sheared zones tended to deteriorate quickly once exposed. These materials slake, potentially undermining the slope if not locally stabilised. Rock bolts which fasten shotcrete and mesh panels were used to seal these sub-vertical clay filled shear zones which were relatively continuous (i.e. 10m to 20m long) in the Elizabeth Street, George Street corner.

The results of inclinometer monitoring along the Lands Administration Building face indicated maximum movement of 8mm into the excavation.

4.0 ROYAL BRISBANE HOSPITAL

4.1 Project and Geotechnical Investigation

The Royal Brisbane Hospital is located in the suburb of Herston, approximately 5kms north-west of the Brisbane CBD. The construction of the Central Energy Building at this hospital required the enlargement, benching, and stabilisation of an existing three sided, 13m high excavation (originally a quarry).

A low cost geotechnical investigation was undertaken comprising:

- drilling and sampling of five (5) shallow test bores
- (ie to maximum depth of 5m)
- mapping of rock exposures

The layout of the proposed excavation is shown on Figure 5

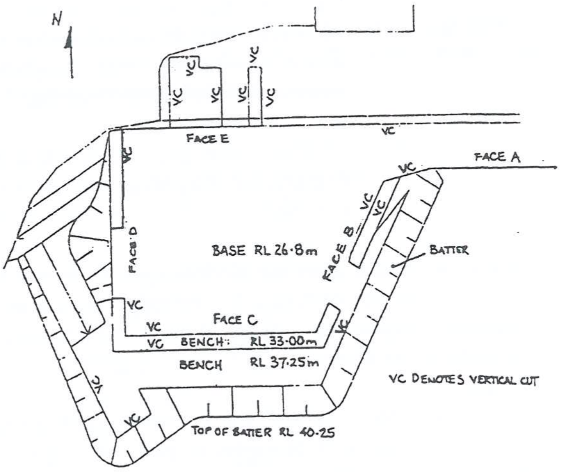

Figure 5 Site Plan

4.2 Subsurface Profile and Regional Geology

The generalised subsurface profile around the top slopes of the quarry comprised a mantle of variable filling materials and natural very stiff to hard silty clays typically 0m to 2m thick. These soils were underlain by bedrock, comprising rhyolitic tuff and ignimbrite of the Triassic Age Brisbane Tuff Formation.

The Brisbane Tuff generally overlies phyllite, argillite (cleaved shale), and greywacke of the Palaeozoic Age Neranleigh-Fernvale Beds in the Brisbane inner city area. The zone between these two rock formations is known as the contact zone. This typically comprises silty clays or weathered rock and varies in thickness. The tuff immediately above the contact zone is often highly brecciated.

The Brisbane Tuff originated as ash ignimbrite ejected by volcanic explosions. The ash flowed down gullies and infilled low lying areas of the original landscape. The rapidly cooling ash typically formed a high strength massive rock mass.

Preferential weathering of the weaker underlying strata has resulted in the development of some tuff cliffs within inner areas of Brisbane. The tuff formation is typically characterised as high strength, massive to jointed. On this site jointing of this rock mass controlled the geotechnical characteristics.

4.3 Rock Slope Stability and Support Design

Dominant joint sets recorded during site mapping of exposed quarry faces are given in Table 9.

Table 9

Joint Set	Dip (°)	Dip Direction (°)
1a	75-90	030 –350
1b	75-90	190 – 220
2a	75-90	305-330
2b	65-85	150-180
3	65-75	250-290
4a	20-35	55-85
4b	30-50	290-330

The geotechnical model for the initial design comprised a uniform rock mass of Brisbane Tuff that was slightly weathered to fresh. Potential failure mechanisms for the existing excavated tuff faces comprised planar slides and block toppling.

Analysis of the proposed 13m high, benched profiles indicated that rock anchors would be required for temporary excavation stability.

Design parameters for the clay coatings on the jointed rock mass were based on past experience and are given in Table 10.

Table 10

Rock Joint Infill Property	All Faces
Effective cohesion (c') kPa	5
Effective friction angle (Ø') degrees	26

Excavation faces A to E were indicated in Figure 5.

The excavation profile of faces B and C was to be a vertical slope, benched at 6m and 9m above the base. The bench widths were to be 3m and 5m wide respectively. Face D was to have a single bench 3m wide at 6m above the base. Face E was to be a vertical 6m high face. Faces B, C, D and E were anchored for temporary support.

Face A, was to be left in its original condition with only localised support provided to wedges. A typical excavated and anchored profile is shown on Figure 6.

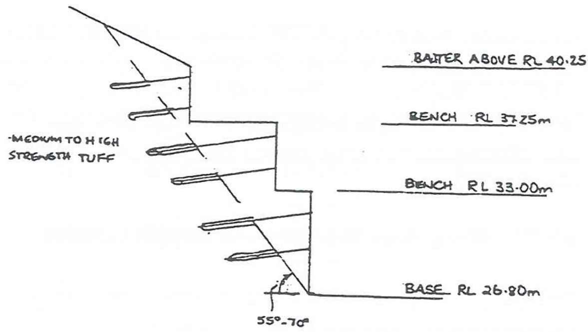

Figure 6 Typical Face Support

Each excavation profile was modeled using planar sliding wedge stability analyses incorporating various surcharge loadings for a factor of safety (FOS) of 1.5. The design estimated anchor loads per meter of excavation face are given in Table 11.

Table 11

Face (m)	Anchor Inclination (°)	Required Anchor Load (kN/m of face)
A	15	NR
B	15	1240
C	15	1775
D	15	1200
E	15	900

NR – not required

5.0 CONSTRUCTION MONITORING

The ground was excavated in a controlled manner Anchors were installed after each bench level was reached before the excavation was deepened to the next bench level. On all sides, high strength, jointed, slightly weathered tuff rock was encountered down to approximately RL 35.0 ie just above the lower bench level. The joints were generally up to 15mm wide, non-continuous, sub-vertical, either rough oxide coated or clay filled.

In the south-western corner of the excavation, at the intersection of faces C and D, the tuff rock strength deteriorated rapidly with depth (ie 1m to 2m) below RL 35.0 to reveal the contact zone. Further localised excavation on face D was undertaken to assess the quality, thickness and orientation (ie. dip/dip direction) of the contact zone materials and the underlying Neranleigh-Fernvale formation.

The mapping confirmed that the contact zone on face D dipped at 17° to 26° to the north-east and plunged to the north.

A typical cross section profile through the tuff, contact zone, and underlying phyllite on face D comprised:

1.2m to 1.5m thick very high strength rhyolitic tuff overlying a 1.5m thick contact zone (ie the older land surface). The latter comprised:

0.1m thick	low strength ash fall tuff
0.15m thick	extremely low strength, tuffaceous claystone, fissured, and slickensided
0.25m thick	very low strength carbonaceous mudstone which contained tree fragments and represented the old land surface.
1.0m thick	very low strength with zones of low to medium strength "scree breccia" and comprising fragments of the underlying phyllite.

Below the contact zone the phyllite rock mass was generally low to medium strength, moderately weathered with a cleavage dipping at 18° to 32° to the north-east and plunging to the north.

The intersection of faces C and D showing the orientation of the contact zone as it daylights in faces and dips below the floor of the excavation is shown in Figure 7 below.

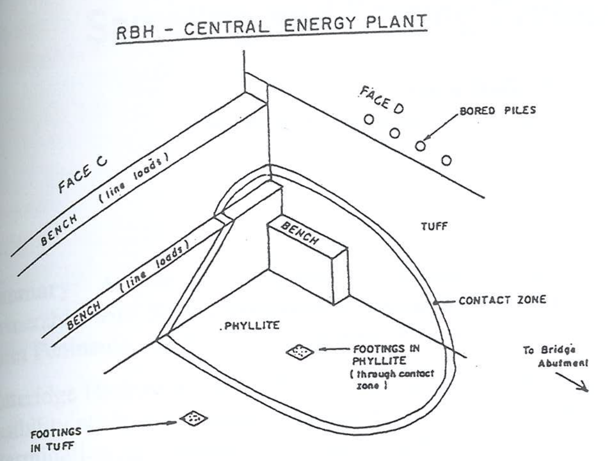

Figure 7 Intersection of Faces C & D

Due to the orientation of the contact zone and the vertical excavation of face D, a 5m to 13m high block of jointed high strength tuff was surcharging the 10m long planar surface of the contact zone. A large planar failure into the excavation was possible.

A check of the orientation of the contact zone behind the face and the length and capacity of the anchors installed above the contact zone indicated an adequate factor of safety (FOS) against planar slide.

An intrusive dyke was encountered below RL 29 in the south-west corner of the excavation, at the intersection of faces B and C. The dyke comprised very low to low strength tuffaceous mudstone which was friable and highly fissured. The mudstone dipped at 45° to 50° into the excavation and plunged north.

The remainder of the excavation encountered jointed, high strength, slightly weathered tuff with some minor moderately weathered zones.

The different rock types encountered in the excavation and the location of the contact zone and the intrusive dyke structure are shown in Figure 8.

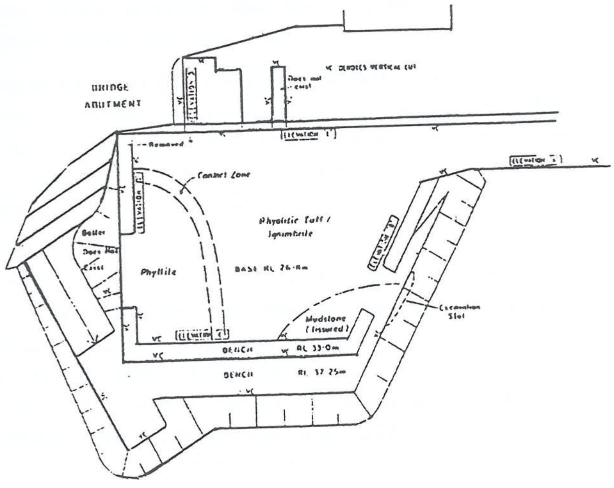

Figure 8 Plan showing rock types and discontinuities

Additional building loads were imposed on the benches as the building design evolved during construction. This necessitated the re-analysis of both local bench and global face stability.

5.1 Implications for the Development

The contact zone and intrusive dyke, which both dipped beneath the floor of the excavation and behind faces B,C, and D, had stability implications for the proposed building and excavation faces.

Due to the poorer quality materials of these major discontinuities, careful assessment of the bearing capacity and associated settlement of high level building foundations located at the floor of the excavation was required. Where insufficient depth of competent tuff existed above the contact zone/dyke materials,. bored piles were redesigned to transfer building loads to the underlying phyllite.

On face D where a major plane of instability was present the building foundations were changed to bored piles which were sleeved through tuff and contact zone and transferred the building loads to the underlying competent phyllite.

6.0 CONCLUSIONS

Where significant excavations are planned an appropriate scale of geotechnical investigation must be undertaken. The potential failure mechanisms can then be identified via modelling of the excavation . The appropriate excavation support can then be designed.

This investigation and modelling process provides a window into the site conditions. Monitoring of the excavation faces must be undertaken as it progresses to check the relevance of the model. Where conditions change, re-analysis must be undertaken and the support methods altered as required. Sometimes this has implications for the proposed development.

In the projects discussed in this paper significant shear zones or contact zones were encountered during the excavation process. In the RBH site the contact zone was not anticipated and in the Casino Carpark the extent of the shear zones could not be accurately gauged prior to excavation.

Only the on-site monitoring during excavation identified how continuous these discontinuity zones were. Proper geotechnical supervision and analysis was required on almost a continuous basis to ensure stability of the excavation and safety of the personnel working on-site.