GEOTECHNICAL MODELLING OF STATION CAVERNS FOR THE EPPING TO CHATSWOOD RAIL LINE PROJECT

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ABSTRACT

The Epping to Chatswood Rail Line project comprises a twin rail tunnel, three new underground stations and the upgrading of one existing station. The station caverns intersect a sequence of horizontally bedded shale and sandstone. The major rock defects consist of bedding planes, bedding plane seams, low angled cross bed partings and sub-vertical joint sets. Local experiences in the Sydney Basin have indicated that the behaviour of the defects and their interaction with the roof support system are critical to the performance of underground excavations. Another key geological feature that can have significant performance impact is the relatively high locked-in in-situ lateral stresses.

Each of the new stations comprises a large span platform cavern, an adjoining concourse cavern, associated escalator shafts and service buildings. The design roof support system is cable bolts with cement grouted end anchorage. The interaction between the bolts and the jointed rock mass within the influences of the various facilities is complex and thus rigorous modelling was employed. This modelling included 3D distinct element and boundary element analyses and 2D finite element approach. The complexity of the numerical models varied from homogeneous rock to layered rock with various discontinuities. Furthermore, both end-anchored and fully grouted bolts have been incorporated.

Various parametric studies were undertaken to assess the effects of various model components (rock mass, defects, in-situ stresses, sequencing and roof support installations) on cavern performance. Based on the modelling results, a system of rock bolts and construction staging has been adopted to optimise the permanent support system.

This paper is confined to the geotechnical modelling aspects of the project and presents the geological setting and the various analytical procedures undertaken. The results of the parametric studies and their impact on the final selection of the support systems are also outlined.

1. INTRODUCTION

The Epping to Chatswood Rail Line project comprises a 13 km long twin rail tunnel, three new underground stations and the upgrading of one existing station. Details of the project and various design and construction parties involved are outlined in a companion paper to this mini-symposium by Gee (2005). In particular, the design of the roof support systems for the station caverns was one of the key components of the project. As part of the roof support system design, a large amount of geotechnical modelling was undertaken to understand the behaviour of the complex interaction of the various components of the works and to predict the performance of the station caverns during and after construction.

This paper describes the geological setting of the stations and the various numerical modelling carried out to simulate the station cavern excavation. Design criteria and key findings of the numerical modelling are also briefly discussed. A further paper by Chan and Stone (2005), dealing with back-analysis and refinement of parameters forms the third paper of a set of three prepared for this mini-symposium.

2. STATION CONFIGURATION

The three new stations at Macquarie University, Macquarie Park and Delhi Road, each have 210 m long platform caverns with a 20m “brain” shaped arched span, 14m high, with similarly proportioned concourse caverns immediately alongside. The stations are served by escalator shafts and service shafts in close proximity to the caverns. The fourth station, Epping, is in a binocular arrangement with somewhat smaller spans but again with service shafts in close proximity to the arches. The arched configuration and size of the three “brain” stations are unique for Sydney which, with the close proximity of the ancillary excavations, has created some very complex interactions in the ground.

The basic forms and configurations of the stations were dictated by the owner in the reference designs of the contract documents and subject to minor changes through the process of design development.
3. GEOLOGICAL SETTING

The Epping to Chatswood Rail Line traverses through the upper strata of the Sydney Basin, namely the Wianamatta Group, the Mittagong Formation and the top of the underlying Hawkesbury Sandstone. These strata comprise a sub-horizontal sequence of middle Triassic Age.

The dominant Hawkesbury Sandstone is a medium to coarse grained, quartz sandstone sequence of fluvial (braided river system) origin. The environment of deposition is reflected in the internal sedimentary structure of the Hawkesbury Sandstone strata which display three (3) facies types, being the ‘massive sandstone facies’, the ‘sheet sandstone facies’ and the ‘shale (or mudstone) facies’.

Along the route the Hawkesbury Sandstone is locally overlain in elevated areas by a remnant capping layer of Wianamatta Group strata, which typically comprise shales with some fine to medium grained sandstones.

4. STATION CAVERN GEOLOGY

The rail link project includes the construction of three (3) new underground stations at Delhi Road, Macquarie Park and Macquarie University as well as an underground upgrade of an existing station at Epping. The Macquarie Park and Delhi Road Station cavern excavations are representative of the range of geological conditions encountered and are described below.

4.1 STATIGRAPHY AND ROCK TYPES

The station platforms and concourse caverns are located in the very top strata of the Hawkesbury Sandstone at a depth below the ground surface that ranges from 16 to 19.5 m. The thickness of Hawkesbury Sandstone in the immediate roof ranges from less than a metre to a maximum of just over 4m. The Hawkesbury Sandstone roof strata are overlain conformably by the thin and upwards grading Mittagong Formation and inturn (disconformably) by the Ashfield Shale, which is the basal and most extensive formation of the Wianamatta Group. Some residual soil and fill horizons complete the sequence to the ground surface.

The Ashfield Shale is characterised by dark grey to black shale and laminite, which in a weathered condition produces clays of medium to high plasticity. Minor fine-grained sandstone laminations can also be present in the shale sequence. Bedding within the Ashfield Shale is usually within a few degrees of horizontal, although some localised warping of up to 30° can occur. Jointing within the Ashfield Shale is generally not persistent over large areas. Sedimentary slump structures and compactional features sometimes produce localised irregular and curviplanar defects of variable orientation, some of which display slickensiding and small displacements.

The Mittagong Formation marks the transition between the marine sequence of Ashfield Shale and the fluvial sequence of the Hawkesbury Sandstone. Mittagong Formation deposits are characterised by interbedded and interlaminated siltstone, laminitie and fine to occasionally medium grained quartz sandstone. In the vicinity of Macquarie Park Station the Mittagong Formation is approximately 4 m to 5 m thick whilst at Delhi Road it is 1 m to 3 m thick. The lower boundary of the Mittagong Formation is gradational with the Hawkesbury Sandstone. The formation grades from a predominance of siltstone at the Ashfield Shale contact, downwards with an increasing percentage of fine sandstone interbeds and laminations.

The Hawkesbury Sandstone is a medium to coarse grained quartz sandstone, with occasional fine grained beds and minor shale and laminitie lenses. The Hawkesbury Sandstone comprises massive, thickly bedded and cross-bedded strata in near horizontal layers, with some siltstone/shale units that are generally less than 4m in thickness.

4.2 ROCK MASS DEFECTS

4.2.1 Bedding Planes

The most prominent rock mass defects throughout the above-described sequence are bedding planes. Bedding is sub-horizontal to gently dipping in the Ashfield Shale and the Mittagong Formation and partings are typically more closely spaced in the shale strata. Within the Mittagong Formation, bedding plane partings are more widely spaced in the siltstone units (750 mm to greater than 1.0 m) and more closely spaced in the sandstone interbeds (100 mm to 300 mm). These partings are typically planar with clay veneers and some clay seams up to 20 mm thick.

It was also observed in borehole intersections at the Delhi Road site that bedding plane seams at the base of the Ashfield Shale, and at both the top and bottom of the Mittagong Formation, display evidence of crushing and localised shearing.
Crushed seams around these stratigraphic levels range in thickness from 30 mm to 100 mm. Bedding planes characterised by weathering to clay seams, ranging in thickness from 5 mm to 100 mm, also occur.

Within the Hawkesbury Sandstone, bedding is typically sub-horizontal (0° to 10°), planar to curviplanar or undulating, variable from smooth to rough on a small scale, with some clayey or micaceous coatings. Bedding plane seams of sandy clay-clayey sand also occur, with thicknesses that can typically range from 1 to 100 mm. A mean spacing of approximately one metre between the open/fractured bedding planes is fairly typical for Hawkesbury Sandstone in the project area.

4.2.2 Cross Beds
The Hawkesbury Sandstone incorporates sheet facies strata, characterised by distinct and sometimes indistinct cross beds between the sub-horizontal primary bedding planes. The cross beds range in dip angle from 10° to 32° with a mean dip of approximately 20° towards the north-east. Cross bed partings are typically planar and smooth, with clayey, carbonaceous and/or micaceous coatings.

4.2.3 Joints
Two main sub-vertical orthogonal joint sets (J₁ and J₂) occur, with approximately NNE and ESE strikes (the latter less developed than the former). These joint sets are widely occurring, planar in nature, sometimes forming localised clusters, particularly the NNE striking set. Local experience has indicated that the majority of these joint plane defects have limited vertical persistence and are typically confined to single bed thicknesses in the Hawkesbury Sandstone. Joint sets J₁ and J₂ are typically inclined at 75° to 90°. They are undulating to planar and rough, and may be either closed or open. Coatings and infillings where present are commonly iron oxide or sandy clay respectively.

At least four other moderately dipping joint sets (J₃, J₄, J₅ and J₆) are represented locally in the Ashfield Shale and Mittagong Formation strata. Sets J₃ and J₄ may be sub-parallel in strike to the predominant Sets J₁ and J₂. Some small displacements and localised shearing can also be evident on these joints which are typically inclined around 30° to 60° from horizontal, are planar to curved, generally smooth and sometimes slickensided.

Joint spacing varies according to stratigraphy and proximity to near-surface weathering and major geological structures. As a general rule, jointing is more prevalent and closely spaced within the Ashfield Shale (e.g. 0.2 to 1 m) and increases slightly into the Mittagong Formation. Recent analysis has confirmed jointing within the Hawkesbury Sandstone is typically 2.5 m to 6 m for the main NNE striking set and more widely spaced (typically 3 m to 30 m) for the ESE striking set.

4.3 GROUNDWATER
The groundwater conditions for this site were investigated by the tunnel designer, as outlined by Gee (2005). In brief, groundwater was above cavern crowns for all stations. However, the caverns themselves were analysed / designed as “drained” facilities with no water pressure build-up in the rock bolt areas.

5. THE GEOLOGICAL MODEL

Particular attention was paid to the compilation of a geological model for each station cavern site which would form the basis for numerical modelling and support design analyses. A methodology was adopted whereby geological data was classified and categorised using both a Sydney Rock Classification system and a project specific Geotechnical Unit Classification system, which enabled the correlation of stratigraphic and weathering profiles. Estimates of rock strength, marker units such as siltstone/shale interbeds and shale clast horizons, persistent weathered or crushed bedding plane seams and the occurrence of cross bedded sandstone units, were delineated and correlated between boreholes at each site. A minimum thickness of 5 mm was adopted for bedding plane seam correlation purposes. Drilling records for water loss and hydrogeological data was also reviewed to assist correlations of open defects and crushed or weathered seams. The geological models of the Macquarie Park and Delhi Road Station Cavern sites are represented in cross section on Figures 1 and 2.
6. **IN-SITU STRESS CONDITIONS**

The natural in-situ stress state of the rock mass has significant impact on both tunnelling conditions and induced ground movements in the immediate area of the tunnelling works. The possible presence of high lateral stresses in the roof and haunches of the caverns could have direct impact upon the excavation sequencing and support design required for cavern construction. Conversely, the presence of low stresses in a low strength/jointed roof could also lead to roof support problems.

The existence of a significant (and variable) in-situ horizontal stress field within the relatively shallow Triassic rocks of the Sydney Basin is well established and is further described by McQueen (2004), Enever *et al* (1990), Pells (1990) and
An initial programme of in-situ stress (Hydrafrac) testing was undertaken in late 2002, to complement/confirm the limited tender data of this important parameter. In particular, no testing had been conducted at the critical cavern sites at tender stage. The 2002 results were significantly higher than expected, particularly some for Delhi Road Station, and as provisionally assumed for the initial design for all station facilities. A detailed review of this issue was subsequently carried out.

The Hydrafrac results for all stations, other than Delhi Road, were below the historical upper limit for Sydney (refer Figure 3) and consistent with nearby M2 Tunnel, and consequently were adopted for detailed design. This decision was subsequently verified by the back-analysis of construction monitoring at Macquarie Park station, as per Chan and Stone (2005). However, as a result of the unusually high values at Delhi Road Station (above historical limit), and their potential impact on the support design, a subsequent series of in-situ stress testing using Borehole Slotter techniques was undertaken at this location. This was done to refute, or otherwise, the previous (unexpectedly high) results. Both the Hydrafrac (for all stations) and Borehole Slotter test results are presented in Figure 3. As can be seen from this figure, the Slotter stress test results ranged from 1.1 MPa to 8.1 MPa, with some of the values at or above the Sydney Basin upper bound envelope. While the Slotter results were variable and still relatively high (compared to Hydrafrac for other caverns), the magnitudes were significantly lower than the previous Hydrafrac results in question for the Delhi Road site.

The adopted design profiles for both major and minor horizontal stresses assumed “statistics of average” for general performance prediction and were related to the overburden stress as follows:

Delhi Road Station:

\[
\sigma_H = 1.0 \text{MPa} + 6.0 \sigma_v \quad (1)
\]

\[
\sigma_h = 0.7 \text{MPa} + 4.0 \sigma_v \quad (2)
\]

All other stations:

\[
\sigma_H = 1.0 \text{MPa} + 4.5 \sigma_v \quad (3)
\]

\[
\sigma_h = 0.7 \text{MPa} + 3.0 \sigma_v \quad (4)
\]

where \(\sigma_H\), \(\sigma_h\) and \(\sigma_v\) are major horizontal, minor horizontal and vertical stresses respectively.
A horizontal to vertical stress ratio, $K=1.0$ condition was assumed for the residual soil and very weak shale (ASc). A transition zone has been assumed between the soil/shale and competent sandstone profiles.

The orientation of both the Hydrafrac results and the Slotter data at Delhi Road Station is given as Figure 4. The orientation is highly variable and ranges between $42^\circ$ (NE) and $156^\circ$ (SSE). The average orientation is $96^\circ$ (E) and is approximately parallel to the station alignment. It is noted that this average bearing at Delhi Road Station is not consistent with published trends for the Sydney Basin, which indicate an average NNE orientation. The site-specific bearing of each station was taken into consideration in modelling and design.

The unusual in-situ stress conditions at Delhi Road Station, i.e. high horizontal stresses and stress orientation, cannot be explained by topography effects. Nevertheless, it is noted that the presence of stronger, less jointed sandstone and evidence of some crushing and localised shearing of bedding plane seams appeared to support the high horizontal stress values.

7. GEOTECHNICAL PARAMETERS

7.1 ROCK MASS PROPERTIES

The geotechnical modelling approach adopted finite element models with discrete discontinuities and utilised rock moduli commensurate with the blocky nature of the idealised system. Macro defects within the rock mass were represented by the discrete discontinuities while the micro defects were modelled by equivalent elastic moduli of the rock blocks. It was assumed that the equivalent moduli of the rock blocks should be between the rock substance moduli and the rock mass moduli.

As the construction of the first (Macquarie Park) station commenced prior to the completion of the final design of the last two stations, the design of the latter stations had the advantage of the limited monitoring results from the exploratory adit excavation. These monitoring results were reviewed and back-analysed as part of the final design work for the last two stations. The results of this work, which are published in the companion paper by Chan and Stone.
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(2005), enabled a refinement of the design rock properties. In summary, while there are technically various apparent “admissible” combinations of parameters that could “match” the limited monitoring results, the limited monitoring results were generally consistent with the originally adopted geotechnical model.

The adopted rock properties for the analytical modelling of one of the (Delhi Road) stations are given in Table 5. These values are generally consistent with the rock mass parameters indicated by Bertuzzi and Pells (2002).

### Table 5: Adopted Rock Properties for Analytical Modelling – Delhi Road Station.

<table>
<thead>
<tr>
<th>Geotechnical Unit</th>
<th>Sydney Rock Classification</th>
<th>Rock Modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>Unit Weight (kN/m³)</th>
<th>UCS (MPa)</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual</td>
<td></td>
<td>0.06-0.08</td>
<td>0.3</td>
<td>20</td>
<td>-</td>
<td>25</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>ASc</td>
<td>Class V Shale</td>
<td>0.113-0.15</td>
<td>0.25</td>
<td>22</td>
<td>1-2</td>
<td>28</td>
<td>0.02</td>
<td>0.05</td>
</tr>
<tr>
<td>AScb</td>
<td>Class III &amp; IV Shale</td>
<td>1.2-1.6</td>
<td>0.20</td>
<td>22</td>
<td>1-10</td>
<td>30</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Asa</td>
<td>Class I &amp; II Shale</td>
<td>3.0-4.0</td>
<td>0.20</td>
<td>23</td>
<td>7-20</td>
<td>35</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>MFb</td>
<td>Class III Shale or Sandstone</td>
<td>1.8-2.4</td>
<td>0.20</td>
<td>23</td>
<td>2-12</td>
<td>35</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>MFa</td>
<td>Class I &amp; II Siltstone or Sandstone</td>
<td>3.0-4.0</td>
<td>0.20</td>
<td>23</td>
<td>10-40</td>
<td>40</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>HSlA</td>
<td>Class I &amp; II Siltstone</td>
<td>3.0-4.0</td>
<td>0.20</td>
<td>23</td>
<td>10-30</td>
<td>35</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>HAWa</td>
<td>Class I &amp; II Sandstone</td>
<td>6.0-8.0</td>
<td>0.20</td>
<td>23</td>
<td>15-40</td>
<td>45</td>
<td>3.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

7.2 ROCK DISCONTINUITY PROPERTIES

The deformation of rock discontinuities is a fundamental component of the performance of a jointed rock mass. Numerical modelling for the cavern support design was thus undertaken using idealised models with discrete discontinuities. Three main types of discontinuities were simulated, namely bedding plane partings, cross-beds and sub-vertical joints. Both stiffness (normal and shear) and strength parameters were assigned to each discontinuity type for each rock unit.

A number of discontinuity samples, including both natural and saw-cut joints, were selected from rock cores recovered from the site investigation. Direct shear and normal compressibility tests were undertaken on the selected samples. Given the limited number of tests undertaken and the geological similarities at the station sites, it was considered that combining all test results for assessment was appropriate.

Very limited published test data is available on stiffness properties of discontinuities in sedimentary rock, particularly in the Sydney Basin as also noted by Bertuzzi and Pells (2002). However, our laboratory test results and adopted design values were in good agreement with the limited data reported by Bandis et al (1983) and Kulhawy (1975) for sandstone and siltstone/shale. Conversely, our design values, which have been generally verified by subsequent back-analysis, are significantly lower than the estimated values given by Bertuzzi and Pells (2002).

The modelling of shear strength behaviour of discontinuities can be broadly divided into two approaches, based on clean versus infilled defects/joints, with different strength models, viz: Barton Bandis (1990) and Mohr-Coulomb models. The shear strength of clean joints was generally described by the Barton-Bandis failure criterion using three joint properties: residual friction angle (ϕr), joint roughness coefficient (JRC) and joint wall compressive strength (JCS). For clay filled joints, the shear strength of the joints is generally governed by the strength of the in-filled material dependent upon the thickness of the filling. In this instance, the cohesion, c and friction angle, ϕ were estimated from the plasticity and consistency of the in-filled clay. The adopted discontinuity parameters for analytical modelling of Delhi Road Station are shown in Table 6.

8. NUMERICAL MODELLING APPROACHES

8.1 GENERAL

The rock mass behaviour is complex and its impact on the performance of the station cavern excavation depends upon a number of factors and parameters which include the geology/stratigraphy, in-situ stress state, rock mass properties,
joint/bed stiffness and strength, location and persistence/tightness of bedding partings and cross-beds, the adopted rock support system and construction sequencing. Moreover, such factors can change along the length of a cavern.

Table 6: Adopted Discontinuity Properties for Analytical Modelling – Delhi Road Station.

<table>
<thead>
<tr>
<th>Project Specific Geotechnical Unit</th>
<th>Discontinuity Type</th>
<th>Normal Stiffness (GPa/m)</th>
<th>Shear Stiffness (GPa/m)</th>
<th>Cohesion (MPa)</th>
<th>Basic Friction Angle (deg)</th>
<th>Barton-Bandis Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASc</td>
<td>Bedding plane</td>
<td>2.0</td>
<td>0.75</td>
<td>0</td>
<td>22</td>
<td>-</td>
</tr>
<tr>
<td>ASb</td>
<td>Bedding plane</td>
<td>3.0</td>
<td>1.5</td>
<td>0</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>ASa</td>
<td>Bedding plane</td>
<td>4.0</td>
<td>1.75</td>
<td>0</td>
<td>26</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Joint</td>
<td>5.0</td>
<td>2.0</td>
<td>0</td>
<td>26</td>
<td>5</td>
</tr>
<tr>
<td>MF/AS</td>
<td>Bedding plane (clay/ground seam)</td>
<td>2.0</td>
<td>0.75</td>
<td>0</td>
<td>22</td>
<td>-</td>
</tr>
<tr>
<td>MFb</td>
<td>Bedding plane (clean)</td>
<td>5.0</td>
<td>2.0</td>
<td>0</td>
<td>26</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Bedding plane (clay filled)</td>
<td>2.0</td>
<td>0.75</td>
<td>0</td>
<td>22</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Joint</td>
<td>6.0</td>
<td>2.0</td>
<td>0</td>
<td>26</td>
<td>5</td>
</tr>
<tr>
<td>HAWa</td>
<td>Bedding plane (right)</td>
<td>8.0</td>
<td>2.5</td>
<td>5</td>
<td>32</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Bedding plane (open)</td>
<td>8.0</td>
<td>2.5</td>
<td>2</td>
<td>32</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Bedding plane (discontinuous clay filled)</td>
<td>2.0</td>
<td>0.75</td>
<td>0</td>
<td>28</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Bedding plane (clay filled)</td>
<td>2.0</td>
<td>0.75</td>
<td>0</td>
<td>22</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Cross bedding</td>
<td>6.0</td>
<td>2.0</td>
<td>0</td>
<td>28</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Joint</td>
<td>8.0</td>
<td>2.5</td>
<td>0</td>
<td>28</td>
<td>8</td>
</tr>
</tbody>
</table>

Numerical modelling of the rock mass behaviour is therefore difficult. Until recently, the design of a rock support system was typically carried out based on experience and empirical correlation alone, or on elementary modelling using homogeneous and/or continuum models. The interaction mechanism between the rock support system and the rock mass/discontinuities was seldom rigorously simulated.

However, given the recent rapid advances in computing power and in rock mechanics modelling software, state-of-the-art simulation including the interaction of various components has become feasible, particularly for large scale projects. The sophisticated modelling has enabled a much better understanding of the behaviour of a rock excavation and its impact on the rock support system including possible long term durability issues.

In order to facilitate the assessment of the complex geological/structure interaction and support requirements for the stations, various studies were undertaken for this project. Furthermore, one feature that distinguishes this design from other projects is that some of the design work was conducted after commencement of cavern construction works at the first station. The subsequent analyses thus have the benefits of the limited initial monitoring results.

In general, the station cavern excavation and support system was analysed using three different analytical computer packages. These analytical packages served to address different aspects of the design and included:

- Examine$^3$D from Rocscience Inc., Canada
- 3DEC from Itasca Consulting Group, Inc., USA
- Phase2 from Rocscience Inc., Canada

Flac from Itasca was also utilised for some verification purposes, but is not described herein.

8.2 EXAMINE$^3$D
Examine$^3$D uses a three-dimensional boundary element stress analysis approach, for linear elastic, isotropic and homogeneous media. It was used mainly for 3-D geometry visualisation of the complex cavern facilities and for
prediction of stresses and displacements of the homogeneous models. The results obtained were then compared with 3DEC and Phase² for verification purposes.

The Examine³D analysis models included the entire Macquarie Park Station and a 30m length section of the twin running tunnels at each end of the station. However, the associated service buildings at the two ends of the station were not included in the model. A typical Examine³D plot showing half of the analysed Macquarie Park Station models is shown in Figure 5.

![Figure 5: Typical Examine³D Model (Half)](image)

8.3 3DEC

3DEC is a 3-D numerical program based upon the distinct element method using discontinuous media. Because of the discrete element approach adopted, 3DEC is well suited to model geological conditions with various rock units and the presence of discrete bedding plane partings/joints. In addition, the time-marching approach allows staged construction modelling by removing blocks in stages. For the station designs, 3DEC was used to compare with the Phase² results for verification purposes and to calibrate Phase² for 3-D impact assessment using 2-D modelling. 3DEC was also used for the assessment of the relative impact of various components of the excavation, i.e. platform and concourse caverns, rock pillars and escalator shaft.

The complexity of the geological models was progressively increased, beginning with a homogenous rock model similar to the one adopted for the Examine³D work, changing to a layered rock model with various rock units for Macquarie Park, and a “bedded” model that included the rock units and discrete horizontal bedding partings. Cross bedding and sub-vertical joints were generally not simulated in the 3DEC models. However, some limited runs included a single inclined joint in the rock pillars between the platform and concourse caverns. These runs were undertaken to assess the impact of the presence of joints within the pillars on the station performance.

The model analysed was the half image of the total station development, namely: the platform cavern; the concourse and the shaft as shown in Figure 6. The initial runs assumed one stage excavation of the station. Further supplementary models were analysed to allow separation of the impact of the concourse cavern and the escalator shaft on the platform cavern. The final run also included seven stages of construction sequencing.

![Figure 6: Typical 3DEC Model (Half)](image)
However, because of the complexity of 3-D model development and the physical amount of time required for stepping to solve the equations to acceptable convergence limits, the number of runs analysed by 3DEC was limited. In particular a typical model of the station with horizontal bed partings and single stage excavation took about 1 week to develop and about 24 hours per run to analyse dependent upon the complexity and fineness of the mesh. It was thus anticipated that a fully jointed model would not be practically feasible given the computer / program limitations and project programme constraints. The simulation of a full support system with construction staging would add further complexity to the model.

8.4 PHASE\textsuperscript{2}

The Phase\textsuperscript{2} software was adopted as the main analytical tool for the geotechnical analysis of the station excavation. Phase\textsuperscript{2} is a two dimensional finite element package designed for calculating stresses and displacements and for estimating support stresses as a result of underground excavations. Phase\textsuperscript{2} discretises the geological model into finite elements and joint elements, which are assigned with pre-defined rock and discontinuity properties.

Apart from the geological model, rock bolts and liners can also be simulated in Phase\textsuperscript{2}. The rock bolts were analysed as structural elements with axial interaction only. Shear and bending of the bolts were not modelled and these effects were catered for by inspection of the analysed results.

With regard to the simulation of liner support, the liners were modelled using beam elements. Both Timoshenko and Bernoulli beam formulations were used dependent upon transverse shear deformation effect assumption. For the subject design, it was decided by the designer that the shotcrete lining would not be treated as a structural lining and thus it was not modelled.

A number of Phase\textsuperscript{2} studies and over 1000 runs were undertaken for the support system design. These runs involved sensitivity studies of various key parameters and also included the cavern shape study, interaction of various station facilities and various construction sequencing. A typical Phase2 model including the jointed rock mass, both the platform and concourse caverns and the modelled rock bolts, is shown in Figure 7.

![Figure 7: Typical Phase\textsuperscript{2} Model.](image)

9. DESIGN PHILOSOPHY AND BOLT TRIGGER LEVEL

A number of design criteria was nominated prior to the numerical modelling that were generally related to an attempt to design “rigid” cavern shapes using “laminated beam” theory. Following the initial shape study, it was recognised that this design approach was not practical for the basic station requirements.

Based on the initial numerical modelling results and experience of cavern excavations with similar geology and lateral stress fields (de Ambrosis and Kotze, 2004), significant shears along discontinuities could be expected. Current published information such as Grasselli (2005), Grasselli et al (1999), Pellet and Egger (1996) and Spang and Egger (1990) also indicates that relatively small direct shear movements (a few mm) only are needed to initiate yielding of
rock bolts under fully grouted conditions. Moreover, it was expected that cracking of cement grout would occur at such localised areas with possible sheath rupture as the magnitude of shear increased. The combination of bolt yield/sheath rupture and cracked grout at a shearing/dilating bed (with likely water ingress) was considered to pose a significant durability risk.

The design philosophy adopted by the designer was thus to maintain the arch geometry and utilise an end-anchored bolt system with installation and/or grouting delayed as long as possible in order to limit the shear impacts on the bolts. Temporary bolts were used to deal with the interim stability of the cavern roof. This “passive” approach essentially meant that the design would not attempt to resist the shears, but use a bolt system to provide for a longer gauge length in terms of the impact of any shears imposed.

With regard to the design criteria for the cavern excavation work, the allowable shear displacement of the installed rock bolts was considered as the key criteria. Moreover, there was little data in the engineering press on this issue. Based on our field experience from other projects within the Sydney Basin (e.g. de Ambrosis and Kotze, 2004), theoretical analyses (Cater, 2004), experimental work (Roberts, 2001) and published information (as referenced above), a trigger level of 15 mm shear displacement along any discontinuity in the vicinity of the installed rock bolts was recommended for re-bolting. This was adopted by the designer for support design and staging strategy.

This trigger level was subject to technical debate between various parties at the time of design and was seen by others as unduly conservative. However, subsequent prototype and field testing by the contractor’s construction stage geotechnical consultant confirmed that the 15 mm trigger level was in fact appropriate (Bertuzzi, 2004). This latter testing showed that the cable bolts would be damaged when subjected to about 20 mm of shear displacement. Furthermore, the corrugated PVC sheathing failed in rupture with about 15 mm shear movement. The surrounding grout cracked at even less shear displacement with sharp fragments of broken grout ultimately puncturing the sheathing.

10. MODELLING RESULTS

The outputs from the various analyses were used to assist in the design of the support systems and construction sequencing for the station cavern excavations. As outlined previously, details of the design are given in the companion paper by Gee (2005). The key findings from our analysis work are summarised below:

10.1 IMPACT OF CROSS BEDS

With regard to the impact of the presence/stiffness of cross beds on the cavern performance, two different cross bed stiffness models were adopted for sensitivity study purposes. One of the models assumed open cross beds with residual joint strength and average spacing of some 3-5m (termed “fully jointed” model). The alternative model assumed that the cross beds were closed and elastic with high cohesive strength (termed “bedded” model). The bedded model in Phase 1 (with elastic cross beds) is thus similar to the horizontally bedded models analysed using 3DEC.

The modelling results indicated that the shear displacements of the horizontal beds were distributed among a number of beds in the fully jointed model, while the shear displacements of horizontal beds in the bedded model were concentrated along only a few main beds (e.g. top bed at the crown and change in rock stiffness). The cumulative shear in the jointed model was similar to the localised shear in the bedded case. This behaviour implied that the impact of bed shear displacements on intersecting bolts could be less problematic in a fully jointed rock mass than that in a bedded model with closed, tight cross beds. Nevertheless, the analysis results also indicated that the impact on the rock bolts would be dependent upon the construction sequencing and the timing of bolt installation/grouting.

10.2 IN-SITU STRESS CONDITIONS

While the in-situ stress test results indicated unusually high horizontal stresses at the stations, it was considered that the in-situ stress state must be compatible with the adopted rock stiffness and strength. The modelling approach adopted thus subjected the idealised models to the design stress as a boundary condition prior to excavation and support staging. As expected, it was confirmed that the stress conditions prevailing in the cavern area after stabilisation changed from the applied uniform stress profiles to irregular stress profiles commensurate with the surrounding rock stiffness. The re-arranged and stabilised stress conditions were used as the datum for all subsequent construction staging.

It can be concluded from the various runs that the cases with higher in-situ horizontal stresses predicted larger crown sag, ground surface settlement and lateral displacements of the shaft. The shear displacements along the bedding planes and the resulting axial stresses of the rock bolts were also relatively high as a result of the assumption of the high stress state.
10.3 ROCK SUPPORT SYSTEM

The improvements in cavern performance by the installation of rock bolts were evident by comparing runs with and without bolts, in spite of the use of an essentially passive system. In particular, the main emphasis of the support was to negate the loss of “key blocks” and thus to minimise de-stabilisation of the roof. Both crown sag and ground surface settlements were halved by the inclusion of the 5m long rock bolts. A typical Phase2 plot with the predicted displacements and axial stresses of the rock bolts is shown in Figure 8.

The typical cable bolt system included an end section grouted at the time of bolt installation to provide end anchorage. The remaining 5m free length section was grouted at a later date for long-term durability considerations. The grout in the free length section was separated from the cable bolt by a “greased” or “heat shrunk” protective sheathing, thus providing some flexibility for the cable bolt to bend and making the bolt system act as permanent end-anchored bolts. Most of our Phase2 modelling therefore adopted the Phase2 end-anchored bolt model. This bolt type has the advantage of evenly distributing the axial force induced by the rock mass deformation. The modelling results showed that a number of the bolts could potentially yield in tension as a result of excavation after bolt installation. The results demonstrated that re-bolting would reduce the axial force on the new bolts.

Apart from the axial forces on the bolts, another key consideration for the long-term performance of the permanent end-anchored bolts was the magnitudes of bed shear displacements experienced by the intersecting bolts. The predicted shear displacement values were therefore inspected to assess the timing of grouting and possible impact on the anchor zone. The strategy for any excessive shear displacements was thus to delay the grouting operations or to carry out re-bolting after the shear displacements are stabilised (as discussed above).

10.4 CONSTRUCTION SEQUENCING

The Phase2 runs and the 3DEC results demonstrated that excavation sequencing was critical to the performance of the installed bolts. Whilst sequencing had minor impact on the final cavern performance, the shear displacements experienced by the installed rock bolts in the platform cavern would be greatly affected by the excavation of the escalator shafts. The predicted shear displacements near the cavern crown level and the differential horizontal displacements at the side wall of the shaft can be seen in Figure 9. By constructing the shafts first, the shear displacements along bedding planes could be induced ahead of cavern excavation. This has the effect of reducing the axial stresses of the rock bolts after bolt installation.

In addition, based on numerous stages of modelling for the various stations, it was known that the excavation of the shaft/building could induce stress relief and corresponding shear displacements along weak bedding planes. One major consequence of the bed shear displacements was their impact on the rock bolts, particularly after grouting of the bolts. Excessive shear displacements across the grout, especially in the end anchorage section, would cause yielding of the cable bolts or cracking of the grout which would impact on the long term performance of the bolts (and the caverns).
Various construction sequences were analysed during the design stage in an attempt to induce the stress relief prior to bolt installation and to minimise the bed shears experienced by the installed bolts. The modelled excavation sequence for the later series of runs also assumed that the adjoining shaft/service building is fully excavated prior to removal of the full platform top heading. It was considered that this “shaft first” sequence was the preferred construction staging option given the nominated platform/concourse/shaft layout.

10.5 IMPACT OF INCLINED JOINTS WITHIN ROCK PILLAR
The integrity of the rock pillars between the concourse and platform caverns is critical in ensuring the stability and satisfactory performance of the station caverns. In particular, the 3DEC results revealed that the presence of inclined joints within the pillars could adversely affect the crown sag and the corresponding ground surface settlements.

The adopted strategy to deal with the above pillar issue was therefore to investigate the stiffness of the pillars and the presence of any joints within the pillars ahead of excavation around the pillars. Borehole drilling at the pillars was therefore undertaken from the ground surface. In addition, exploratory excavation was carried out to inspect and map one sidewall of each of the two pillars for any unfavourable geological conditions.

Figure 9: Predicted Displacements Near the Escalator Shaft.

11. CONCLUSIONS

The performance of roof support systems for underground caverns depends heavily on the geology of the area and the behaviour of the surrounding rock mass as a result of excavation. Detailed geological assessment and appropriate geotechnical modelling are therefore the key to the success of roof support system design.

For large-scale infrastructure projects, particularly with large span underground openings, it is thus important to develop an appropriate geological model including the nature and extent of various defects. Such geological models can then be analysed using appropriate numerical simulation in order to appreciate the mechanisms of the various behaviours and to predict the likely performance of the caverns and roof support systems. The predicted performance can then be used to compare with construction-stage monitoring results for verification and risk management purposes.
For the subject project, the unique wide span “brain” shaped caverns with low roof cover, together with the bedded sedimentary rock environment and unusually high in-situ stress conditions created significant engineering challenges in the roof support design. Moreover, the decision to rely solely on the permanent rock bolts as the primary support for a 100-year design life meant that the rock bolt design could not be undertaken using rudimentary modelling or empirical design approaches. Over 1000 modelling runs were conducted using various state-of-the-arts modelling software and sophisticated idealised models. The modelling results were then used by the designer to determine the rock bolt details and the appropriate construction sequencing.

Finally, it is noted that the numerical models had been verified in part by subsequence construction-stage monitoring and back-analysis as presented in a companion paper by Chan and Stone (2005).

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13. REFERENCES


