BACK-ANALYSIS OF MONITORING RESULTS AT MACQUARIE PARK STATION, EPPING TO CHATSWOOD RAIL LINE

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ABSTRACT

The design of the Epping to Chatswood Rail Line project was conducted nearly in parallel with the initial construction work. The Macquarie Park Station caverns were partially excavated while the design of the other three stations was still underway. This provided an opportunity to use as-constructed performance to refine the design parameters for the subsequent caverns.

Geotechnical monitoring of the initial Macquarie Park Station excavation included inclinometers, extensometers, surface settlement points, endoscopes, convergence points, crown sag points and rock bolt load cells. The geology of the excavation faces was also carefully mapped.

On the basis of the mapping, the geological model for this station was refined slightly. The monitoring results were reviewed and back-analysed. The rock mass moduli and the joint stiffness values of the various rock units were varied to “match” the various monitored behaviour of the excavation.

The back-analysis work generally indicated that the original models adopted for design were reasonable. Two “admissible” combinations of slightly revised geotechnical parameters were identified. Models for subsequent design of the other stations were adjusted to reflect the calibrated parameters. The back-analysis work was also consistent with the relatively high in-situ stresses adopted for the project.

This paper discusses the back-analysis work undertaken and demonstrates that appropriate monitoring is a useful tool for verifying and refining design models.

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, tunnelling work for such infrastructure projects generally includes various forms of movement monitoring and geological mapping, for design verification purposes. The monitoring results are usually compared with pre-set movement trigger levels for risk management purposes and action as appropriate. However, the results are rarely back-analysed rigorously, possibly due to technical limitations and/or project budget constraints. In the past, some attempts have been made to back-analyse tunnelling monitoring results using simple techniques (e.g. Kirsten, 1976; Pells *et al*, 1981). Nevertheless, such techniques usually assume that the rock surrounding the tunnel behaves as an isotropic, homogeneous, elastic medium. While this assumption could be valid for competent rock at great depth, it has been well documented that the sedimentary rock at shallow depth typically found within Sydney Basin is horizontally bedded and contains various defects/discontinuities. It thus follows that the application of the simplistic back-analysis techniques might not be appropriate for the sedimentary environment. Other more rigorous back-analysis methodology is therefore desirable if the behaviour of the rock is to be simulated correctly.

This paper documents the back-analysis work undertaken for the Macquarie Park Station caverns and demonstrates that appropriate monitoring and back-analysis can be a useful tool for verifying and refining design models and parameters. A further paper by Chan, Kotze and Stone (2005), dealing with geotechnical modelling of station caverns is the other paper of a set of three prepared for this mini-symposium.

2 PROJECT APPRECIATION

The Epping to Chatswood Rail Line project consists of twin bore rail tunnels, running from the Main North Line at Epping to Chatswood on the North Shore Line, via stations at Epping, Macquarie University, Macquarie Park and Delhi Road. Each of the stations comprises a platform cavern, a concourse cavern with passenger access to the surface by way of escalator, stair and lift shafts. Service shafts and buildings are also included within each station complex. The details and interaction of the various facilities within a station have been outlined in a companion paper presenting the geotechnical modelling work undertaken for the station excavations.

3 MONITORING PROGRAMME

As an integral part of the design process, a monitoring programme was developed by the designer, as outlined by Gee (2005). This instrumentation was installed by GHD Longmac and included inclinometers, surface extensometers, crown extensometers, surface settlement points, endoscopes, convergence points, crown sag points and load cells. It is noted that endoscopes formed a significant part of this monitoring strategy, following on our experience with the Elgas caverns (de Ambrosis and Kotze, 2004). The monitoring was generally carried out by the contractor's construction-stage geotechnical team. The general layout of some of the key instruments at Macquarie Park Station is given on Figure 1.

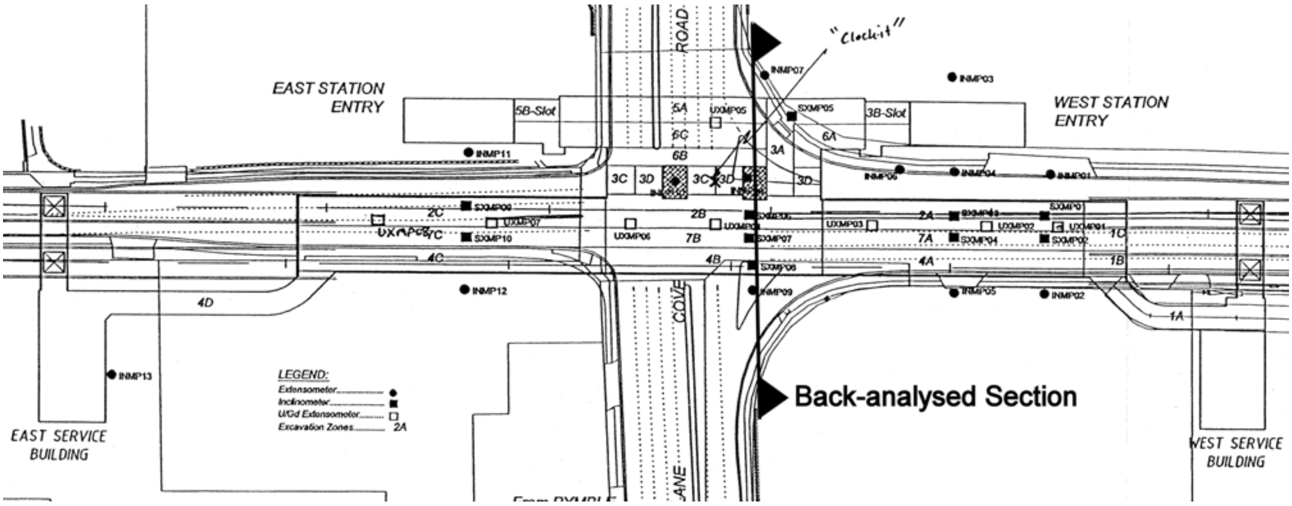

Figure 1: General Layout of Some of Key Instruments Installed at Macquarie Park Station

Because of construction timing constraints and construction activities, a number of the instruments was installed after excavation commenced. The position of the excavation face relative to each instrument during datum reading was therefore important. While any displacements that may have occurred prior to rock bolt installation would not affect the performance of the rock bolts, displacements induced by the excavation before datum readings were taken were critical in the back-analysis work.

4 BACK-ANALYSIS APPROACH

The possibility that the excavation of the various facility openings at each station could result in significant shear displacements being induced along bedding planes in the rock was appreciated early in the initial cavern design process. These shear displacements were considered problematic, in that they provided a concentrated point of loading for any intersecting reinforcement. The potential detrimental impacts of these localised loadings upon the installed rock bolts and their corrosion protection were of concern, as outlined in our companion paper (Chan Kotze and Stone, 2005). In response to these concerns, a 'shaft first' excavation sequence was proposed for one of the stations (Macquarie University Station). Such a sequence manages the detrimental effects of these bed shears by delaying the placement and grouting of rock bolts so that a significant proportion of the induced shears would have occurred prior to the installation of the rock bolts.

However, site acquisition constraints had made a 'shaft first' construction sequence for Macquarie University Station impractical and thus made a more refined analysis of the eastern portion of Macquarie University Station necessary. It was then decided that the initial excavation monitoring results available at the time should be back-analysed in order to confirm the geotechnical model/behaviour and to refine the design parameters.

In summary, the requirements for the back-analysis work were as follows:

- The limited excavation work carried out at the Macquarie University Station at the time of our back-analysis meant that the monitoring results available from this site would not make any back-analysis meaningful. A concentrated effort was thus made to remodel the observed adjacent Macquarie Park Station ground responses. It was then proposed that any refinements made for Macquarie Park Station should be then applied to the re-analysis of Macquarie University Station.
- Whilst the calculation of shear displacements along bedding planes was considered as a key requirement, it was proposed that the back-analysis should attempt to match the full suite of monitored behaviour (i.e.

ground surface displacements, roof sag, side-wall convergence and bedding shears directly adjacent to and at some distance from the excavation). This is contrary to the conventional approach of using simple homogeneous models for back-analysis of convergence monitoring in isolation from other behaviour.

- Attempts should also be made to ensure the correlation remained equally applicable for each of the completed construction stages.
- The installed instrumentation at Macquarie Park Station was generally grouped into a number of distinct 'sections'. Due to time constraints, the decision was made to provide a detailed back-analysis of only one of these instrumentation sections. Following an initial review of the available data, the monitoring section near the centre of the station was selected (Refer Figure 1). This section was chosen because of the relatively large amount of monitoring data available here (3 inclinometers, 3 surface extensometers), of the critical nature of the cavern geometry at this point (this section cuts through the platform cavern, the concourse cavern and the concourse pillar) and of the magnitude of the movements already recorded in these instruments.

5 GEOLOGICAL MODEL

The basic geology of the station sites has been described in the companion paper (Chan, Kotze and Stone, 2005) dealing with the geotechnical modelling work. In summary, the stations intersect a typical sequence of sedimentary rock found in the Sydney Basin, comprising Ashfield Shale, the transitional Mittagong Formation and the underlying Hawkesbury Sandstone. The sequence is predominantly horizontally bedded with major defects consisting of bedding planes, bedding plane seams, low angled cross bed partings and sub-vertical joint sets. Based on the local variations at each station, idealised geological models were developed for each station for numerical modelling purposes.

At the time of commencing the back analysis, the construction works at Macquarie Park Station had advanced to a stage where the full top heading in the platform cavern, the concourse cavern cross-link tunnels and two top headings in the concourse cavern had been fully excavated. Face mapping conducted during this excavation process was used in conjunction with preliminary monitoring data to revise the geological model at this location. Care was taken to ensure that bedding discontinuities identified from mapping and monitoring were represented within the revised numerical model.

Figures 2 and 3 present the pre-construction and back-analysis stage numerical models respectively. A number of important changes existed between the two models:

- The previously assumed horizontal bedding has been changed so that it dipped slightly (approximately 2 degrees) to the NE.
- In particular the revised geology as shown in Figure 3 included a 'clay filled' bedding plane intersecting the crown of both the platform and concourse caverns. In the pre-construction stage model, the bedding plane that intersected the crown of the concourse cavern passed above the platform concourse crown. As is mentioned above, the decision to locate this slightly dipping bed in the cavern crowns was based on face mapping, visual inspection and monitoring data.
- A transitional upper Hawkesbury Sandstone zone had been added to the revised model below the base of the Mittagong Formation.
- The level of the base of the Mittagong had been altered to reflect endoscope observations and additional borehole information.

The geological model also considered relatively widely spaced 'plastic cross beds' based on face mapping results.

6 EXCAVATION SEQUENCE

In addition to the revisions made to the geological model, a number of changes were required to the construction sequencing considered by the numerical analyses. Figures 4 presents the back-analysis stage excavation sequence. The back-analysis stage sequence had been developed to mirror the actual excavation sequence as much as was practicable. Variations between the actual construction sequence and that assumed during pre-construction analysis, means that great care must be taken when comparing actual monitored data with pre-construction calculations.

Furthermore, a number of changes to the shape of the platform cavern profile and intermediate construction stages were made to reflect the slight changes in the as-constructed geometry such as the crown of the headings. Again the effects of such changes needed to be considered when comparing actual monitored data with pre-construction calculations. In particular, caution needed to be exercised when comparing the relative impact of different construction stages, as changes to heading geometries affects the extent of stress relief associated with each heading.

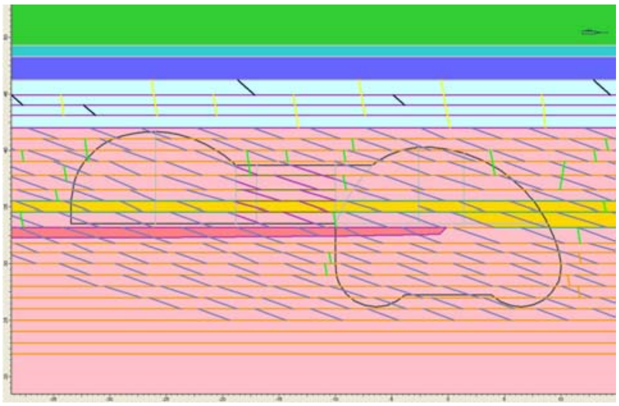

Figure 2: Pre-construction Finite Element Model

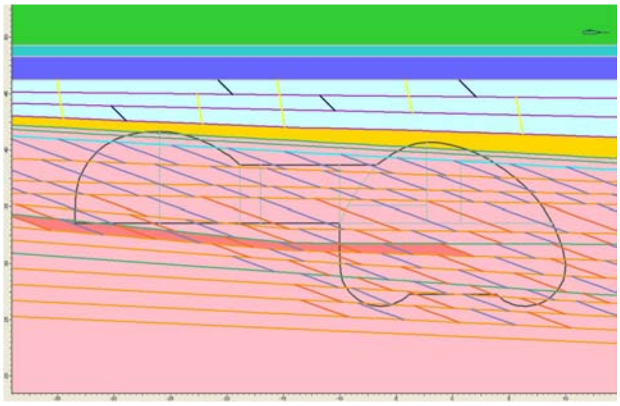

Figure 3: Back-analysis Finite Element Model

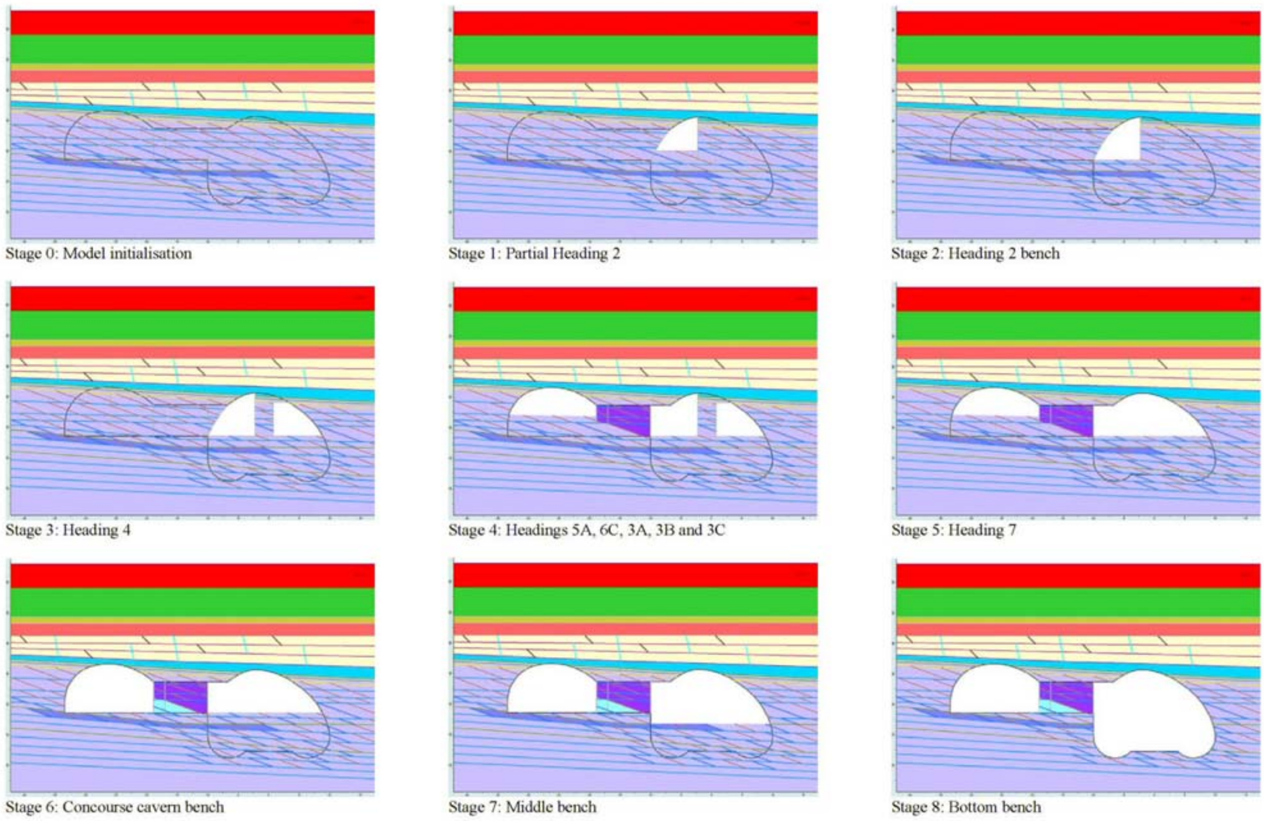

Figure 4: Back-analysed Construction Stages

7 SUMMARY OF MONITORING RESULTS

The initial task undertaken for the back-analysis work was to collate and review the subject Macquarie Park Station monitoring results. Some general observations regarding the instrumentation located in the proximity of the back-analysed section are summarised as follows:

- A line of surface survey marks was located generally along the line of the cavern. Survey data indicated that approximately 15mm of ground surface settlement had occurred above the platform cavern crown as a result of the excavation of the platform and concourse top headings.
- Two crown sag points were located in the general vicinity of the section. Delays recording datum readings for these two points had meant that they could not be used to assess the impact of the partial first heading

excavation. Notwithstanding these limitations, these points as of the 17/7/03 had recorded sags of 11 and 15mm as a result of subsequent excavation stages.

- Three inclinometers were located along the analysis line. Delays recording the datum for one of the inclinometers (INMP09) had meant that this inclinometer could only be used to observe the effect of excavation after the partial first heading excavation. Similarly, the proximity of the partial first heading excavation face to the other two inclinometers (INMP07 and INMP08) at the time of datum reading had meant that there existed some doubt that these inclinometers had recorded the full effect of partial heading excavation. A number of independent methods were used to try and estimate the unrecorded displacements. These are discussed further in section 10. The uncorrected inclinometer data showed approximately 9.5mm shear across the 'clay' bedding plane that intersected the crown of the two caverns in INMP07 and approximately 9mm across this same bed in INMP08. INMP09 showed about 4mm shear across this bed since the removal of the partial first heading excavation.

- Discernable shear displacements (about 1-2mm) were recorded in INMP07 due to the excavation of the partial first heading. This is notable because INMP07 was some distance away (approximately 30m) from the excavation associated with the partial first heading, indicating the large influence zone from the cavern excavation.

- Two sidewall convergence points were located close to the analysis section. One of the points had difficulties with initial readings and as a result was datumed 15.5m (approximately 2.5diameters of the first heading opening) behind the face. Notwithstanding this, this point still recorded 7.1mm convergence due to the partial first heading excavation. The other convergence point was datumed 1m from the face. When the excavation had continued to a point where the pin was 14.5m from the face of the initial partial heading, this pin recorded 26.5mm convergence. A subsequent reading found the pin to be loose and as such the point was abandoned and the 26.5mm reading deemed unreliable. A range of 23-27mm sidewall convergence had been observed elsewhere as a result of excavation of full first heading.

8 ANALYTICAL MODELS CONSIDERED

The analytical approach and numerical models adopted for the modelling of the station caverns are described in detail in the companion paper (Chan, Kotze and Stone, 2005). In summary, the finite element models included macro defects within the rock mass, while micro defects were simulated by equivalent elastic moduli of the elastic rock blocks between the macro defects. The equivalent moduli of the rock blocks were chosen to be between rock substance moduli and the rock mass moduli.

With regard to the rock discontinuities, three main types of defects were simulated, namely main bedding partings, cross-beddings and sub-vertical joints. Both normal and shear stiffness parameters were assigned to each discontinuity type. The shear strength behaviour of the discontinuities were modelled using either Barton-Bandis failure criterion or Mohr-Coulomb model as appropriate.

It was recognised early that, due to the complex nature of the analytical approach adopted, a large number of possible models with various combinations of parameters could be appropriate. In order to deal with this complexity in a timely manner, the parameters were broadly divided into three main groups, namely in-situ stress conditions, rock mass properties and discontinuity properties. A preliminary series of back-analyses was then run targeting each of these three parametric groups. The main purpose of these initial runs was to gain an insight into the model sensitivity to each of the selected groups.

The results of the preliminary work indicated that we could not "match" the overall monitored cavern response using a lower in-situ stress state alone. This agreed with the results of simple homogeneous analyses (convergence only), which also inferred that the in-situ stresses at this location could be relatively high. As such, the in-situ stress conditions adopted for the pre-construction design were assumed to be applicable for the back-analysis work.

This assumption reduced the considered parameters to two broad groups (rock mass properties and discontinuity properties). A number of finite element runs using computer programme Phase^2 were carried out by adopting different combinations of rock mass modulus factors (RMF) and joint stiffness factors (JSF). These factors were multipliers used to vary the base values of various rock mass moduli and joint (normal and shear) stiffness. Table 1 presents the parametric combinations considered.

Table 1: Combinations of Back-analysed Parameters Considered

Case No.	Rock Modulus Factor (RMF)	Joint Stiffness Factor (JSF)
1	1	1
2	1	10
3	1	100
4	1	1000
5	2	1
6	2	10
7	2	100
8	2	1000

9 COMPARISON OF CALCULATED AND MONITORED RESULTS

A systematic method was then developed for comparing the calculated ground response to the collected monitoring data. The modes of cavern behaviour included in the comparison were:

- Shear displacements at inclinometers INMP07, INMP08 and INMP09
- Ground surface settlement
- Sidewall convergence
- Crown sag

By way of example, Figure 5 shows a plot with surface displacements after excavating Heading 2,4,7 and concourse top heading. The calculated surface displacements (above the centre of the platform cavern crown) are presented on this plot for the following six parameter combinations as shown.

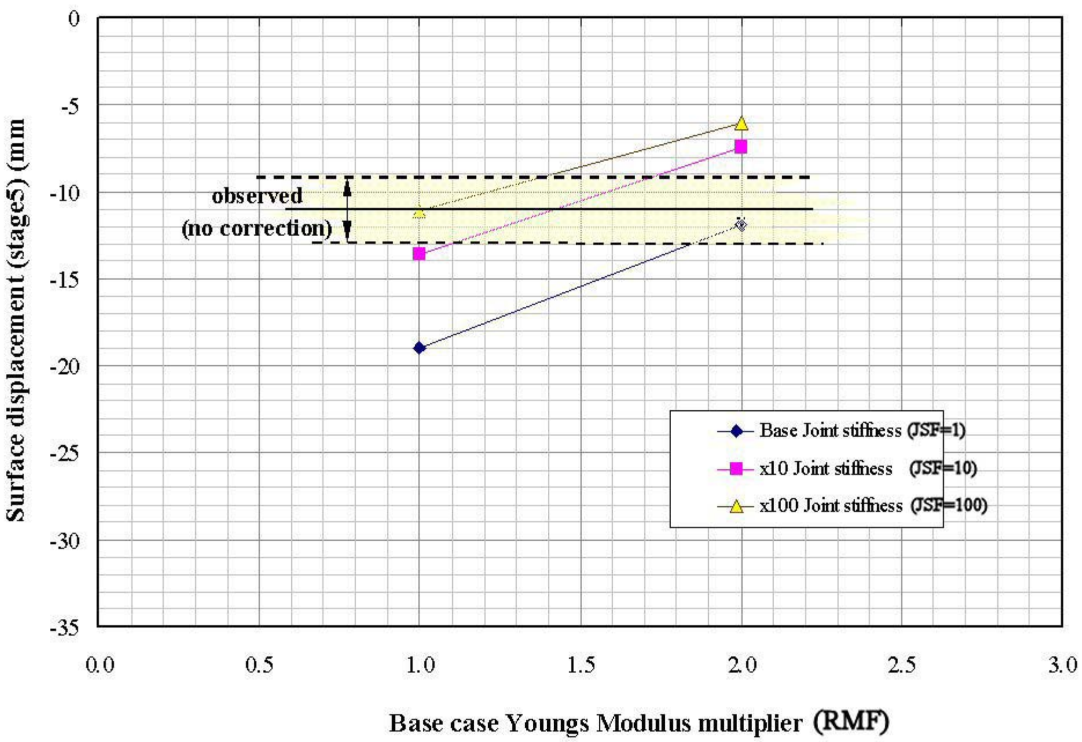

Figure 5: Plot of Calculated and Measured Ground Surface Displacements with Various Combinations of Rock Mass and Joint Stiffness Parameters

The calculated value of surface displacement for each analysis forms the ordinate for each data point, while a linear variation was assumed for each queried model response in respect to RMF and JSF (i.e. for a given JSF, the calculated surface displacement was assumed to vary linearly with RMF).

The actual observed surface displacement was about 9-13mm. For an average surface displacement of 11mm, the back-figured admissible combinations of parameters thus varied from RMF=2.1/JSF=1 to RMF=1.0/JSF=100.

By considering the admissible parameter combinations for a number of cavern displacement modes and in particular for orthogonal displacement modes, the range of admissible parameter combinations can be reduced. This process of model vetting is presented visually in Figure 6.

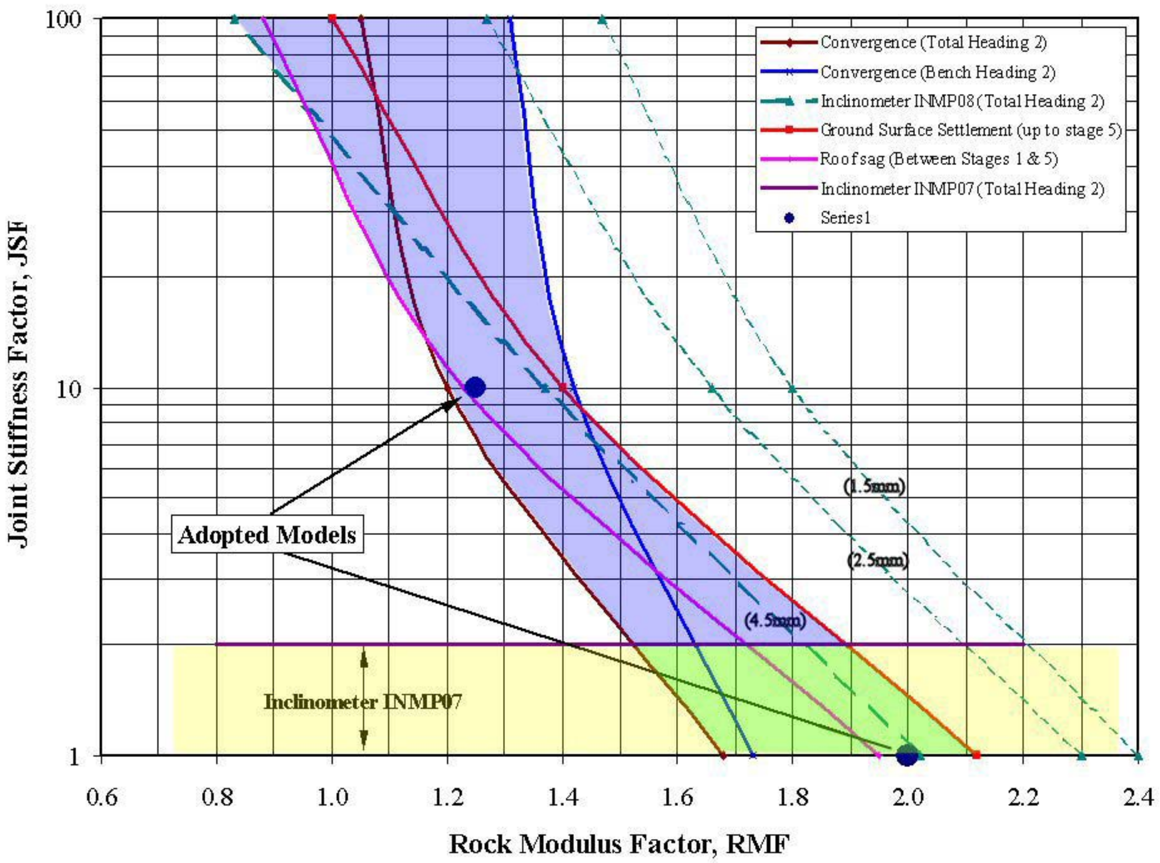

Figure 6: Summary of Back-analysis Results

As can be seen from Figure 6, the admissible parameter combinations for six different modes of ground response are presented with respect to JSF and RMF as follows:

- Sidewall convergence due to total initial Heading 2,
- Sidewall convergence due to Heading 2 bench,
- shear displacements across the clay bed intersecting the cavern crown in INMP08,
- ground surface settlement above platform cavern crown (as per Figure 5),
- roof sag between model stages 1 and 5 and
- bed shear as observed in INMP07.

With regard to the shear displacements at INMP08, it was recognised that the datum reading was taken when the excavation face was close to the subject inclinometer. In addition, some uncertainties existed as to the exact chainage of the excavation face at the time of datum reading. Depending upon the reported face chainage, it was estimated that the correction for the shear displacements could vary from 1.5mm to 4.5mm. The parameter combinations for three different corrections (1.5mm, 2.5mm and 4.5mm) have been plotted on Figure 6. A review of these three corrections appears to indicate that the 4.5mm correction is more consistent with the other modes of ground response than the other two corrections.

Using this approach the parameter combinations that provided the most comprehensive level of agreement with all six modes of deformation can be identified. The following two parameter combinations were adopted as reasonable models:

- Low joint stiffness model: RMF=2 and JSF=1
- High joint stiffness model: RMF=1.25 and JSF=10

As can be seen from Figure 6, both models provided a reasonable calibration with the monitored behaviour. It is however worth noting that when the inclinometer response at the far side of the concourse cavern (INMP07) is taken into consideration, then the low joint stiffness model seems to provide a slightly better correlation to the observed behaviour.

The back-analysed rock mass and discontinuity properties for Macquarie Park Station are given in Tables 2 and 3 respectively, for the low joint stiffness model.

Table 2: Back-analysed Rock Properties – Macquarie Park Station

Geotechnical Unit	Sydney Rock Classification	Rock Modulus (GPa)	Poisson's Ratio	Unit Weight (kN/m^3)	UCS (MPa)
Residual	-	0.04	0.3	20	-
ASc	Class V Shale	0.2	0.25	22	1-2
ASb	Class III Shale	2.0	0.20	22	7
ASa	Class II Shale	3.0	0.20	22	10
MFb	Class III Shale or Sandstone	3.0	0.20	23	10-15
HAWa (upper)	Class II Sandstone	4.0	0.20	23	15-20
HAWa (lower)	Class I Sandstone	8.0	0.20	23	30-40
HSIb	Class III Siltstone	1.5	0.20	23	15

Table 3: Back-analysed Discontinuity Properties – Macquarie Park Station

Project Specific Geotechnical Unit	Discontinuity Type	Normal Stiffness (GPa/m)	Shear Stiffness (GPa/m)	Cohesion (MPa)	Basic Friction Angle (deg)	Barton-Bandis Parameters	
						JRC	JCS (MPa)
AS	Bedding plane	2.0	1.0	0	25	-	-
MFb	Bedding plane	2.0	1.0	0	25	5	2.0
	Joint (sub-vertical)	20.0	8.0	0	30	5	2.0
	Joint (35°-45° dip)	1.5	0.5	0	25	1	1.0
HAWa	Bedding plane (tight)	3.0	1.5	5	32	-	-
	Bedding plane (open)	3.0	1.5	0	27	5	4.0
	Bedding plane (clay filled)	1.0	0.5	0	22	-	-
	Cross bedding (tight)	3.0	1.5	8	35	-	-
	Cross bedding (open)	3.0	1.5	0	27	8	1.0

10 CONCLUSIONS

In summary, the back-analysis work utilised various excavation monitoring results including lateral displacements, ground surface settlements, crown sags and bedding plane shear. The adopted back-analysis approach was to adjust the various geotechnical model/parameters using the "actual" geological model and construction sequence in order to "match" the measured behaviour.

Preliminary back-analysis work confirmed that the previously adopted high in-situ stress conditions were appropriate for the subject site. The subsequent back-analysis work thus targeted mainly on variations of rock mass moduli and of joint stiffness values.

Based on this back-analysis exercise, the most 'admissible' combination of geotechnical parameters (Rock Modulus Factor (RMF) and Joint Stiffness Factor (JSF)) were identified as providing a reasonable correlation with the observed behaviour:

- Low joint stiffness model: RMF=2 and JSF=1

It is acknowledged that this combination did not represent the full set of 'admissible' models. However by conducting the above back-analysis, the uncertainties of the models had been greatly reduced from the models used during pre-construction stage design.

It is considered that the complex back-analysis work is justifiable and beneficial to the project when complex geometry and geological conditions together with an optimised design are present. The back-analysis results for the subject project confirmed that the pre-construction geotechnical models and design parameters were appropriate for the station cavern excavation work.

11 REFERENCES

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