Tunnel Modelling – Differences in Behaviour Between Homogeneous Models and Discontinuum Models

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ABSTRACT: The behaviour of underground excavation in jointed rock mass is complex and depends upon a number of factors and parameters. Numerical modelling of such rock mass behaviour is therefore difficult. Conventionally, numerical modelling of tunnel excavation was typically carried out using idealised equivalent homogeneous rock and/or continuum models. However, with recent rapid advances in technology, simulation using discontinuum models has become commercially viable for tunnel designers. A study has been conducted using distinct element modelling techniques to assess the differences in behaviour among models with various complexities. The numerical models included homogeneous models, layered continuum models and discontinuum models with horizontal beds. It was found that the differences in behaviour were dramatic and would lead to very different predictions in underground excavation behaviour.

1 INTRODUCTION

Rock mass behaviour is complex and its impact on the performance of underground excavations 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. Moreover, such factors can change along the length of the underground excavation.

Until recently, the design of a roof support system for tunnelling 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 roof 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, simulation including the interaction of various components has become feasible. The sophisticated modelling has enabled a much better understanding of the behaviour of a rock excavation and its impact on the roof support system.

This paper uses a case study of underground excavation design to illustrate the differences in predicted behaviour between continuum models and discontinuum models.

2 PROJECT APPRECIATION

The Epping to Chatswood Rail Link was a new major railway project in Sydney, Australia and comprised a 13km long twin rail tunnel, three new underground stations and the upgrading of one existing station. Details of the project were presented in Chan *et al.,* (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.

Each of the stations comprised a platform cavern, a concourse cavern with passenger access to the surface by way of escalator, stair and lift shafts. Service shafts and buildings were also included within each station complex.

The three new stations each had 210m long platform caverns with a 20m "brain" shaped arched span, 14m high, with similarly proportioned concourse caverns immediately alongside. The stations were served by escalator shafts and service shafts in close proximity to the caverns.

3 PROJECT GEOLOGY

3.1 *Geological setting*

The Epping to Chatswood Rail Link 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 origin. 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.

The caverns were located in the very top strata of the Hawkesbury Sandstone at a depth below the ground surface that ranges from 16 to 20m. The thickness of Hawkesbury Sandstone in the immediate roof ranged from less than 1m to a maximum of just over 4m. The Hawkesbury Sandstone roof strata were overlain by the thin and upwards grading Mittagong Formation and in turn by the Ashfield Shale. Some residual soil and fill horizons complete the sequence to the ground surface.

The adopted geological model of a typical station is shown in cross section on Fig. 1.

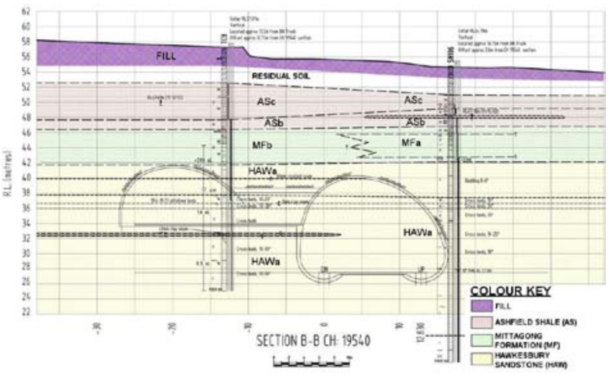

Fig. 1 Macquarie Park station – geological model

3.2 *Structural geology*

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. These partings are typically planar with clay veneers and some clay seams up to 20 mm thick.

Within the Hawkesbury Sandstone, bedding is typically sub-horizontal, 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.

The Hawkesbury Sandstone incorporates distinct and sometimes indistinct cross beds between the sub-horizontal primary bedding planes. The cross beds range in dip angle from 10º to 32º towards the north-east.

Two main sub-vertical orthogonal joint sets occur and are widely occurring. The majority of these joint plane defects have limited vertical persistence and are typically confined to single bed thicknesses in the Hawkesbury Sandstone.

4 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 impact of in-situ stresses on underground excavation has been assessed and reported by Chan and Stone (2014).

The existence of a significant (and variable) in-situ stress field within the relatively shallow Triassic rocks of the Sydney Basin is well established and has been described by McQueen (2004), Enever *et al* (1990), Pells (1990) and Enever (1999).

For the case study, the horizontal stresses were the major principal stresses while the vertical stress was the minor principal stress, consistent with typical Sydney Basin geological environment. Two different horizontal stress assumptions were analysed as follows:

$$\sigma_H = 1.0\ \sigma_v \quad \text{(i.e. K=1.0)} \qquad (1)$$

$$\sigma_H = 4.0\ \sigma_v \quad \text{(i.e. K=4.0)} \qquad (2)$$

where σ_H and σ_v are major horizontal and vertical stresses respectively.

5 GEOTECHNICAL PARAMETERS

The geotechnical parameters for the rock mass and rock discontinuities were selected based on typical values for sandstone and shale encountered in the Sydney Basin. The adopted parameters are given in Table 1.

6 NUMERICAL MODELLING APPROACH

Because of the complexity of the station set up and the interaction among the various components, 3-D modelling using a distinct element software package, 3DEC from Itasca Consulting Group was adopted for the study. 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.

The complexity of the geological models was progressively increased, beginning with a homogenous rock model, changing to a layered rock model with various rock units and a "bedded" model that included the rock units and discrete horizontal bedding partings. Cross bedding and sub-vertical joints were not simulated in the 3DEC models due to the complexity of 3D modelling, but were included in subsequent 2D modelling. The 2D cross bed and sub-vertical joint modelling confirmed the general behaviour of the "bedded" model results, but is not reported herein.

Table 1. Adopted Parameters for analysis

Rock Type	Rock Mass Parameters	Bedding Parting Parameters
High strength Sandstone	E = 4,000 MPa ν' = 0.2 γ = 23 kN/m^3 ϕ' = 45 deg. c' = 5 MPa f_t = 1.5 MPa	k_n = 10,000 MPa k_s = 4,000 MPa c_{peak}' = 0.1 MPa $c_{residual}'$ = 0 MPa ϕ' = 25 deg. f_t = 0 MPa
Medium to high strength Sandstone	E = 2,000 MPa ν' = 0.2 γ = 23 kN/m^3 ϕ' = 40 deg. c' = 2.5 MPa f_t = 0.75 MPa	k_n = 8,000 MPa k_s = 2,000 MPa c_{peak}' = 0.075 MPa $c_{residual}'$ = 0 MPa ϕ' = 24 deg. f_t = 0 MPa
Medium to high Strength Shale	E = 1,000 MPa ν' = 0.25 γ = 23 kN/m^3 ϕ' = 35 deg. c' = 2 MPa f_t = 0.35 MPa	k_n = 6,000 MPa k_s = 1,500 MPa c_{peak}' = 0.06 MPa $c_{residual}'$ = 0 MPa ϕ' = 23 deg. f_t = 0 MPa
Low Strength Shale	E = 500 MPa ν' = 0.25 γ = 22 kN/m^3 ϕ' = 30 deg. c' = 0.5 MPa f_t = 0.05 MPa	k_n = 2,000 MPa k_s = 500 MPa c_{peak}' = 0.02 MPa $c_{residual}'$ = 0 MPa ϕ' = 21 deg. f_t = 0 MPa

Where E = Young's modulus, ν' = Poisson's ratio, γ = unit weight, ϕ' = friction angle, c' = cohesion, f_t = tensile strength, k_n = normal stiffness, k_s = shear stiffness

The model analysed was the half image of the total station development, namely: the platform cavern; the concourse and the shaft as shown in Fig. 2. One stage excavation of the station was assumed in the study.

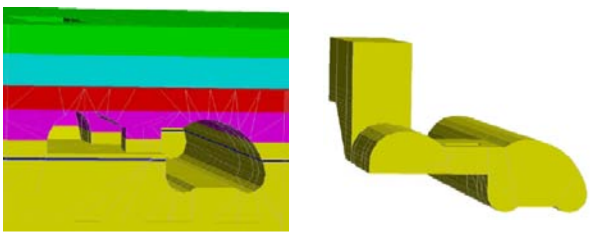

Fig. 2 Typical model with horizontal layered rock units

7 MODELLING RESULTS

The 3DEC results were assessed at various cross sections such as those shown on Figs. 3 and 4 for the K=4 case. The total displacement results for the various runs are presented in Tables 2 and 3 for K=1 and K=4 respectively. The cross sections reported herein include sections at the centreline of the rock pillar and through the escalator shaft.

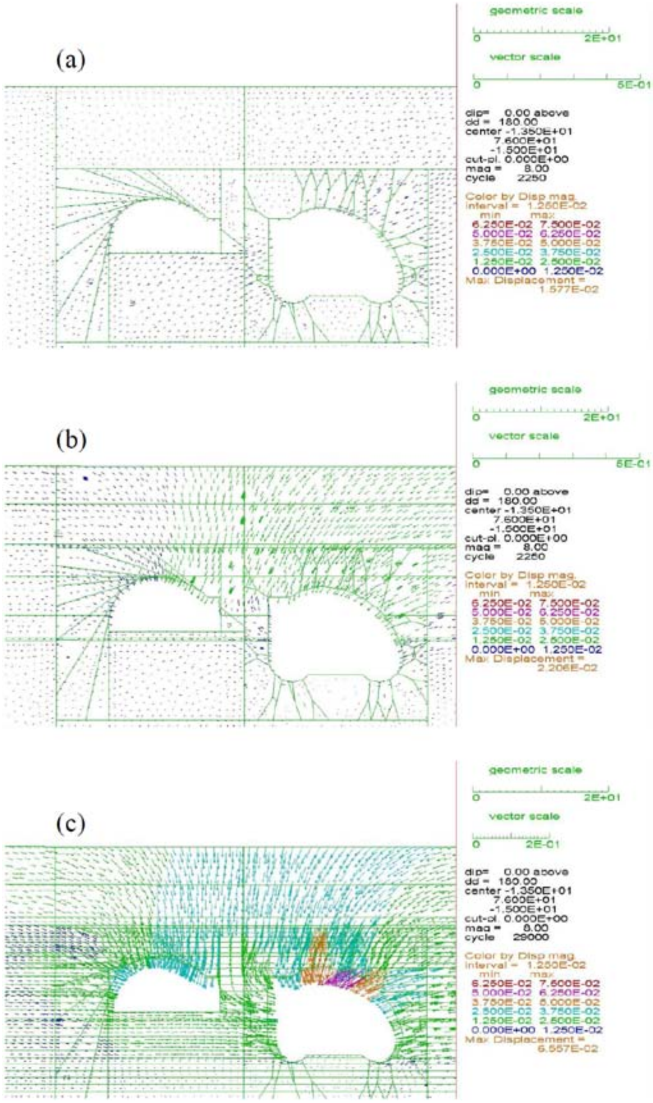

Fig. 3 Total displacements at the middle of pillar with K=4 (a) homogeneous model (b) layered model (c) bedded model.

Table 2. Total displacement for K=1

Analytical model	Total displacement (mm)		
	Cavern centreline	Pillar	Shaft
Homogeneous	9	8	6
Layered	25	22	20
Bedded	43	35	25

Table 3. Total displacement for K=4

Analytical model	Total displacement (mm)		
	Cavern centreline	Pillar	Shaft
Homogeneous	18	16	16
Layered	26	22	85
Bedded	67	66	116

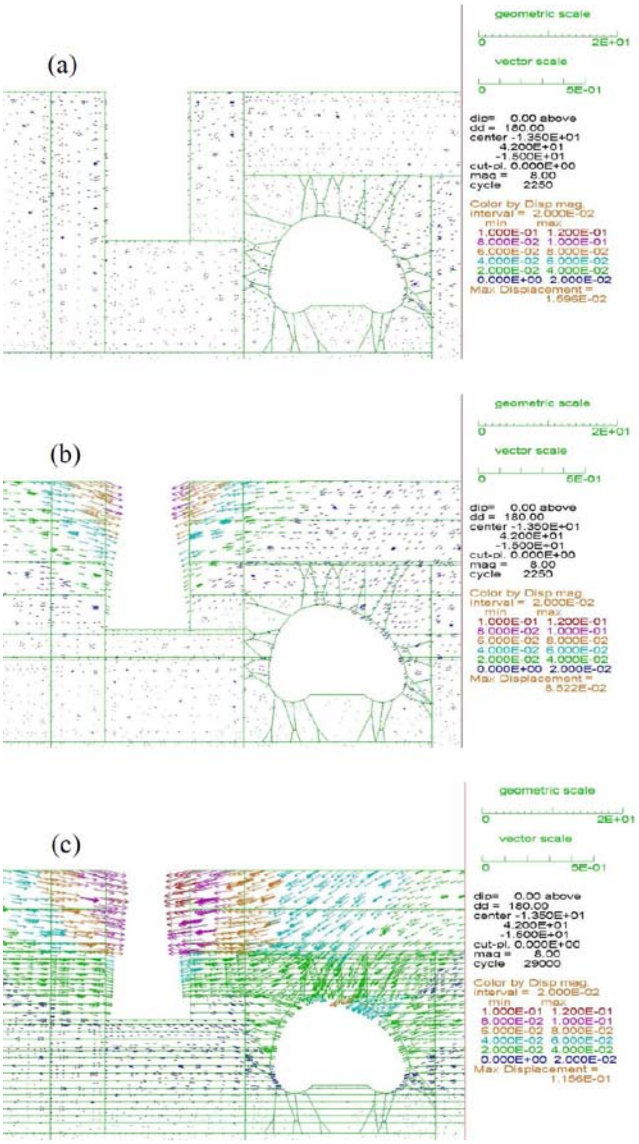

Fig. 4 Total displacements at the escalator shaft with K=4 (a) homogeneous model (b) layered model (c) bedded model.

The cases presented include the homogeneous rock, layered rock and the "bedded" rock models. The two constant K factor (4.0 and 1.0) cases were presented for the various models.

8 DISCUSSION OF MODELLING RESULTS

The presented tunnel modelling results show that there are significant differences in the predicted ground displacements and the behaviour of the excavation.

The homogeneous models under-predicted the total displacements when compared with both the layered models and the bedded models for the cases analysed. Out of the three modelling approaches studied, the bedded models with explicit bedding planes simulated predicted the largest magnitude of displacements.

Moreover, inspection of the bedded model results indicates that noticeable shearing occurred across major bedding planes. The shearing as depicted by differential displacements across bedding planes was particularly prominent where the escalator shaft was close to the cavern excavation as shown in Fig. 4(c).

9 CONCLUSIONS

The modelling of rock mass behaviour is difficult due to the presence of defects within the rock. It is usually not practical to attempt to develop a constitutive model for the rock mass to include the behaviour of the various discontinuities within the rock.

While it is desirable to use simple continuous, homogeneous, isotropic, linearly elastic (CHILE) idealisations to simulate rock masses, the analyses must take cognisance of the fact that natural rocks are typically discontinuous, inhomogeneous, anisotropic and non-linearly elastic (DIANE).

The case study demonstrated the significant differences in the behaviour of rock mass and in predicted displacements induced by the underground excavation among different modelling approaches. In particular, shearing and differential displacements across defects cannot be predicted using simplistic continuum models alone. In that instance, appropriate discontinuum models must be used.

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