The use of instrumentation to back analyze rock mass modulus during a cavern construction in Hong Kong

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ABSTRACT

A Centennial Campus is being developed at the western side of The University of Hong Kong Campus. The scope of this infrastructure project comprises the design, construction and commissioning of two new salt water and two new fresh water service reservoirs. Cavern was excavated in mostly tuff with intercalated tuffaceous sandstone to accommodate the new salt-water reservoir in a twin-cell tunnel system. With the requirements of minimal damage and disturbance to the rock mass during the excavation, it provided an ample opportunity to study the stress induced by the cavern as the excavation approached a virgin zone. This paper presents an evaluation of the magnitude of stresses acting on the crown of the large span tunnel at different stages of ground movement.

Back analysis using two-dimensional finite element analysis was carried out based on the observed stress change and deformation resulted from approximately 6 m high top heading tunnel excavation to assess the deformation modulus of the bedrock and in-situ stress condition.

The observed monitoring records also demonstrated that approximately 2 mm of vertical deformation was mobilized to provide an efficient temporary support to the tunnel crown. The induced stress calculated was in good agreement with those measured if a constant stress field (Ko > 1) was applied where the in-situ stresses were obtained from a nearby hydro-fracturing test.

This paper presents a case study of the rock mass behavior due to excavation of an underground opening.

Keywords: Cavern, stress and movement measurement, back analysis, rock mass modulus, instrumentation

1 INTRODUCTION

A 17 m span rock cavern (see Figure 1 for the site layout plan) is being constructed at the western end of the existing Main Campus of The University of Hong Kong. The cavern is to accommodate two salt-water service reservoirs and it is constructed inside a sloping ground covered with dense vegetation consisting of shrubs and mature trees. Access to the cavern was located between the existing filter building and the future fresh water service reservoir. Starting at the portal, an about 30 m long, 7.2 m span access tunnel was constructed and then separated into two reservoir tunnels. Two 10 m long transition zones were constructed and then the tunnels were enlarged from 7.2 m span to 17 m span to create the cavern for the new salt-water reservoir.

2 BACK ANALYSIS OF ROCK MASS MODULUS

2.1 Site Geology

The site is located to the west of, and outside, the Mid Levels Scheduled Area. The site is underlain by the Mount Davis Formation of the Repulse Bay Volcanic Group, which was intruded by the Kowloon Granite. The volcanic rock consisted of uniform coarse ash crystal tuff with intercalated tuffaceous siltstone and sandstone beds up to 25 m thick. The granite typically consisted of light grey to light pink, equigranular medium-grained biotite granite.

The reservoir tunnels have been located with optimized bedrock cover. The bedrock consisted of strong to extremely strong, slightly metamorphosed coarse ash tuff with dykes of granite and rhyolite. About 80% of the rock excavation was in a steep tabular and blocky rock mass with rough planar, stained joints occurring in three sets, with additional random joints. The remaining 20% had encountered steeply dipping belts of very blocky and highly fractured rock, with the most adverse rock concentrated in seams up to 3 m wide. Potential failure modes included shearing failure of closely jointed rock, and detachment of crown blocks along medium to widely spaced joints.

The characteristics of the rock, the geology encountered and the construction methodology was described in detail by Kwong et al. (2009).

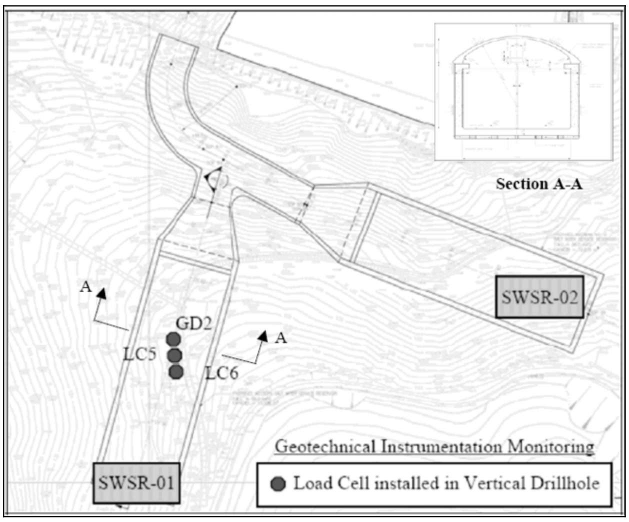

Figure 1: Layout plan of proposed reservoirs

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-1919

2.2 Instrumentation Monitoring

Prior to commencement of the tunnel excavation, strain gauges (Figures 2 and 3 refer) were installed in a pre-drilled hole to monitor the stress behavior of the rock due to the tunnel excavation. The proposed stress monitoring aimed to investigate the ground behaviors resulted from adjacent cavern excavation works and to provide data for further back analysis studies and interpretations.

Three numbers of strain gauge referenced as GD2, LC5 and LC6 (see Figure 1 for locations), were installed at the crown of 17 m span reservoir tunnel. These strain gauges were installed in a vertical pre-drill hole that was drilled from the existing ground level with the depth ranging from 20 m to 35 m (see Table 1 for depth and rock cover). It recorded the pressure changes of rock in vertical direction in a 24 hours real time basis.

Table 1: Schedule of instrument monitoring

Drill hole Refer ence	Chainag e (m)	Offset from Tunne l Centre (m)	Terminate d Level (mPD)	Tunnel Crown Level (m)	Rock Cover below Strain Gauge (m)
GD2	CH 72+00 0	0.0	+103.850	+102.300	1.55
LC5	CH 74+00 0	1.0	+103.730	+102.050	1.68
LC6	CH 76+00 0	2.0	+104.030	+101.700	2.33

2.3 Monitoring Results

The installation of the strain gauge was completed in mid-November 2007. Baseline monitoring was carried out in an extended period. The excavation of the reservoir tunnel passed through the strain gauges in mid-March 2008. Figure 4 shows the measured stress changes in rock (approximately 2.0 m above the tunnel) against time during the 17 m span top heading excavation.

From this figure, it showed that pressure change of rock in vertical direction was about -365 kPa (at the center) above the tunnel crown and the length of the influence zone during which

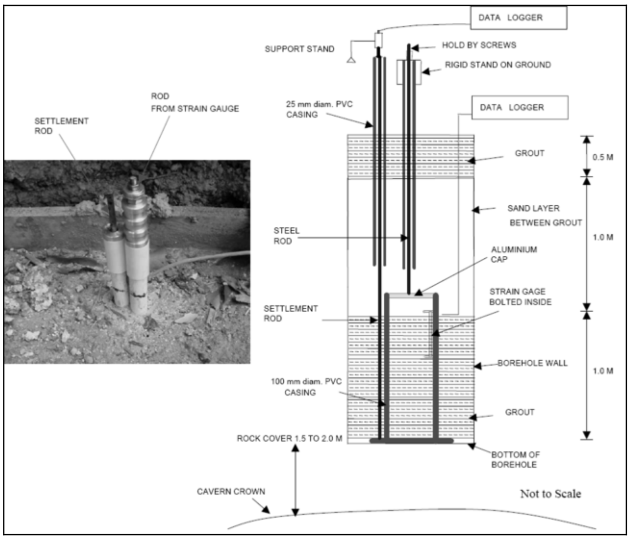

Figure 2: Sectional details of Settlement Rod and Strain Gauge

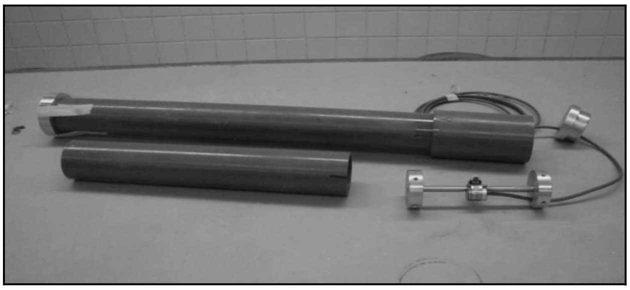

Figure 3: Strain gauges installed inside PVC tube

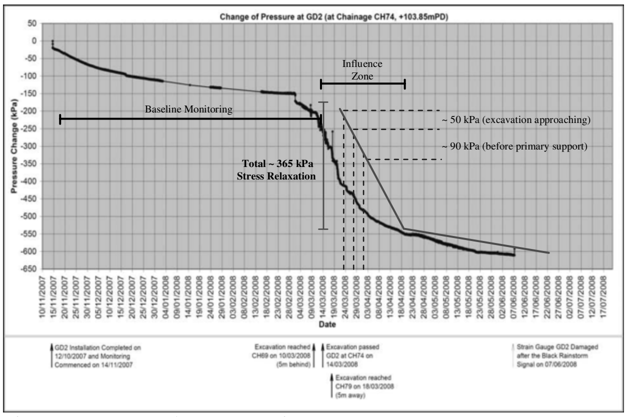

Figure 4: Pressure change at strain gauge GD2

the excavation approached and passed away the strain gauges was ~ 5 m and 10 m respectively. The stress changes in the baseline monitoring period was due to stress relaxation within the connecting rods inside the PVC tube, as evidence showed that their gradients against time was the same after the tunnel passed the influence zones.

It illustrated that the insitu stresses in the rock started dropping (i.e. -50 kPa) when the tunnel excavation was approximately 5 m ahead the strain gauges. Further decrease of rock stresses of approximately -90 kPa was recorded prior to installation of temporary rock supports such as rock bolts and steel fibre shotcrete. After the installation of temporary support, stress relaxation of the rock immediate above the tunnel continued until reaching a state of equilibrium. Owing to the relatively small deformation, the attribution of stress relaxation due to the mobilization of temporary support was very limited. In other words, the strength of the temporary support was not mobilized before reaching equilibrium.

2.4 Computer Model and Design Parameters

Based on the abovementioned monitoring results, a two-dimensional finite element model (Figure 5) was set up to back analyze the appropriate rock design parameters for the HKU reservoir tunnel excavation project. The computer program called "Phase^2 (Roscience 2001)" was adopted in calculating stresses around the 17 m span tunnel.

Based on field mapping (Figure 6), laboratory tests and sensitivity analysis, the rock parameters eventually adopted into the "Phase^2" finite element model are given in Table 2.

The in-situ deformation modulus of this rock mass was ranged between 4.6 GPa (Hoek et al., 2002) and 10.9 GPa (Serafim and Pereira, 1983).

Using the rock mass properties as shown in Table 2, the vertical stress change from the computer program was about -410 kPa (see Figure 7 for stress and Figure 8 for displacement), which agreed very well with that measured from GD2 at the center (-365 kPa in Figure 4).

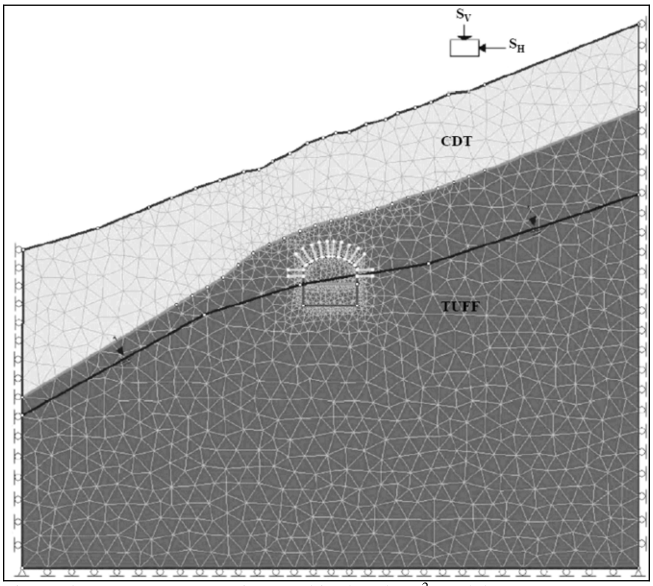

Figure 5: 17 m span reservoir tunnel in the Phase^2 model

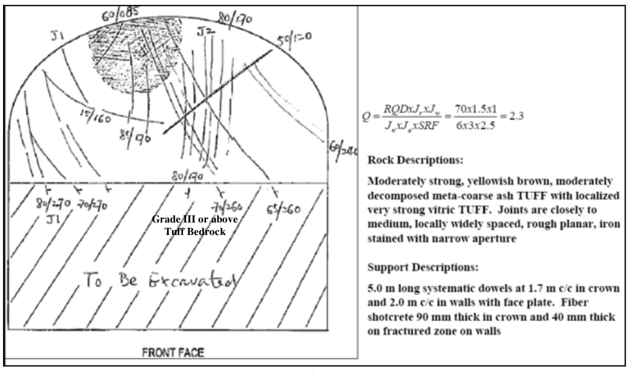

Figure 6: Example of mapped rock face

Table 2: Rock design parameters for finite element analysis

Rock Parameters	Value	Remarks
Unit weight of rock	26 kN/m^3	General parameters
Poisson's ratio	0.25	
Intact compressive strength	50 MPa	Hoek-Brown Criterion 2002
Geological Strength Index, GSI	50	
Hoek-Brown constant, m_i	13	
Distribution factor (D)	0	
Intact rock modulus, E_i	15 GPa	
Deformation modulus of rock	4.6 GPa	
Hoek-Brown constant, m_b	2.18	Input parameters for Phase^2 finite element model (i.e. back-analysis data)
Hoek-Brown constant, s	0.0039	
Hoek-Brown constant, a	0.506	
Mapped Q-value on site	2.3	
RMR value (9lnQ+44)	51	
Deformation modulus of rock	4.6 to 10.9 GPa	

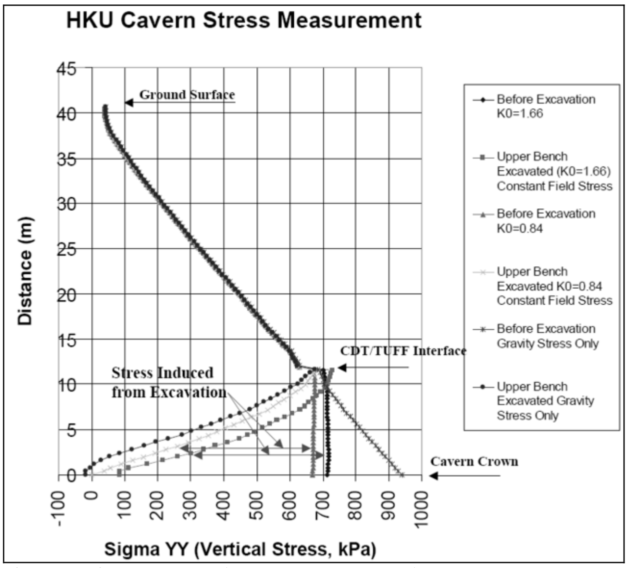

Figure 7: Sigma 1 stress above 17 m span tunnel crown

For sensitivity analysis on the induced stress due to excavation, 3 different cases of in-situ stresses are studied.

Case 1: Applied Constant Stresses,

$$S_H = 2.76\,MPa, S_V = 1.66\,MPa, or\ K_0 = 1.66$$

This was based on recent hydraulic fracture test carried out in the nearby area (MeSy GmbH, 2008).

Case 2: Applied Constant Stresses,

$$S_h = 1.40\,MPa, S_V = 1.66\,MPa, or\ K_0 = 0.84$$

(lower range in MeSy GmbH, 2008)

Case 3: Applied Gravity Stress only

Results of this sensitivity analysis are summarized in Table 3.

Table 3: Comparisons of induced stress and displacement between simulated and measured

Case	Horizontal Stress Applied (MPa)	Vertical Stress Applied (MPa)	K_0	Vertical Stress Induced (kPa)	Vertical Displacement Induced (mm)
1	2.76	1.66	1.66	410	2.6
2	1.40	1.66	0.84	390	3.5
3		Gravity Only	0.33	750	5.3
Measured				365	2 to 3

In addition to the stress monitoring, deformation monitoring point was installed in pre-drill hole to record the rock displacement. Approximately 2 mm downward displacement was recorded due to the reservoir tunnel excavation, as shown in Figure 9.

Using a constant applied stress field of S_H (2.76 MPa) and S_V (1.66 MPa), i.e., K_0 equal to 1.66, which has been measured in the nearby area, the computer program generated an induced stress of 410 kPa and an induced vertical displacement of about 2.6 mm, which agreed very well with that measured from GD2 at the center (measured stress of 365 kPa and settlement of 2 to 3 mm). Based on this agreement, the best back-calculated rock mass modulus ranged between 4.6 to 10.9 GPa.

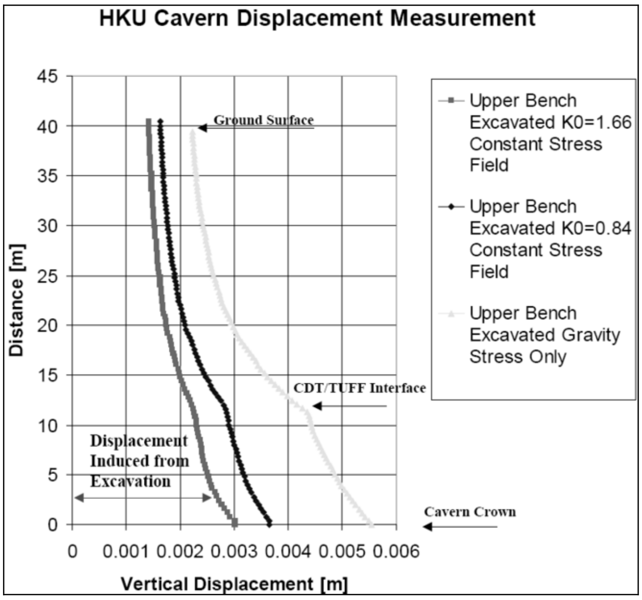

Figure 8: Vertical displacement above tunnel crown

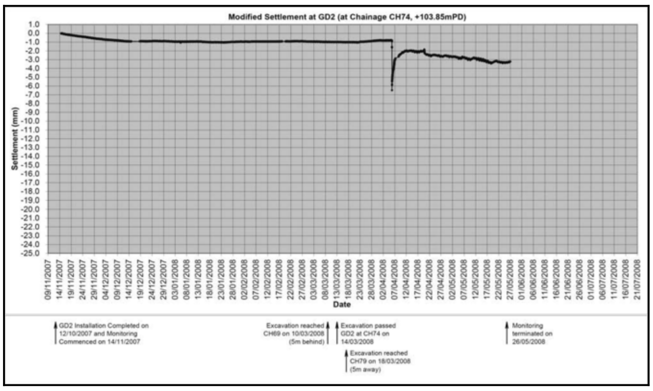

Figure 9: Recorded displacement of monitoring point GD2

3 CONCLUSIONS

In general, the deformation modulus of insitu rock mass is a principal parameter for the numerical analysis in estimating the deformation of rock mass due to the underground openings. In this paper, it presented the rock mass behavior by evaluating the magnitude of stress changes acting on the crown of the large span tunnel at different stages of ground movement with associated temporary supports.

Back analysis was carried out based on the observed stresses and deformation resulted from an approximately 6 m high top heading tunnel excavation. Two-dimensional finite element analysis program, Phase^2, was utilized for this back analysis. Based on the as-mapped Q-value and its corresponding RMR, the rock mass modulus was found to be in the range of 4.6 to 10.9 GPa. With this rock mass properties adopted and with a constant stress field ratio of 1.66 (principle horizontal stress to principle vertical stress), the computer generated induced stress due to excavation agreed very well with that measured. The use of a stress field ratio of 1.66 is based on actual hydro-fracturing test at a nearby site. If gravity stress is used instead of constant stress field is applied, the simulated stress induced is almost 2 times higher than that measured. This is an important finding that the vertical stress change will be less when a cavern under excavation is subjected to high horizontal stress.

In this paper, it aimed, apart from, to present a case-study of the rock behavior due to the underground opening, it also illustrated that monitoring data is very important in understanding rock deformation and its corresponding induced stress for proper evaluation of in-situ stress field and rock mass modulus. With the aid of numerical modeling, it can help to predict the in-situ stress field within an acceptable range.

ACKNOWLEDGEMENTS

This paper is published with the kind supports from the Gammon Construction Limited for the sponsoring of instrument installation. The authors would also acknowledge particularly Mr. T. C. Chan, Mr. Stephen Wan, Mr. Chi Ng for carrying out the fieldwork and data collection.

The views expressed in this paper are those of the authors only and do not represent the views of the WSD of HKSAR.

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