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Support assessment and 3D stress analysis of large underground excavations: A case study using *3DEC*

Rabi Bhusan, Sripad R. Naik & B.H. Vijay Sekar
Numerical Modelling Department, National Institute of Rock Mechanics, Bengaluru, India

1 INTRODUCTION

The Pranahitha-Chevella Sujala Sravanthi Lift Irrigation Scheme (PCSSLIS) consists of excavation of tunnels, caverns and shafts for the purpose of providing irrigation and drinking supply facilities by pumping water from low lying areas to higher elevated areas. This project is being executed by the Government of Telangana, India (NIRM 2014). Excavation of the underground pump house complex (pump house, surge pool, draft tube, vertical shaft and delivery tunnels) lies mainly in granitic rock mass. The dimension of the pump house cavern is 150 m (L) × 19 m (W) × 40.5 m (H) and the surge pool is 350 m (L) × 15 m (W) × 44.33 m (H). Geological data like Q-values, joint set data, rock contact details have been collected from the excavated central gullet of both the caverns. Weathered granite is observed in boreholes up to a depth of 24 m from surface. The surface ground level elevation of the study area is 526 m, the crown of the pump house is 483 m and the bottom level is 442.5 m. The crown level of the surge pool is 493 m and the bottom level is 448.67 m. The detailed rock variants along with major discontinuities have been considered for 3D modelling purpose. Initially support was assessed based on guidelines given by Grimstad & Barton (1993). The same support system was used in the 3D Models. Core samples collected from the exploratory drill hole from the pump house and surge pool area are tested for physico-mechanical properties.

This paper mainly deals with the design of support system and analysis of stress regime distribution during stage wise excavation of the two large parallel caverns and associated components. The global stability of the excavations was evaluated by using Three-Dimensional Distinct Element Code (*3DEC*) (Itasca 2013).

2 DESIGN AND ANALYSIS

A 3D discontinuum model was constructed with the actual geometry of the pump house, surge pool, draft tubes, cable duct and delivery tunnels as shown in Figure 1(A). In the model, the excavation of bench height varies from 6 to 11 m. Excavation was simultaneously carried out in the pump house and surge pool. The underground pump house complex consists of a pump house, surge pool connected to main tunnel of length 9.18 km, vertical shaft having a diameter of 10 m and 60 m depth, four draft tubes and delivery tubes having a diameter of 3.6 m. The primary data required for modelling was collected from the geological mapping, bore hole logs and lab test reports. For the discontinuum modelling, joint sets play a vital role for the block deformations. Based on the central gullet geological mapping data of the pump house and surge pool, it was observed that the granitic rock mass is highly jointed and contains four predominant joint sets (J), dip (D)/ dip direction (DD) J1=80º/340º, J2=70º/154º, J3=80º/080º and J4 82º/264º with spacing varying from 6 to 60 cm. Classification of rock mass was carried out using Tunneling Quality Index (Q) Barton et al (1974) and the values range from 11.81 to 55.21. From drill hole logs indicated highly disintegrated rock mass up to 2.5 m depth, and from 2.5 m to 24 m consisted of weathered granite followed by fresh granite. Physico-mechanical tests were carried out from the samples collected at different depths of boreholes. The uniaxial compressive strength of the intact rock varies from 221 to 246 MPa. The orientation of the long axis of the caverns is N150°.

In-situ stress measurements conducted by NIRM show horizontal stress ratio of 1.84 and direction of maximum principal stress direction as N 150°. The rock mass parameters considered for analysis are estimated based on Hoek et al. (2002) and Hoek & Diederiches (2006) and are given in Table 1. The depth of overburden considered is 30 m for estimating the properties. A blasting zone thickness of 3 m is considered all around the excavations of the pump house and surge pool cavern. A disturbance factor of 0.8 was considered for arriving at the blast zone material properties. Normal and shear stiffness of joints considered for analysis are 100 GPa/m and 75 GPa/m respectively. Cohesion of 8 MPa, friction angle 46° and tensile strength of 8 MPa were considered for joint sets. The cable command available in 3DEC was considered for simulating the 5 m and 6 m rock bolts. 100 mm thick Steel Fibre-Reinforced Shotcrete (SFRS) is modelled as liner elements.

Table 1. Property of rock mass based on Hoek and Brown Criteria.

Hoek and Brown classification – input parameters				
Property	For Fresh Granite		For Weathered Granite	Unit
Uniaxial compressive strength, (lab Value)	217	217	100	MPa
GSI	66	66	66	
m_i	32	32	16	
Disturbance factor	0	0.8	0	
Young's modulus	75.7	75.7	37.5	GPa
Poisson's ratio	0.25	0.25	0.25	
Hoek and Brown failure criterion - parameters				
m_b	9.50	4.23	4.75	
s	0.022	0.0058	0.022	
a	0.50	0.50	0.50	
c	2.53	1.42	1.63	MPa
ϕ	71.3	68	64.11	deg
Rock mass parameters				
Uniaxial compressive strength of rock mass	32.59	16.36	15.02	MPa
Rock mass tensile strength	0.52	0.29	0.48	MPa
Rock mass strength	91.82	60.31	30.85	MPa
Elastic modulus of rock mass	49.44	18.18	24.49	GPa
Bulk modulus	28.15	10.3	14.07	GPa
Shear modulus	20.24	7.44	10.12	GPa
Density	2600	2600	1800	kg/m^3

3 RESULTS AND DISCUSSION

The required supports for surge pool and pump house are estimated using the integrated approach outlined in Indian Standard (IS) code 15026-2002, Barton et al. (1974) and modelling. Accordingly, 25 mm diameter and 5 m long rock bolts @ 2.3 m center to center (c/c) spacing were considered in the crown and 6 m long rock bolts @ 2.5 m c/c spacing were considered in the walls, and 100 mm SFRS for the crown and walls of the surge pool and pump house. This support system was put in the model at each excavation stage. Sections at every 10 m interval were analyzed for the results. Figure 1(B) shows the state of the rock mass, (C) displacement, and (D) minimum principal stress distribution at RD 90 m from pump house and RD 81 m from surge pool. The displacements were concentrated around the openings and the supports were found capable of arresting the movements further away from the cavern rock line. It may be noted from different sections of the model, that crown displacements are negligible and wall displacements in the pump house and surge pool vary from 2 to 3.7 mm. The higher stress concentrations are observed at a few places and are mostly seen in the corners of the excavations. The yield zone of the rock mass is mostly concentrated around 5 to 6 m away from the excavation. There is no breaking of the bond between rock mass and SFRS in the surge pool and pump house. Axial force induced in the rock bolts are analyzed (see Fig. 2(A)) and maximum and minimum membrane stresses (stress parallel to the plane of the liner), are shown in Figure 2(B). At few places in the pump house the axial force has exceeded 0.183 MN (18.66 tons) this may due to excavation of the delivery tunnel in the vertical direction.

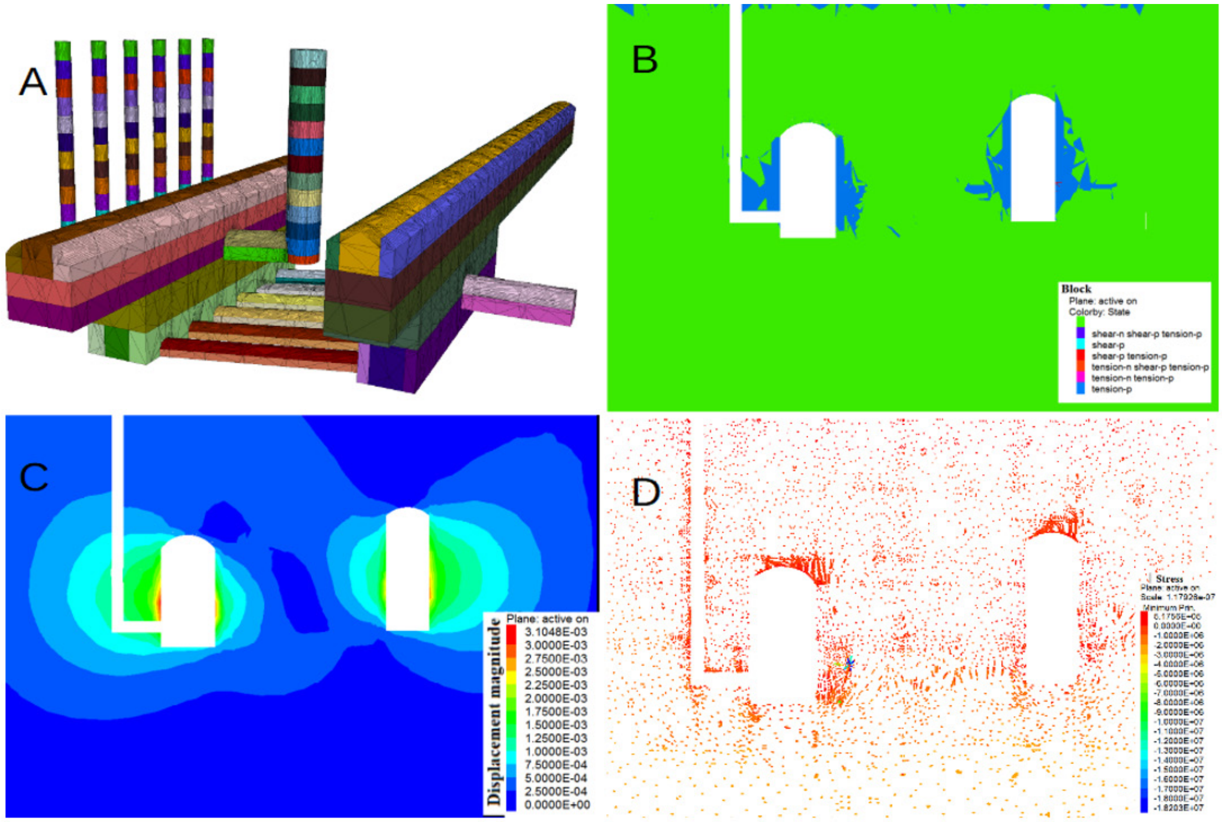

Figure 1. Three-dimensional model. (A) Geometry of pump house complex, (B) State of the rock mass after excavation, (C) Contour displacement after excavation, and (D) Minimum principal stress distribution.

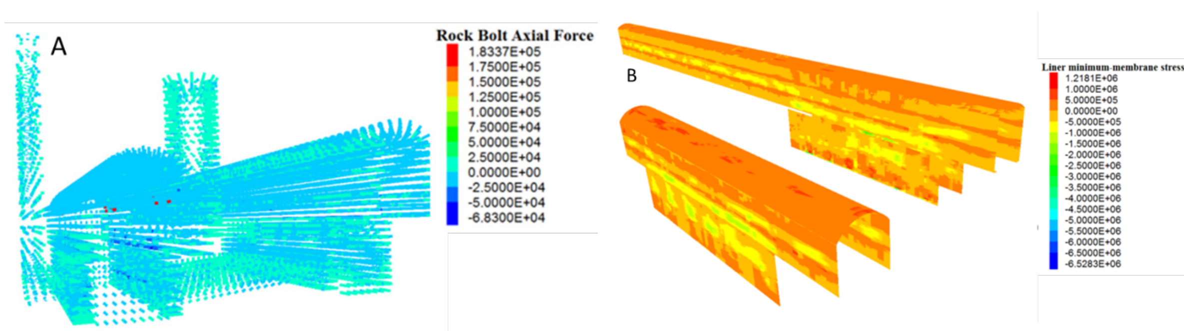

Figure 2. (A) Axial force in rock bolts after complete excavation, and (B) Maximum and minimum membrane stresses (stress parallel to the plane of the liner).

4 CONCLUSIONS

The crown displacements were negligible and wall displacements in the pump house and surge pool vary in the range of 2 to 3.7 mm. The supports were found capable of arresting the movements further away from the cavern rock line. It may be noted that Strength to Stress ratio at most of the places is more than one. The required supports for surge pool and pump house are estimated using empirical approach and their adequacy for the current geological setup was ensured using 3D discontinuum model. The support considered for pump house and surge pool is given below:

- 25 mm diameter 5 m long rock bolt in crown @ 2.3 m c/c spacing
- 25 mm diameter 6 m long rock bolt in walls @ 2.5 m c/c spacing
- 100 mm SFRS to be put in crown as well as in the walls.

Following recommendations are made to improve the stability of the caverns with the support system

- Controlled blasting techniques may be adopted in order to minimize the damage in the immediate vicinity of the excavation.
- Systematic instrumentation needs to be implemented and observed during excavation to reconcile the model results.

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