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Placement and Stress Analysis of Underground Powerhouse Cavern

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Abstract

Planning an underground cavern is mainly influenced by geological investigation of the project area. Stress distribution around the excavation contour plays the vital role for stability of underground cavern. The cavern's overburden is 304 to 330 meters. By dividing the maximum intersection angle between the two main joint directions, the cavern's orientation is achieved. Three joint sets, one foliation plane of 45–55 degree dipping SW, another joint set of 75–85 degree dipping NE, and another joint set of 55–65 degree dipping NE, make up the majority. The cavern's length axis lies between the foilation plane and the joint set dipping 75–85 degrees. Hydrofacturing test and diametrical cone deformation analysis are used to measure stress.10.2 MPa of major horizontal stress and 2.2 MPa of minor horizontal stress are measured dipping in the N21.4W and N68.6E, respectively. The major principal stress makes a 49.4 degree clockwise angle with the cavern's length axis. Then, using empirical techniques and numerical modeling, the redistribution of stress around the excavation's contour is estimated. Since it is not dependent on the size of an underground opening, numerical modeling is more successful than an empirical approach. The deformation monitoring data that MPBX recorded at the project site is used to validate the model.Plastic zone of the crown and wall are 8m and 18m respectively and support provided at the cavern is optimized.

Keywords

Orientation, stress, slate, powerhouse cavern, spalling

1. Introduction

Tunnels and underground caverns are inevitable in most of the hydropower projects of Nepal since they are located in topographically steep areas with a risk of landslide and high tectonic activity. However, the complex geological setup of the Himalayan region and the ongoing tectonic activities have increased geological uncertainties and caused considerable stability problems for tunnels and underground caverns[1]

In the case of hard, massive and joint free-hard rocks an underground powerhouse is recommended. The distribution of strata, lithology, and initial geo-stress should be calculated and tabulated to determine the site condition[1]. The excavation process and corresponding rock mass support measures are based on the discontinuities developed in the rocks and groundwater condition. For large caverns, the deformation and failure characteristics of the surrounding rock mass, the stress characteristics of anchorage structures in the cavern complex, and numerical simulations of surrounding rock mass stability and anchor support performance are the prerequisites for the sound design. In some cases, when there is the problem of rock fall or sliding on the surface of the slope, an underground powerhouse can be safer. Land slide-prone valleys often make a surface station unfeasible hence, an underground cavern should be preferred.

2. Case Study: Tanahu Hydropower Project

Project site is located at 150 km west of Kathmandu and 50km east of Pokhara on Seti river at Rhising Rural municipality of Tanahun District. The head works of the project is located at foothill of Manaug hill top. Powerhouse cavern is located at Patan, Tanahun. Tanahu hydropower project is installed with capacity of 140MW and design discharge of 131.2 m3/sec. Power house is an underground type of length 89 m, wide 22 m and height of 45 m.

2.1 Project area geology

The project area lies in lesser Himalaya region sandwiched between the main boundary thrust (MBT) in south and main central thrust (MCT) in north consisting of low to medium grade metasedimentary rocks such as slate, phyllite, dolomite, marble in addition to meta-sedimentary rocks. Rocks encountered at the project location are phyllitic slate at the powerhouse cavern and dolomite at the dam site. Norpul formation, Dhading dolomite and Benighat slate are distributed in project area. These strata trend E-W to WNW-ESE, namely parallel to the Seti River, and dip southward. Underground powerhouse site was shifted to the river for the purpose of reducing loads, because geological investigation had revealed that the main foundation rocks of the site was not hard dolomite of CH-B class, but moderately hard slate of CM class[2].Geological map of the project area is shown in given figure 1.

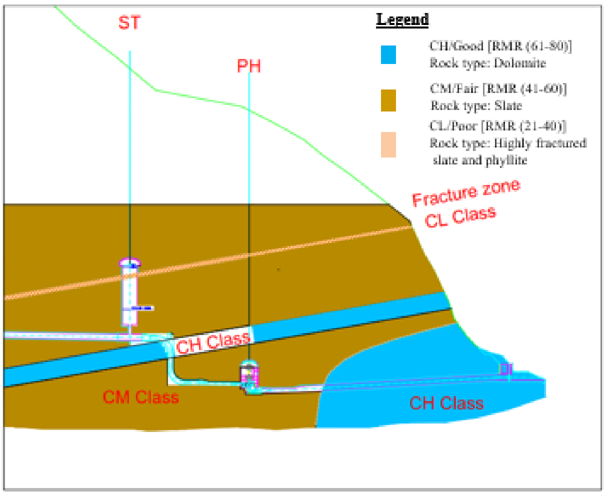

Figure 1: Modified geological map of project area

2.2 Jointing characteristics

Mainly, three joint sets one foliation plane of dip amount 45-55 degree dipping SW, another joint set of dip amount 75-85 degree dipping NE and another joint set of dip amount 55-65 dipping NE. Intercalation of slate and phyllite is found in the powerhouse cavern. Slate rock area has moderately fractured joints with a smooth and undulating surface. Soft and hard infillings were not present but dripping of water was seen.[2]

2.3 Orientation of powerhouse cavern

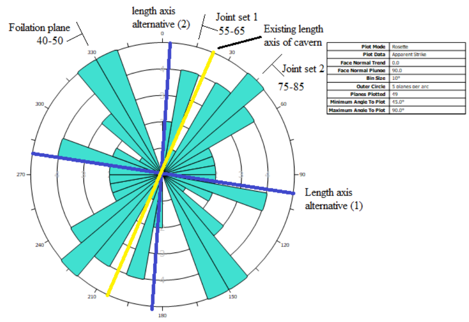

Figure 2: Graphical representation of joint rosette showing length axis of cavern

In figure 2 length axis of alternative 2 is the bisection line of maximum intersection angle between two predominant joint directions. But, according to (Selmer-Olsen and Broch, 1977)[3] most stable orientation is obtained when the length axis of the underground opening makes an angle of 15°-30° to the horizontal projection of major principal stress. From the hydro fracturing test and diametrical cone deformation analysis major principal stress is measured to be 10.4 MPa dipping 21.4° NW. Minor principal stress is measured 2.2 MPa dipping 68.6° NE[2]. Here, the length axis of alternative 2 is about 25° to the horizontal projection of major principal stress.If the direction of principal stress is close to the direction of bedding of foliation planes in highly anisotropic rocks such as crystalline schist and flagstones then length axis of the opening is oriented with an angle relative to the strike of the foliation plane. Length axis of the opening should be oriented with the maximum angle 5° with respect to the strike of the foliation plane and 35°should consider as an absolute minimum. Orientation of cavern is adjusted to 28^0 NE and the maximum angle with respect to strike of foilation plane is 58° which is shown by yellow line figure 1.

2.4 Estimation of redistributed stress and failure extent

Kirsch's equations are applicable to estimate the magnitude of the tangential stresses. The cavern geometry allows the Kirsch's equations only to be applied to the roof, due to its arched shape. Possible tension in the walls are difficult to calculate analytically. Table 1 gives maximum stresses for

Table 1: Estimation of maximum tangential stress by Kirsch's equation

Description	σ_1	σ_3	$\sigma_{\theta max}$	$\sigma_{\theta min}$
PH cavern	10.4	2.2	**29**	**-3.8**

circular tunnels so, for powerhouse cavern Hoek and Brown (1980)[4] is more applicable because it considers shape factor as A and B. So table 2 estimates maximum tangential stress at roof and maximum tangential stress at wall. Now maximum

Table 2: Estimation of maximum tangential roof and wall stress

Description	A	B	K	σ_z	$\sigma_\theta roof$	$\sigma_\theta wall$
PH cavern	4	1.5	0.74	9.8	19.26	7.44

tangential roof stress and maximum tangential wall stress are compared with uniaxial compressive strength of rock and spalling or bursting is predicted and shown in table 3.

Table 3: Determining rock spalling or rock bursting

Description	PH Cavern
σ_1 (Mpa)	10.4
UCS	34.345
σ_1/UCS	0.21
k	0.74
Predicted failure of extent	severe spalling, requiring moderate support

2.5 Model setup and input parameters

Modelling is carried out as plane strain analysis using Gaussian eliminator as solver type. Both elastic and plastic material properties are applied in the analysis.Strength factor of material is analyzed using elastic material.Redistribution stress, displacement and rock mass failure is examined using plastic material.

Model is run in forty stages. Firstly, middle crown is excavated and damaged zone is also described due to blasting effect and smilarly support is also provided sequentially after excavation.

A graded mesh type with 3 nodded triangles has been used in modelling. A gradation factor of 0.1 and a number of excavation nodes equal to 110 has been used. Discretization of the excavation boundary is determined by the number of excavation nodes. Discretization of all other boundaries in the model is determined by a gradient factor in conjunction with a number of excavation nodes.

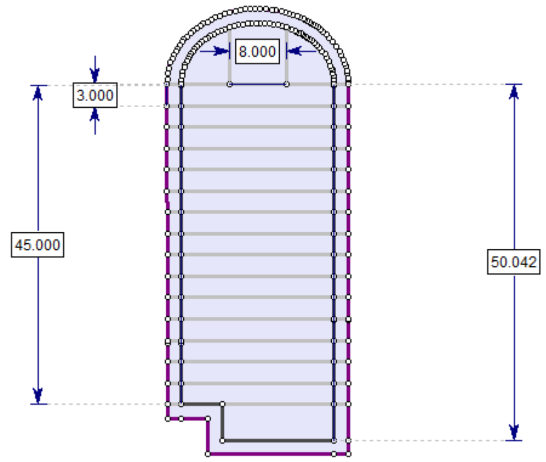

Figure 3: Geometry for numerical modelling

Since Phase2 is a two-dimensional program, projection of horizontal stresses into the relevant cross-section is most important. Equation is derived from the equilibrium state in a two dimensional stress plane.Laboratory data are collected from the report provide by NEA[5].

Table 4: Stress input data

Principal stress	Magnitude	Direction
σ_1(Mpa)	8.23	Vertical
σ_3(Mpa)	6.92	In plane
σ_z(Mpa)	5.67	Out plane

Table 5: Input rock mass properties

Parameters	Value
UCS	34.345 (Mpa)
E modulus	30.5 (Gpa)
Poissons ratio	0.3
GSI	45
Distrubance factor (D)	0.5

Here, three joint sets along with foilation plane is observed. Therefore, according to Hoek et. al, 2002 generalized Hoek and Brown failure criteria is used for further analysis. Two types of bolt are used in the support system. One of them is double corrosion protective bolt and fuly grouted bolt. Bolt properties are shown in table 6. Likewise steel reinforced

Table 6: Rockbolt properties

Description	Values	
2-3	**DCP bolt**	**Fully bonded bolt**
Length (m)	25	8
Spacing (m * m)	3*3	1.5 * 1.5
Diameter (mm)	47	25
Bolt modulus (Mpa)	200000	200000
Tensile capacity (MN)	0.1	0.1
Residual tensile capacity	0.01	0.01
Out of plane spacing	3	1.5

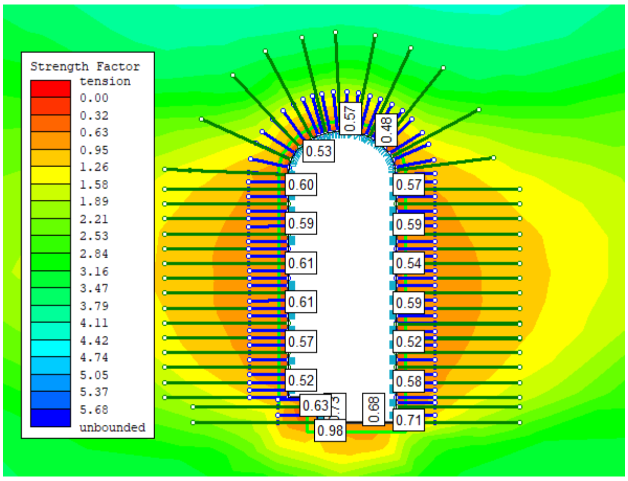

Figure 4: Strength of factor for elastic

Table 7: Shotcrete properties

Properties	Value
Shotcrete modulus	30000
Thickeness	20 (cm)
Poisson's ratio	0.3
Material type	Plastic
Compressive strength	25 (Mpa)
Tensile strength	3 (Mpa)
Beam element formulation	Timoshenko

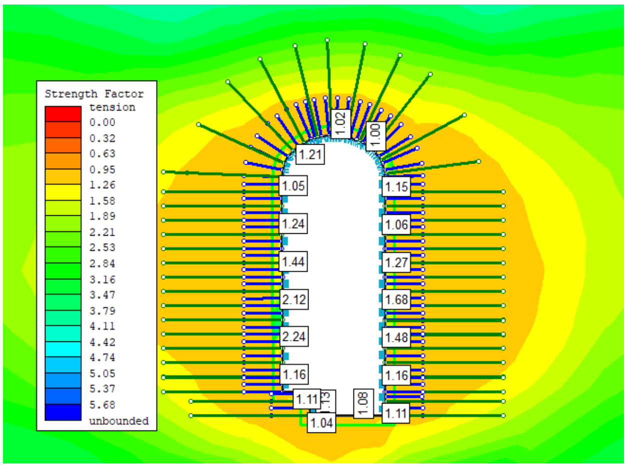

Figure 5: Strength of factor for plastic

shotcrete of strength 25 MPa of 20cm is used. Shotcrete properties is given in table 7. Strength of factor of rockmass is less than 1 in elastic part that means strength of rockmass is less than maximum tangential stress and it fails. So, analysis is done in plastic part for further analysis. In plastic part strength of factor is greater than 1 so strength of rockmass is greater than maximum tangential stress. It is shown in figure 4. Plastic zone of around the excavation is found to be about 18m in wall and 8m in roof from figure 4. Stress distribution around the excavation is shown in figure 5 and figure 6.

Major principal stress is mainly varied with vertical stress in this case. In plane and out plane horizontal stress is less than vetical stress. Maximum total displacement of the rock mass is found to be 0.15m which is shown in figure 7.

Displacement of rock mass is seen to 10 mm from the modelling as in figure 8. From MPBX monitoring data deformation is seen to be 10 mm as shown in figure 9. Thus, this graph validates the model.

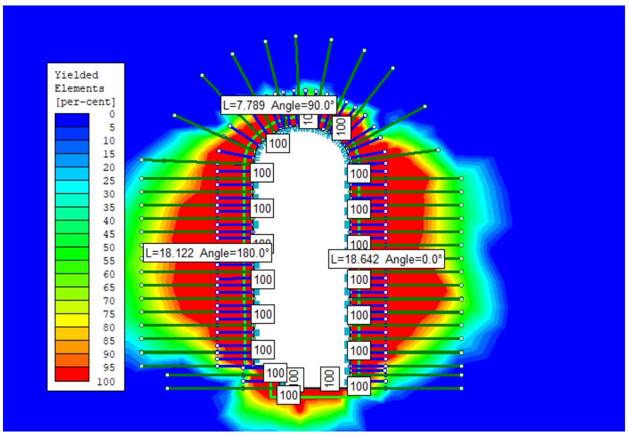

Figure 6: Plastic zone around the excavation

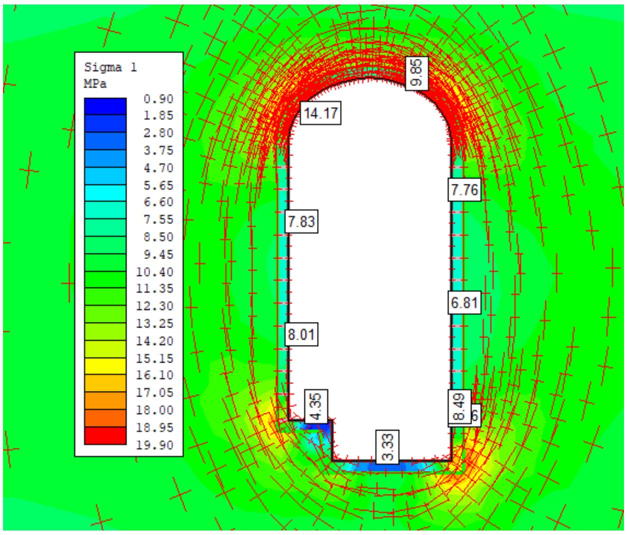

Figure 7: Stress distribution of major principal stress

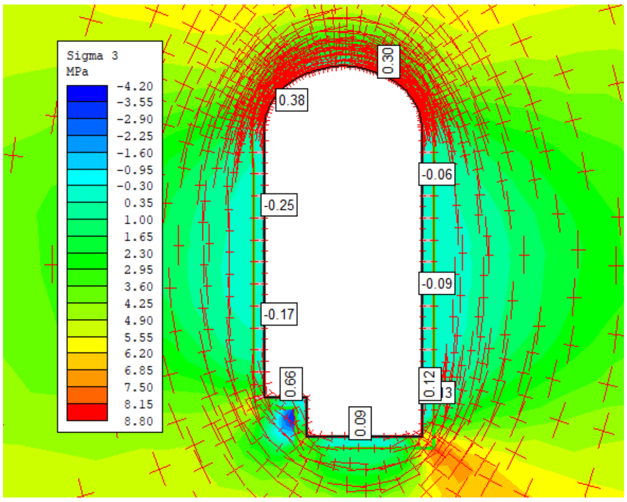

Figure 8: Stress distribution of minor principal stress

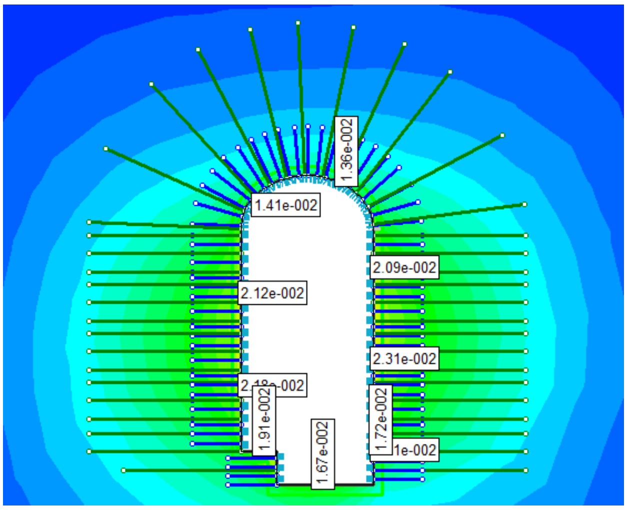

Figure 9: Total displacement of rockmass

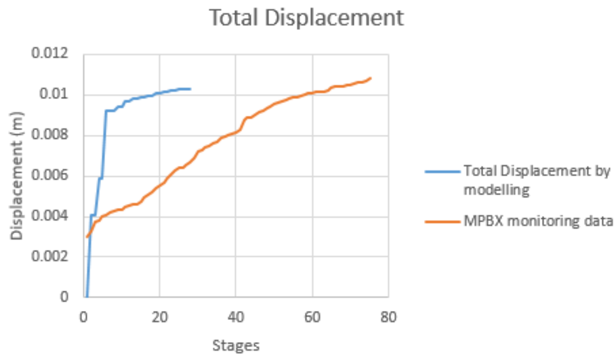

Figure 10: Comparison of deformation data from numerical modeling and MPBX monitoring data

3. Discussion and Conclusion

Placement and orientation of the cavern consideration is important in design phase. Orientation of length axis is 68.6° which is at 58° with the strike of foilation plane. It is more stable in context of stress induced instability. Plastic zone at the crown and wall lies upto 8m and 18m respectively. Due to low plastic radii at crown there will be moderate spalling and support provided is effective for it . According to Hoek and Brown 1980, severe spalling is seen and moderate type of rocksupport is required.Empircal apporach for stress redistribution only examines about major and minor principal stresses and shape of the underground opening. It is independent to size and dimension of the underground opening.So, empirical methods are limited to size but modeling calculates stress redistribution in context to geometry and its opening dimension. Stress around the cavern is mainly characterized by horizontal stress and keeps the cavern roof stable by arching effect. Major and minor principal stress are maximum at crown and minimum at wall. DCP bolt length is greater than plastic zone at indicates support provided to the caven wall and crown is optimized. Here northen and southern wall of the cavern is not optimized in modeling so, 3D modeling is recommended for better analysis.

Acknowledgments

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