Stability issues associated with the construction of underground caverns of Super Dordi Hydropower Project, Nepal

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ABSTRACT: The rock mass is heterogeneous and makes underground space construction a challenge to engineers. The rock formation, mineralogical composition, degree of schistosity, and weathering, are the major factors that determine the stability condition and rock support requirements. This manuscript deals with the stability situation of the underground settling basin caverns of Super Dordi Hydropower Project (SDHP) in Lamjung district of Nepal which is located at the lower boundary of the Higher Himalayan rock formation. The two parallel underground settling basins are 120m long and have an approximate cross-sectional area of 113 sq. m. The manuscript further discusses geological and rock mass quality conditions and evaluates the stability of the underground settling basins using 2D numerical modeling. The outcomes of the analysis presented in the manuscript have been helpful for the optimization of applied rock support.

Keywords: Underground Space, Rock Support, Settling Basin Caverns, Super Dordi Hydropower.

1 INTRODUCTION

Underground caverns are generally used for constructing civil engineering structures, storage purposes (food, nuclear waste, etc.), performing laboratory work, etc. In hydropower engineering, the caverns are mainly used for the construction of powerhouses and settling basins. The challenging factors for the construction of underground caverns are geological investigations, stability analysis, and support design. It should be followed by the excavation method and the redesign of the rock support based on construction observation (Tezuka & Seoka 2003). The major stability problems in underground caverns are block falls, rock bursting, rock spalling, and rock squeezing. The primary factor influencing these stability problems is associated with rock mass quality. The major factors that influence the rock mass quality in the underground opening are rock mass strength, rock deformability, strength anisotropy, discontinuities in the rock mass, and degree of weathering (Panthi 2006). Some of the other stability problems are linked to water inflow and rock swelling. Rock bursts in the underground cavern have strong correlations with rock mass properties, excavation method and speed, and depth where the underground structures are located (Lee et al. 2004). A case study on the large underground powerhouse with a 34m span and 88.7 m height showed that rock fracturing and spalling appeared quickly after the excavation and continued to develop with the advancement of the working face (Liu et al. 2017). Squeezing in the caverns occurs due to weak rock mass, high overburden pressure, and large radius/span of the underground openings (Dwivedi et al. 2014). Providing suitable and optimized support can overcome the squeezing problems in the tunnels (Romero et al. 2007). The large-scale study of the tunnel site including the topography effect, induced stresses, and various other parameters can be done by numerical modeling. Numerical modeling is the tool to study/predict the spalling and rock burst in underground structures (Li et al. 2018, Manouchehrian & Cai 2018).

This paper discusses the challenges faced in the underground settling basins also known as the desander bay of the Super Dordi Hydropower Project (SDHP) located in Lamjung district of Nepal. There are two 120 m long underground settling basins with an approximate cross-section area of 113 sq. m. The favorability of the orientation of the underground settling basins is assessed, and the support provided is numerically analyzed by using the 2D finite element modeling software RS2. The stability is studied based on the deformation results for the support condition provided.

2 UNDERGROUND SETTLING BASIN CAVERNS

SDHP is a run-of-river project with an installed capacity of 54MW. The project has a gross head of 638 m. The project facilitates a diversion weir and intake, desander basins, gravel trap, headrace tunnel, surge shaft, penstock shafts, powerhouse cavern, access tunnels, and tailrace tunnels. All the project structures are underground except the diversion weir and switchyard. The project is in the Higher Himalayas and consists of medium to high-grade metamorphic rocks like schist and gneiss (Peoples Hydropower Company (P) Ltd. 2012). The feasibility of the study showed that there was no severe geological risk associated with the project. The project layout plan of SDHP is shown in Figure 1.

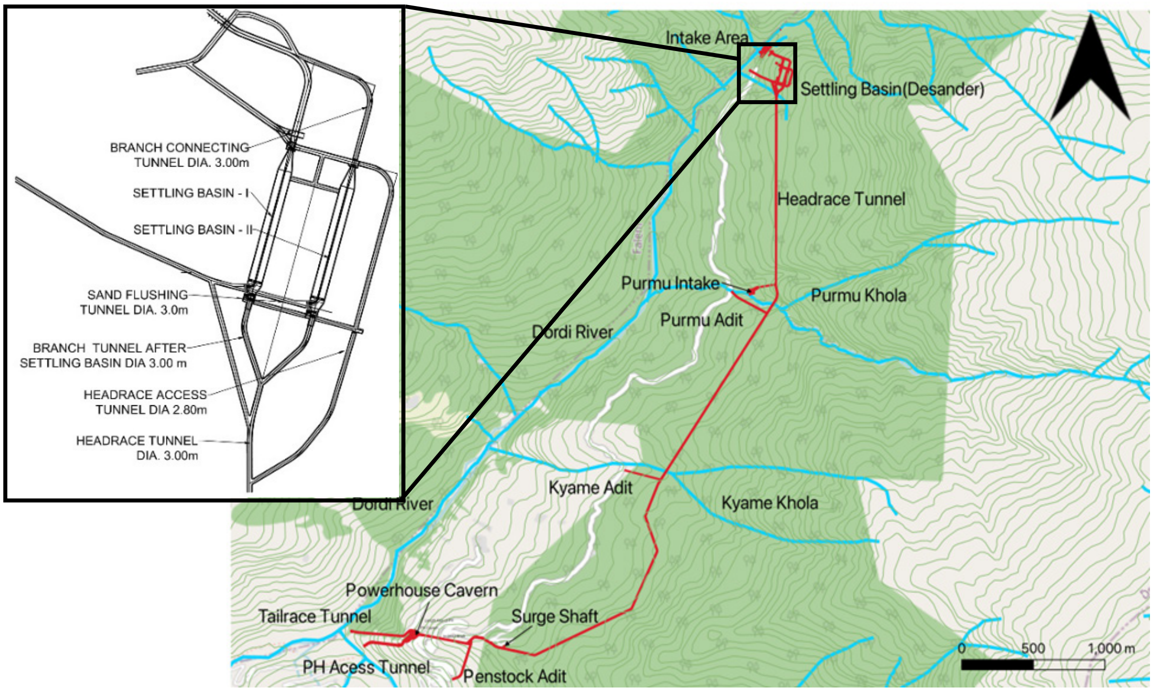

Figure 1. Project layout plan of Super Dordi Hydropower Project.

The settling basins of SDHP are double-chambered underground basins, approximately 120m long, 11 m wide, and 12 m high. The two underground desander basins are separated by a 40m wide rock pillar. Each has a circular crown with vertical walls and an inclined/sloped portion for settling the sediments (Figure 2). The drainage at the bottom has a maximum depth of 1.5m and a slope of 1:100.

The site is visited by the authors and the geometry and support details are provided by Peoples Hydropower Company (P) Ltd. The gneiss rock along the alignment consists of layers of quartz veins. The rock mass quality was assessed by using RMR and Q-system of rock mass classifications (Table 1). Table 1 shows, the rock mass in the caverns mapped as poor-quality rock class.

Table 1. A Typical Rock Mass Quality Classification.

RMR-System (Bieniawski, 1989)		Q-System (Barton et al., 1974)		
Parameters	Value	Parameters	Symbol	Value
- Strength of Intact Rock	9	- Rock Quality Designation	RQD	52
- Rock Quality Designation	13	- Joint Set Number	J_n	6
- Spacing of Discontinuities	5	- Joint Roughness	J_r	2
- Condition of Discontinuities	9	- Joint Alteration	J_a	3
Roughness	*3*	- Joint Water Reduction	J_w	1
Continuity	*2*	- Stress Reduction factor	SRF	2.5 to 5
Joint Wall Separation	*1*			
Joint Wall Weathering	*3*			
- Infilling	1	Q Value: RQD/J_n × J_r/J_a × J_w/SRF = 1.15 to 2.31		
- Groundwater Condition	15	RMR = 50		
- Rating adjustment for joint orientation	-2	GSI = RMR- 5 = 45		

The wall and crown of the cavern were to be provided with 4m and 6m long, 25 mm diameter rock bolts, placed at the spacing of 1.5m alternately staggered position. In addition, 20cm thick steel fiber reinforcement was applied. During excavation, a weakness zone of 3 meters wide was encountered from chainage 0+053 m to 0+057m which led to the modifications of the support system. The modified support system at this location consisted of reinforced ribs, fiber-reinforced shotcrete (RRS), and systematic bolting. The cross-section details of the settling basins are shown in Figure 2.

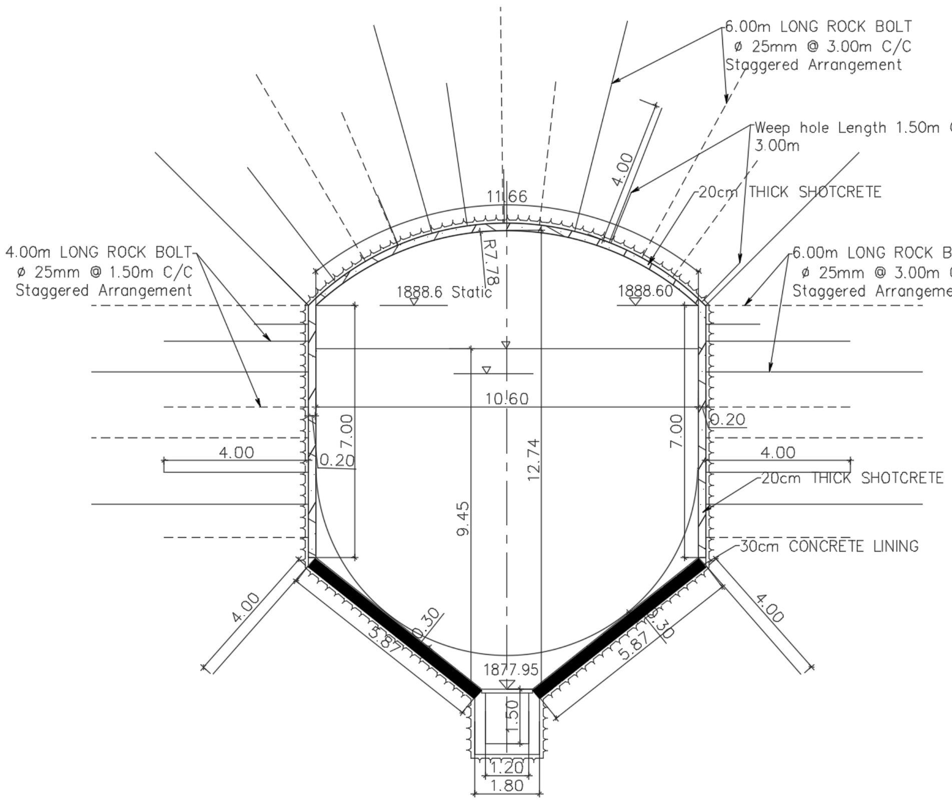

Figure 2. Cross section with support details of the settling basin of SDHP.

The joint measurements from both caverns were made after each round of blasting. Figure 3 shows a joint rosette and length axis alignment of the caverns. As one can see in the Figure, there are two major joint sets and occasional random joints were encountered in both caverns. It is seen that the cavern orientation is relatively favorable in terms of the joint pattern. The cavern is oriented at N15^{0}E where there is no intersection of the major joint sets.

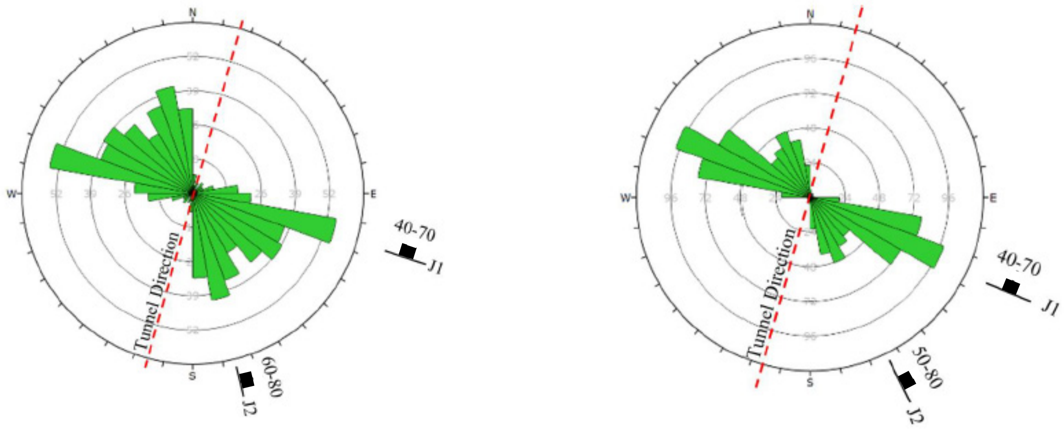

Figure 3. Joint rosette for the two underground caverns.

3 EVALUATION OF INPUT PARAMETERS

The input parameters are collected based on the tests performed at different locations of the project site. The in-situ stress conditions are mainly influenced by the overlying strata, plate tectonics, and stress due to topographic effects. Various stress measurement methods like flat jack test, hydraulic fracturing, 3D-overcoring, etc. are available for estimation of the in-situ stress. In this project, these tests were not performed. Therefore, stresses caused by overburden were calculated analytically and 4.5 MPa tectonic stress (σ_{tec}) is assumed before the analysis. The value is adopted based on the tectonic stress of the headrace tunnel of the Upper Tamakoshi Hydropower Project (UTHP) located in a similar geological formation. Basnet & Panthi (2017) applied major tectonic stress of 15 MPa at zero elevation from the sea level with an orientation N350^{0}E and validated with the measured stress at the powerhouse cavern of UTHP which indicated about 4.5 MPa tectonic stress at a similar depth as these caverns. The caverns are oriented at N15^{0}E which makes an angle of 25 degrees against the direction of tectonic stress. Table 2 summarizes the input parameters used for the modeling of the underground caverns.

Table 2. Input Parameters for the settling basin caverns.

Stress Parameters	**Notation**	**Value**	**Unit**
Overburden	h	185	m
Poisson's Ratio	ν	0.31	--
Tectonic Stress	σ_{tec}	4.5	MPa
Uniaxial Compressive Strength of intact rock	σ_{ci}	39	MPa
Deformation Modulus of intact rock	E_{ci}	19.6	GPa
Density of rock	γ	27	kN/m^3
Support Parameters	**Grade**	**Thickness**	**Remarks**
Shotcrete	M30	20cm	Provided on wall,
Concrete	M30	30 cm	crown, and invert
Rock Bolts (Tensile capacity 0.1 MN)	E = 200 GPa	25 mm dia.	as shown in Fig 2

The vertical stress which is the function of overburden height and density of rock is calculated as 5 MPa and total horizontal stress which is the summation of gravitational stress and tectonic stress is calculated as 6.75 MPa. Equation 1 and Equation 2 show the calculation of the horizontal and vertical stresses. The minor principal stress is in the vertical direction and the major principal stress is in the horizontal direction.

$$\sigma_v = \gamma \times h = 27 \times 185 / 1000 \text{ MPa} = 4.995 \approx 5 \text{ MPa} \quad (1)$$

$$\sigma_h = \sigma_v \times \nu / (1- \nu) + \sigma_{tec} = 2.25 + 4.5 \text{ MPa} = 6.75 \text{ MPa} \quad (2)$$

4 NUMERICAL ANALYSIS

The numerical modeling of the double-chambered settling basin is done using the finite element method using RS2. The continuum model is made where the rock material is considered a single homogeneous mass. External boundaries of the models are placed at a sufficient distance from the caverns and are restrained in both x and y directions. The support conditions listed in Figure 2 are modeled to check the deformation condition in the caverns. The cavern's overburden is 185 m, and both caverns are well inside the mountain from the valley side slope. Thus, the prepared model matches the field conditions even if the variation in the topography is not modeled. Generalized Hoek and Brown (Hoek et al, 2002) failure criteria were considered for the modeling. The other input parameters needed for the numerical modeling are shown in Table 2.

Figure 4 shows the displacement contour of the double-chambered settling basin achieved by numerical modeling in RS2. The red color in the boundary of the figure indicates yielded elements in the cavern walls. It is observed that the displacement is of the same magnitude in both caverns but in a symmetric manner. The deformed shape in the enlarged scale is also shown in the figure. The maximum displacement is 31 mm and is seen in the outer invert of the cavern. However, on the site, no deformation is observed during the visual inspection. The rock bolts with fiber-reinforced shotcrete, placed on the wall and crown are effective in controlling the instability due to potential deformation. Similarly, the 30 cm thick concrete linings in the invert prevented further deformation in the underground structure.

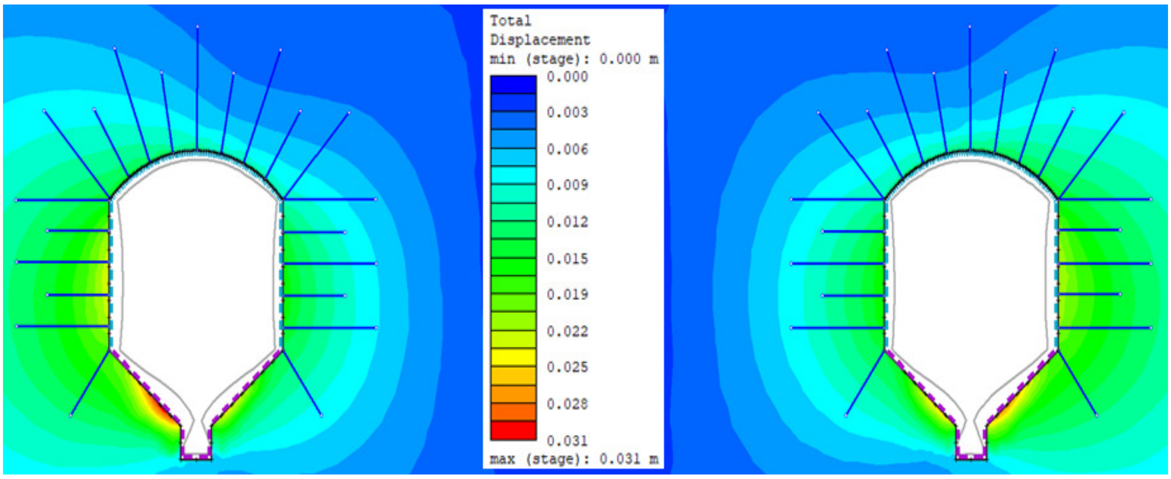

Figure 4. Displacement contour of settling basin in RS2.

A total of 1054 finite elements were yielded in shear and tension, 160 liner elements, and 45 bolt elements were yielded in the plastic analysis of the model. This indicates that the applied support function well. The elements near the cavern border suffered yielding in the shear and tension. The yielding of the elements is mostly on the crown of the cavern, it is because the major principal stress is in the horizontal direction making the tangential stress concentration high in the crown compared to the wall. The strength factor of the cavern revealed values less than 1 in the crown which indicates that the crown is vulnerable to deformation. A similar observation was made in both caverns in the symmetric pattern. The supports (rock bolts and fiber-reinforced shotcrete) helped to stabilize the crown and prevent further deformation into the rock mass. The numerical modeling was effective to understand the deformation and strength factor pattern throughout the boundary of the excavation.

5 CONCLUSIONS

The double-chambered caverns at SDHP are provided with sufficient support to withstand the instability caused by the vertical overburden and horizontal tectonic stresses. The revision made in the rock support system at the weakness zone looks very effective. The use of fiber-reinforced shotcrete with systematic bolting has helped enhance ductility in the applied support in the crown and walls of the two caverns and are assumed to be effective in preventing instability in the underground caverns. The concrete lining in the invert is functioning as support as well as the protective cover to the invert from the sediments that would flow with the water. Due to the higher magnitude of the tectonic stress the major principal stress direction was also oriented in the horizontal direction. This led to a higher concentration of tangential stress in the crown and invert compared to the wall. From the numerical modeling, it is seen that the support provided in the caverns controlled the deformation extent. The orientation of the caverns is fairly suitable in terms of the major joints encountered and horizontal stress orientation.

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