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STABILITY EVALUATION FOR POWERHOUSE CAVERN AT BETAN KARNALI HYDROELECTRIC PROJECT, SURKHET AND ACHHAM DISTRICTS, NEPAL

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Abstract

The planning and evaluation of an underground powerhouse cavern pose significant challenges, primarily due to the geological conditions in the project area. This task requires meticulous consideration of factors such as the cavern's location, orientation, and dimensions. This article specifically focuses on the evaluation of the stability of a large powerhouse cavern of Betan Karnali Hydroelectric Project. The Proposed Powerhouse Cavern having a dimension of 202m length, 23.5m wide and 54.48m high lies in strong shale rock mass. Geological assessment of the powerhouse area was carried out through construction of test adit tunnel in the study area. Combination of geological understanding, laboratory experimentation, established formulas, and simulated stress conditions from valley model informed the selection of the input parameters used in the analysis. Empirical approach: Q-system and Semi-Analytical approaches namely Hoek and Marinos, and Panthi and Shrestha are all used to study plastic deformation. Wedge failure was evaluated using Unwedge and Numerical analysis using RS2 and RS3. The assessment results are analyzed, and particular conclusions are drawn.

Introduction

Nepal has enormous hydropower potential because of its steep topography and perennial rivers that flow from its high, snow-capped mountains. In many Nepalese hydropower projects, the presence of steep terrain and susceptibility to landslides and intense tectonic activity often necessitates the construction of underground structures. In situations where there is a heightened risk of rockfalls or slope instability on the surface, opting for an underground powerhouse becomes a more favorable choice. These areas, characterized by valleys prone to landslides, often make it impractical to build surface powerhouses, making the underground cavern option the preferred and more viable solution. During underground excavations, accuracy should be maintained from the start of geological investigation, as the results of the investigation plays a crucial role in selection of the Cavern alignment. Considerable discrepancies have been found between expected and actual rock mass characteristics, resulting in severe cost and schedule overrun for most of the tunneling projects [1]. Nonetheless, past instances in Nepal have shown that the geological assessments during the planning phase for underground operations often suffer from insufficient quality [2].

Project Description

The Betan Karnali Hydroelectric Project is located along the lower limb of the Karnali River, in the mountainous region of Achham and Surkhet District. The PROR project site is near Betan, within Chaukunne rural Municipality of Surkhet District and lies between 81°11'43" E to 81° 24'42" E longitude and 28°50'57" N to 28°56'04" N latitude. The ongoing project under development is designed to handle a flow rate of 536 m3/s and possesses a gross head of approximately 100 m, resulting in an installed capacity of 439 MW.

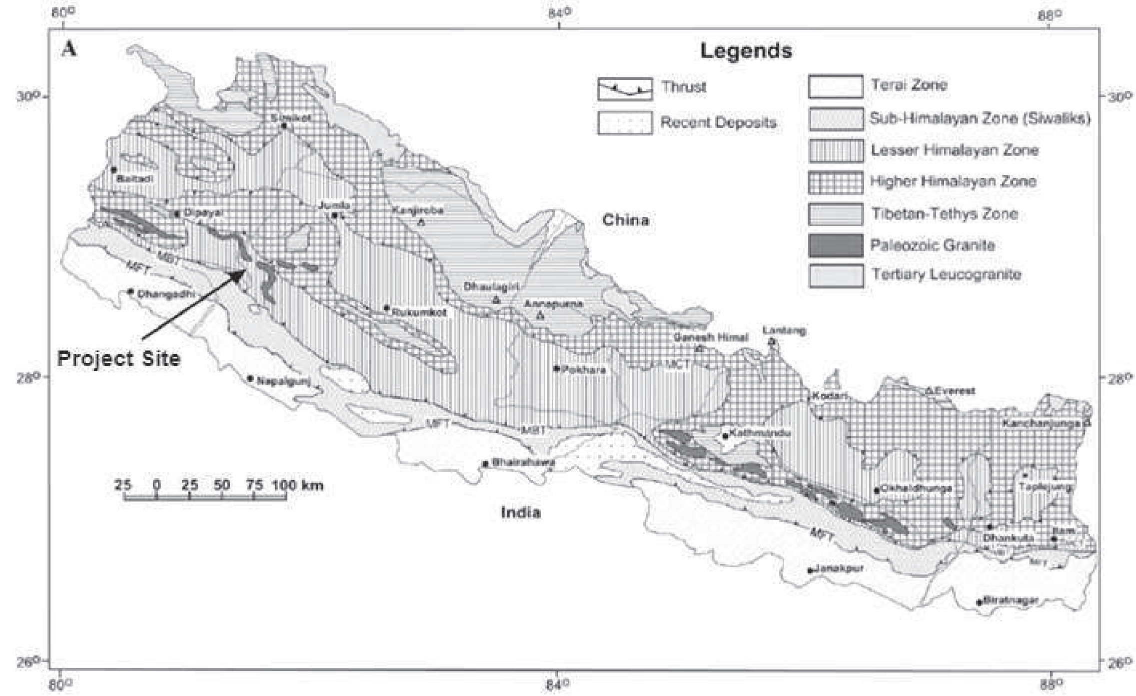

Figure 0.1: Geological map of Nepal showing Project Location

1.1 Project Geology

The rock encountered in the alignment of the test adit consists of Dolomite, Slate, and a thin band of sandstone. Dolomite rock mass with 3+1 random joint sets is encountered around the tunnel portal up to 86 meters chainage. From 86m to 94m chainage, the inter bedding of dolomite and slate along with quartz vein can be observed. After around 95m chainage, dominant black slate along with quartz vein is observed until 101m. From 195m to 203m, inter-bedding of calcareous sandstone and slate is observed. After 203m to 338.02m, major rock type observed is slate. The properties of shale rock dominantly present in the powerhouse cavern area are examined and assessed for evaluation purposes.

1.2 Orientation of Powerhouse Cavern

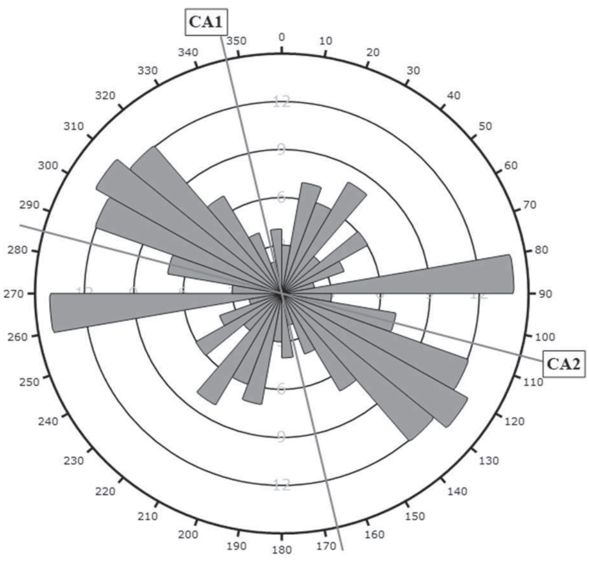

Figure 0.2: Joint Rosette showing Length axis of Cavern

The attitude measurement from the face mapping of the test adit was plotted in a joint rosette to ascertain the orientation of cavern. For a shallow seated cavern, the bisector of the bigger intersection angle between the two predominant joint directions is the suitable alignment of the cavern length axis. CA1 represents the best alignment for the powerhouse cavern and CA2 represents the alternative alignment.

Establishment of Input Parameters

1.3 Intact Rock Properties

The laboratory test of intact rock core samples obtained from test adit was done. The UCS, Elastic modulus, Poisson's ratio and tensile strength of the rock cores was tested. The average value of different parameters obtained from the laboratory test results was used to estimate different rock mass properties. The result of the laboratory test is included in Table 0-1.

Table 0-1: Laboratory Test Data of BKHEP

S.N.	UCS, MPa	Ei, GPa	vi	σ_t, MPa
1	104.94	56.4	0.35	5.79
2	53.15	63.5	0.19	14.8
3	105.51	45.8	0.35	13.16
4	70.77	53.6	0.34	16.86
5	71.86	71.1	0.2	15.78
6	74.14	36.9	0.31	
7	71.18	56.4	0.36	
8	120.67			
9	155.07			
10	132.51			
11	100.94			
12	34.96			
N	**12**	**7**	**7**	**5**
Mean	**91.31**	**54.81**	**0.30**	**13.28**
SD	**34.69**	**11.18**	**0.07**	**4.40**

1.4 Estimation of Rock Mass Properties

The mechanical characteristics of a rock mass are closely linked to its strength and ability to deform, and these factors are crucial when it comes to simulating underground chambers or caverns. Aside from that, the stress conditions, existence of any weakness and shear zone, and 3D topography in the project region are critical for cavern modeling.

Elastic parameters: The value of rock mass deformation modulus (Ecm) was calculated using the empirical formula proposed by [4] which is suitable for numerical modelling. The disturbance factor was taken as D = 0.5 based on the chart by [3] in the disturbed zone near the excavation boundary. The radius of the disturbed zone due to blast damage is considered to be 2m.

$$E_{cm} = E_{ci} \times \left(0.02 + \frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{60+15D-GSI}{11}\right)}} \right)$$

Using above equation, the modulus of deformation was found to be E_{cm}(undisturbed) = 12258.3 MPa and E_{cm}(disturbed) = 5803.5 MPa.

Rock Mass Characterization: The classification of the rock mass and survey of joint patterns were conducted within the test adit section and results pertaining to the highest overburden was then utilized for subsequent analysis and evaluation.

Residual GSI Value: Because the mechanical parameters σ_{ci} and mi do not change when the rock is fractured, only the volume of the block and the roughness conditions of joints change. The residual Hoek Brown constants for the rock mass can be calculated from a residual GSI(R) value using the same formulae as for peak strength parameters, according to [5]. In order to simulate a strain softening model, a GSI(R) value of 20 was used for the residual condition.

Hoek-Brown parameters: Hoek-Brown constants mb, s and a were calculated using the following equations:

$$m_b = m_i exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{-\frac{20}{3}}\right)$$

Where D is the factor that depends upon the degree of disturbance due to blast damage and stress relaxation [3]. The value of disturbance factor (D) is taken as 0.5 on the basis of the chart proposed by [3] as it was observed that the gneiss found in the cavern was of good quality and had moderate effects on surroundings. The value for mi for shale was taken as 6. GSI of 45 was found appropriate based on the GSI characterization chart and correlation with Q-value parameters. The values of Hoek and Brown parameters obtained are:

Table 0-2: Hoek and Brown Parameters of Study Section

Parameters	Undisturbed Zone	Disturbed Zone
GSI	45	45
mb	0.842	0.437
s	0.00022181	0.0006534
a	0.5081	0.5081

1.5 Evaluation of Rock Stresses

In-situ stresses in rock mass are the result of overlying strata, plate tectonics, and stresses due to topographic effects. Generally, in-situ stress is measured using methods like hydraulic fracturing, and 3D over coring. Since we don’t have measured data for the selected site, we investigate reviewing similar nature projects. In the case of the Tanahun Hydropower Project, the hydro fracturing and diametrical core deformation analysis method concluded tectonic stress of 8.2 MPa with direction N10° E. Because the project location is oriented similarly to Tanahun Hydropower Project and lies in Lesser Himalayan Zone, we are adopting a similar value for tectonic stress. Three-dimensional stress measurement is still necessary to confirm and is presented as a recommendation.

The gravity of earth results in two components of the gravitational stresses i.e., horizontal and vertical components. When the surface is horizontal, the vertical gravitational stress at a depth of z is:

$$\sigma_{\mathrm{v}} = \gamma z$$

In an elastic rock mass with a Poisson's ration of ν, the horizontal stress induced by gravity is:

$$\sigma_{\mathrm{h}} = \frac{\nu}{1-\nu}\,\gamma z$$

Because of tectonic stress at shallow and moderate depths, the total horizontal stress is often higher than the horizontal stress induced by gravity alone. According to [6], the magnitude of total horizontal stress can be calculated by:

$$\sigma_{\mathrm{H}} = \frac{\nu}{1-\nu}\,\gamma z + \sigma_{\mathrm{tec}}$$

Where, σ_v and σ_h are the vertical and horizontal stresses in MPa, σ_{tec} is the tectonic stresses due to plate tectonic movement, γ is the specific weight of rock mass in MN/m^3, and z is overburden depth in meters.

Since RS2 is a two-dimensional program, the horizontal stresses must be projected into the relevant cross-section for the model. This can be done from equations derived from an equilibrium state in a two-dimensional stress plane [7].

$$\sigma_{\alpha} = \sigma_{\mathrm{H}} cos^2\alpha + \sigma_{\mathrm{h}} sin^2\alpha$$

$$\sigma'_{\alpha} = \sigma_{\mathrm{h}} cos^2\alpha + \sigma_{\mathrm{H}} sin^2\alpha$$

Where, σ_α and σ'_α are in-plane and out-plane horizontal stresses and α is the angle between tunnel axis and minimum horizontal stress.

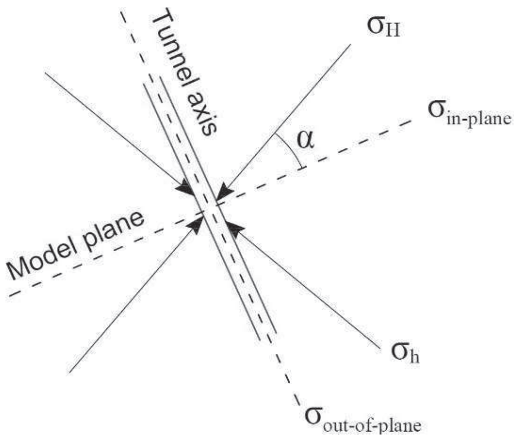

Figure 0.1: Illustration of the use of equations

The value of stress calculated for the numerical model is given in the Table.

Table 0-3: In-situ Stress Calculations

Input Parameters			
Overburden	h	217.62	m
Poisson's Ratio	ν	0.3	
Tectonic Stress	σtec	8.2	MPa
Trend of Tectonic Stress	θtec	N10°E	
Trend of Cavern	θc	N344°E	
Angle between Tectonic Stress and Cavern Length Axis	α_t	26°	
Density of Rock	λ	0.027	kN/m^3
Due to Gravity			
Vertical Stress	σ_v	5.88	MPa
Horizontal Stress	σ_h	2.52	MPa
Total Horizontal Stress	σ_H	10.72	MPa
Horizontal Stress			
In-Plane	σα	9.14	MPa
Out of Plane	σα′	4.10	MPa
In-Plane Stress Ratio	K	1.55	
Out Plane Stress Ratio	k	0.696	

Stability Assessment of Powerhouse Cavern

Stability assessment is done by different methods depending upon the type of failure. There are two distinct types of failure that occur in the roof and wall of the underground cavern; i.e., structurally controlled instability and stress induced instability. To carry out stability assessment Empirical methods, Semi-Analytical methods and Numerical Modelling was carried out for the powerhouse cavern.

1.6 Empirical Methods

Support chart given in Q-system is used to preliminarily define the rock support needed for the powerhouse cavern.

Table 0-1: Recommended Support from the Q-system

Description	Span/ ESR	Correction for Wall Support	Support System
Roof	23.50	-	6 m bolts, 1.7 m c/c, E=700J Shotcrete: 12 cm
Wall	54.48	2.5	12 m bolts, 2 m c/c, E=800J Shotcrete: 12 cm

1.7 Prediction of Failure mode

σ_1 [MPa]	UCS [MPa]	σ_1 / UCS	RMR
6.14	91.31	0.0672	50

Corelating the values in Table given by [8], falling or sliding of blocks and wedges will occur in the powerhouse cavern.

1.8 Semi-Analytical Methods

The analysis of the plastic deformation was done through semi-empirical methods viz. (Hoek & Marinos, 2000) and (Panthi & Shrestha, 2018). Table summarizes the squeezing predictions which shows that there is no squeezing but might have some support problems.

Table 0-2: Squeezing Prediction using Semi-empirical methods

Hoek & Marinos (2000)		Panthi & Shrestha (2018)			
Strain % without support	Squeezing Condition	ε_{ic} %	ε_{fc} %	Strain %	Squeezing Condition
0.0944	Few support Problems	0.4889	0.9241	< 2	Few support Problems

1.9 Numerical Modelling

In the analysis and detailed modelling of the Betan Karnali HEP powerhouse cavern, the numerical methods such as Unwedge and RS-2D software have been used.

1.9.1 Unwedge Analysis

The required jointing parameters for structurally controlled instability analysis are presented in Figure. The longitudinal axis of the cavern is orientated at N344°E. Considering uncertainty related to joint orientation, engineering geological properties and cohesion, probabilistic Unwedge analysis is performed. The potentials result on the most critical wedges are based on the finding from maximum support pressure required, maximum wedge depth, and minimum factor of safety and probability of failure.

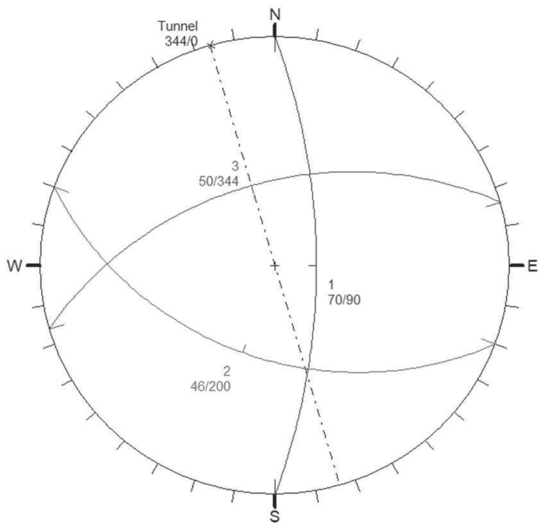

Figure 0.1: Stereographic projection of jointing conditions with cavern alignment

Unwedge result shows maximum required support pressure of 0.059 MPa, maximum wedge depth of 3.84m and wedge weight in the roof ranges from between 0.011 and 2.035 MN. Figure shows minimum factor of safety for the wedge failure on the roof of the cavern.

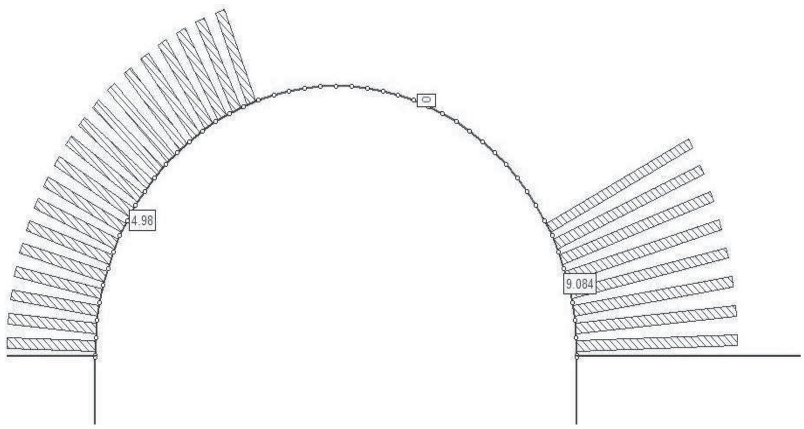

Figure 0.2: Results of Unwedge on minimum factor of safety at each segment

Figure shows the probability of failure for each segment. The probability of failure is the ratio of the number of failed wedges to the number of samples.

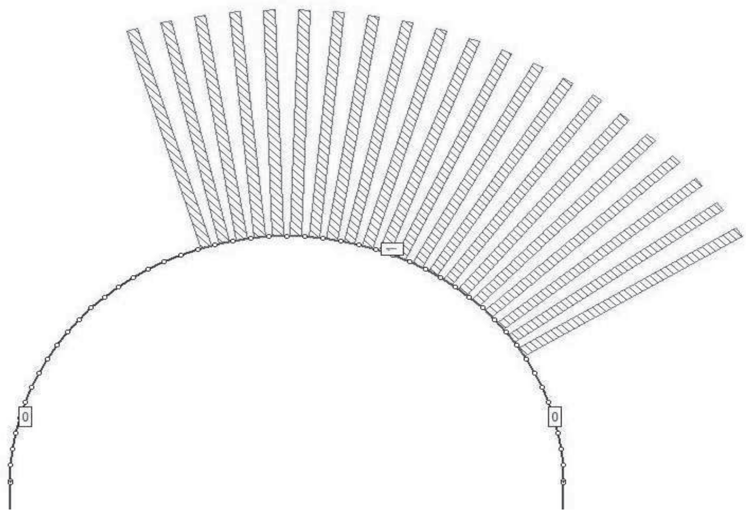

Figure 0.3: Results of Unwedge on Probability of failure for each segment

Generally, in the design of an underground support system, a combined method of rock bolt and shotcrete is applied. 6m long grouted dowel at 1.5m * 2m spacing along with a thin layer (5cm) of Shotcrete has been applied as a support system in the roof of the cavern. The application of rock bolts and shotcrete increased the factor of safety of the most critical wedges and decreased the probability of failure to zero, which confirms the effectiveness of applied support.

1.9.2 RS-2D Analysis

In the numerical modelling, a finite element software package was used. RS-2D [9] represents two-dimensional FEM program used for rock engineering application where multi stage and complex models can be created and analyzed quickly. RS2 has been used to model and analyze the stability of the underground powerhouse cavern. The numerical modeling in RS2 is done as a plane strain analysis with Gaussian elimination as the solver type.

1.9.2.1 Valley Model

A 2D topographical model is used to address the analytical problem. One cross-section is employed, which is perpendicular to the length axis of the powerhouse cavern. The topographical profiles are imported from the hydropower project's working drawings, and the topographical model of the cross-section is presented in Figure. The model's bottom border is constrained in both the X and Y directions, the sides in just the X direction, and the top surface is free to move in both directions. Further gravity-type field stress was adopted, with real ground surface being used because the model profile has varied elevation. The material is assumed to be elastic in order to examine the stresses in the rock mass. As a result, the strains can develop without the material failing. The unit weight of the shale is taken as 27 KN/m^3. The modeling is done as a planar strain analysis with a Gaussian eliminator as a solver type. Table summarizes the in-situ stress ratio (both in and out of plane) utilized in the model.

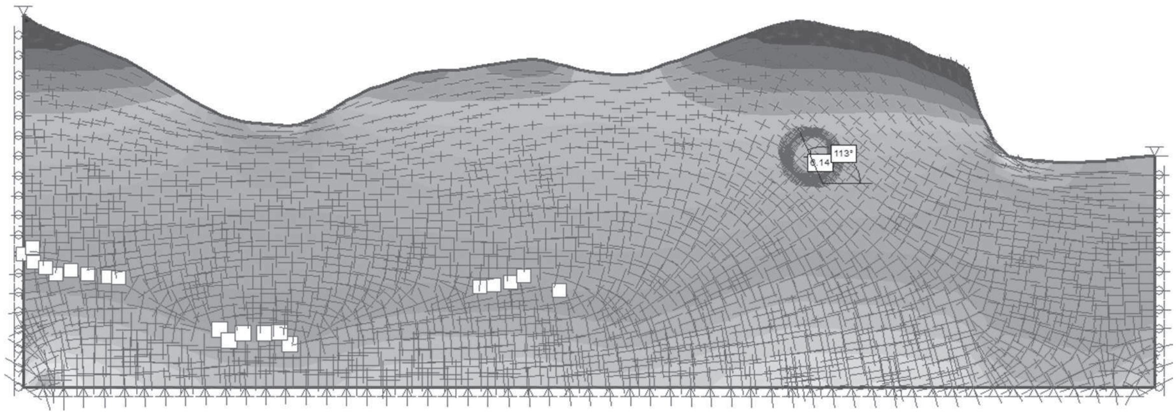

Figure 0.4: Simulation of stress conditions in the valley model (σ1).

Table 0-3: Stress Analysis Results from Valley Model

σ_1 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_1 angle from horizontal (°)
6.14	4.07	4.61	113

Table shows the results from the valley model, which contains maximum, minimum and intermediate horizontal stresses and directions stresses at the point of the underground powerhouse cavern.

1.9.2.2 Powerhouse Cavern Model

Model Setup in RS2

The vertical cross section of the cavern is a slight simplification of the original cross-sectional geometry excluding the busbar tunnels and draft tube in order to ease the modelling. Large scale caverns will normally be excavated in several stages. As the scope of this task is the overall stability, the number of excavation stages are reduced pursuant to the original excavation plan. Number and order of model stages are illustrated in Figure 0.5. The external boundary is rectangular box with an expansion factor of 4, which is considered sufficient in order to avoid end effects. Figure 0.5 also shows the cavern of 54.48m ×23.5m cross-section modelled in RS2 as described above. The obtained stress in Table was used in this model. Because the primary purpose of this model is to design a thin (typically 0.2 m) shotcrete lining in a relatively large cavern, it is necessary to select a mesh that allows vertices to be spaced at approximately half the thickness of the shotcrete lining as widely spaced vertices in beam elements used to model the shotcrete lining will have poorly distributed forces [10]. According to [10], a six-nodded triangular element mesh produces good results, but the vertex spacing on the excavation border must be precisely defined. A disturbance zone of radius 2m having decreased strength was also included in the model.

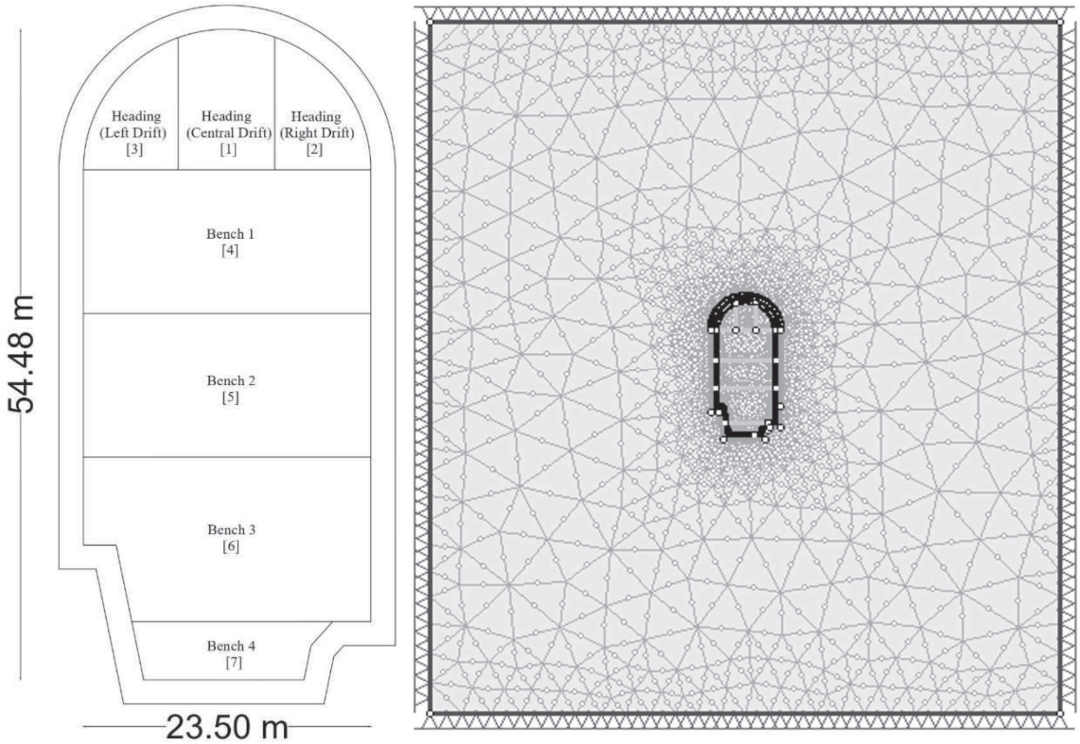

Figure 0.5: Excavation stages of Powerhouse Cavern (Left) and Model of powerhouse cavern profile and excavation stages in RS2 (Right)

Modeling and Support Considerations

Rock support should be planned so that a stable condition is attained during every stage of excavation as well as the last stage. Ground relaxation was modeled in the numerical model by applying a uniformly distributed load on the excavation profile at each stage of excavation. Before installing the support system, 50% ground relaxation was allowed in the analysis. Understanding the stages of excavation, the behavior of rock mass displacement, and the interaction of support measures is critical for developing an acceptable support system design. Early bolt application, according to the analysis, would build a sturdy foundation and prevent rock blocks from falling loose. Designing the shotcrete liner presents greater complexity due to various practical factors that need to be taken into account. Given the displacement characteristics of the rock mass, it's not advisable to apply a single, thick shotcrete layer at an early stage. A "composite element" is used to account for the time effect on rock mass relaxation and the stage of application of various layers of shotcrete. Shotcrete has several layers and can be applied at different delaying times with this element. The adopted support procedure for analyses in RS2 is shown in Table.

Table 0-4: Excavation Stage and Rock Support procedure

Excavation Stage	Parts	Stress Relaxation	Rock Bolt		Shotcrete	
			6 m	12 m	100 mm	100 mm
1	No Excavation					
2	[1]	50%				
3		50%	[1]		[1]	
4		100%				
5	[2]	50%				
6		50%	[2]		[2]	
7		100%				
8	[3]	50%				
9		50%	[3]		[3]	
10		100%				
11		100%				[1,2,3]
12	[4]	50%				
13		50%		[4]	[4]	
14		100%				
15	[5]	50%				
16		50%		[5]	[5]	
17		100%				
18		100%				[4,5]
19	[6]	50%				
20		50%		[6]	[6]	
21		100%				
22	[7]	50%				
23		50%		[7]	[7]	
24		100%				
25		100%				[6,7]

1.9.3 RS3 Analysis

RS3 is an additional software tool developed by Rocscience, offering the capability to perform three-dimensional finite element analyses for various applications in civil engineering and mining. In RS3, modeling is executed through an uncoupled analysis approach utilizing an automatic solver type.

1.9.3.1 Powerhouse Cavern Model

The powerhouse cavern cross-section in RS3 has been made less complex, featuring fewer excavation stages compared to the RS2 model, as illustrated in. For the RS3 analysis, the input data remains the same, except for modifications made to the mesh setup and displacement properties. In RS3, a graded mesh type is implemented, specifically utilizing a 4-noded tetrahedron with gradation parameters. These parameters include an offset of 2, a gradation factor of 0.1, and an external gradation factor of 1.

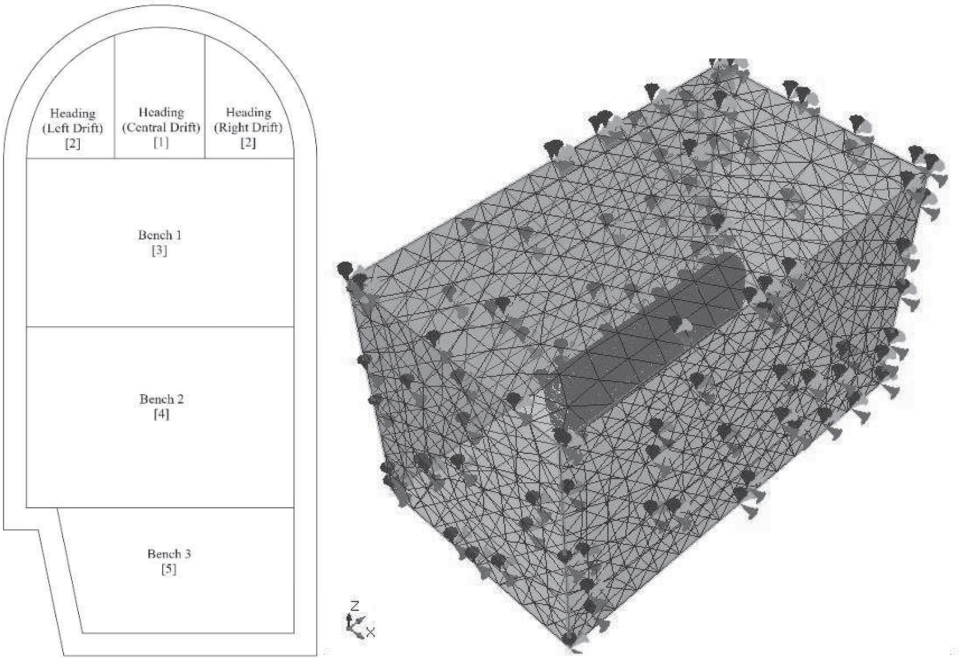

Figure 0.6: Excavation stages of Powerhouse Cavern (Left) and Meshing in Model geometry in RS3 (Right)

The properties of the support system, which include elements like rock bolts and shotcrete, remain consistent between the RS3 and RS2 analyses. This consistency also extends to the spacing and application procedure of these support elements.

Results and Discussions

The Figure shows the displacement observed in the powerhouse cavern with the applied support. The roof is able to remain largely stable thanks to the bolt system and first layer of shotcrete. However, there are a few minuscule locations where the model's line elements show signs of yielding by coloring them red. As a result, both in reality and in the model, the second layer of shotcrete must be applied before the next benching. In the final stage, when the final benching and supporting work is completed, the support for the cavern is system bolting of 6 m long, 2m spacing and 12 m long, 2m spacing with two layers of shotcrete applied, initially with a thickness of 50 mm, followed by a final layer with a thickness of 300 mm. A maximum displacement of 70mm in the walls was observed in the RS2 model whereas displacement reduced to 50mm in the RS3 model. In the RS3 analysis, the areas where elements have undergone yielding are more restricted and less critical when compared to the RS2 analysis results.

Table 0-5: Comparison between RS2 and RS3 Analysis Results

Particulars		RS2	RS3
Displacement (mm)	Before Support	95	66
	After Support	70	50
Percentage of Yielded Elements	Before Support	85.08%	60.16%
	After Support	82.00%	57.98%
Yielded Bolts per m	After Support	89	12
Percentage of Yielded Liners	After Support	4.5%	3.36%

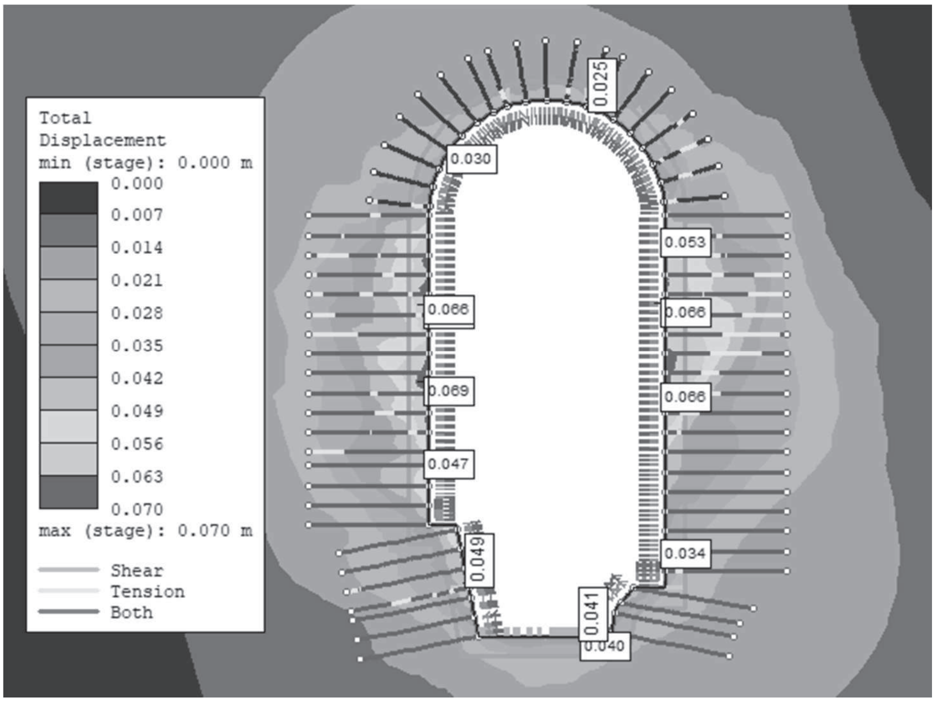

Figure 0.7: Total displacement and yielded elements at the final stage of RS2 analysis.

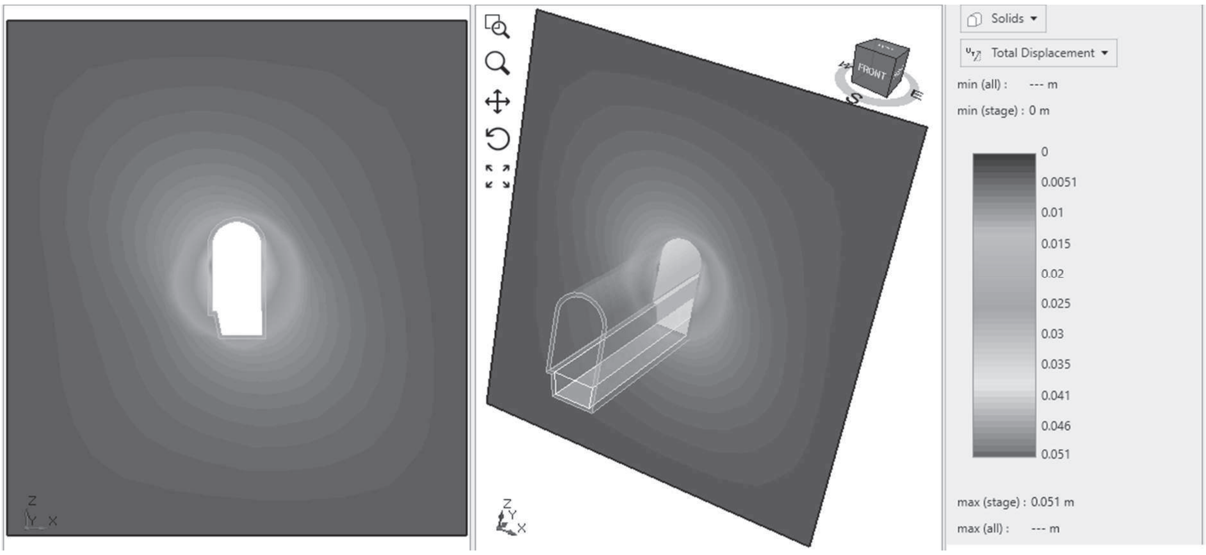

Figure 0.8: Total displacement and yielded elements at the final stage of RS3 analysis

Conclusion

The assessment of the orientation and stability of the powerhouse cavern in the Betan Karnali Hydroelectric project involved a comprehensive analysis that combined empirical, semi-analytical, and numerical methods. To ensure the stability of the cavern, empirical and semi-analytical techniques were initially employed to estimate the necessary rock support. This preliminary information served as a crucial input for the subsequent numerical analysis which takes into account a wide array of factors, including in-situ stress conditions, mechanical properties of the rock, and properties of the support materials, all within a unified framework. It was evident from the analysis that precise settings within numerical software programs are essential, as well as accurate geometric data pertaining to the underground cavity. A key takeaway from this assessment is the recognition that relying solely on one method when designing an underground cavern is not advisable. Instead, it is highly recommended to adopt multiple approaches, as exemplified in this study, which combines various methodologies to ensure a robust and reliable evaluation of cavern stability.

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