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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 underground powerhouse cavern of Betan Karnali Hydroelectric Project. The Proposed Powerhouse Cavern having a dimension of 202m length, 23.5m wide and 54.48 m 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. The input parameters used in the analysis were determined by considering several sources of information and data. Combination of geological understanding, laboratory experimentation, established formulas, and simulated stress conditions from valley model informed the selection of these input parameters. Empirical approaches: 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 which showed deformation resulting up to 70 mm. The assessment results are analyzed, and particular conclusions are drawn.

Keywords
Powerhouse cavern, Test Adit, Numerical model

1. 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. Severe differences have been discovered between expected and actual rock mass characteristics, resulting in severe cost and schedule overruns for the majority of 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].

2. Project Description

The Betan Karnali Hydroelectric Project is situated in the hilly terrain of Achham and Surkhet Districts, specifically on the lower stretch of the Karnali River. 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.

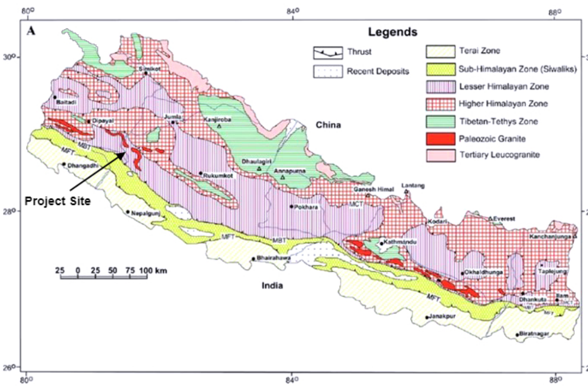

Figure 1: Geological map of Nepal showing Project Location (modified after Dahal 2006)

2.1 Project Geology

The rock encountered in the alignment of the test adit tunnel 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.

2.2 Orientation of Powerhouse Cavern

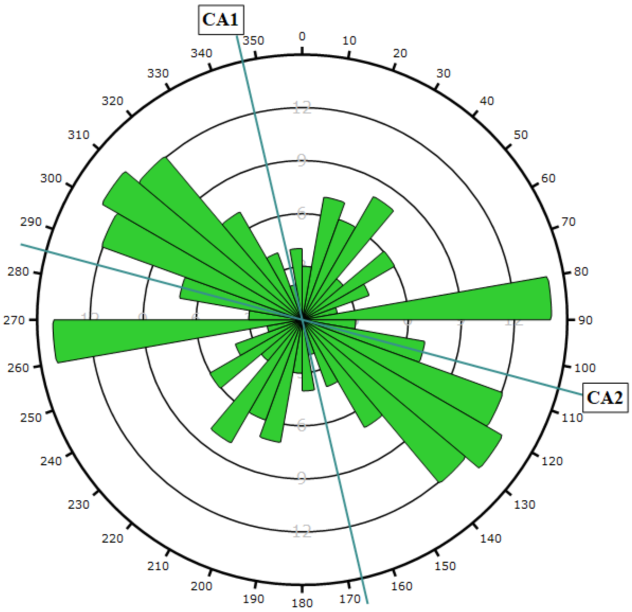

Figure 2: Joint Rosette showing Length axis of Cavern

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. In figure 2, CA1 represents the best alignment for the powerhouse cavern and CA2 represents the alternative alignment.

3. Determination of Input Parameters

3.1 Intact Rock Properties

Intact piece of rock in its natural, undisturbed state is called intact rock. A piece of drilled core is an example of intact rock. Its properties are determined by carrying out laboratory testing. The laboratory test of the number of intact rock core samples (N) obtained from test adit tunnel was done. The UCS, Elastic modulus, Poisson's ratio and tensile strength of the rock cores was tested. The mean value of several characteristics acquired from laboratory test results was used to determine the qualities of various rock masses. The result of the laboratory test is included in Table 1.

3.2 Rock Mass Properties

The term "rock mass" refers to the natural material found in its original location, comprising both the solid, unbroken rock and any fractures and discontinuities within it. 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. Additionally, it is essential to consider factors such as the prevailing stress conditions, the presence of any structural weaknesses or shear zones, and the three-dimensional topography of the project area when undertaking cavern modeling.

Table 1: Laboratory Test Data of Rock Samples of BKHEP

S.N.	UCS, MPa	Ei, GPa	v	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	0.20	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
S.D.	34.69	11.18	0.07	4.40

Elastic parameters The deformation modulus of the rock mass, denoted as Ecm, was determined using formula by [3], which is well-suited for numerical simulations. To account for the effects of disturbance, a disturbance factor, represented as D, was set at 0.5. This choice was informed by [4], specifically in relation to the blast-damaged zone in close proximity to the excavation boundary. It's important to note that this blast damage zone is estimated to extend approximately 2 meters around the cavern.

$$E_{c_m} = E_{c_i} * (0.02 + \frac{1 - D/2}{1 + e^{(}60 + 15D - GSI)/11}) \quad (1)$$

The deformation modulus was calculated using the aforementioned equation for the damaged zone and entire rock mass. Ecm(undamaged) = 12258.3 MPa and Ecm(damaged) = 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 The peak Geological Strength Index (GSI) value is adjusted by considering the reduction in the two primary influencing factors within the GSI system, namely, the remaining block volume and the remaining joint condition factor. This adjustment results in the residual GSI (r) value. Only the volume of the block and the roughness conditions of joints change when the rock is broken since the mechanical parameters $\sigma_(ci)$ and m_i do not change. The residual Hoek Brown constants for the rock mass can be computed based on the residual GSI(r) value using the same equations employed for determining peak strength parameters, as outlined by [5]. In this context, a GSI(r) value of 20 has been selected to represent the residual condition, which is essential for creating a strain softening model.

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

$$m_b = m_i \times \exp(\frac{GSI - 100}{28 - 14D}) \quad (2)$$

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

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \tag{4}$$

Where D is the factor that depends upon the degree of disturbance to which the rock has been subjected to blast damage and stress relaxation [4]. The disturbance factor (D) has been set at 0.5, guided by [4]. Additionally, a value of 6 is assigned for the mi parameter, specifically for shale rock. To determine the appropriate GSI value, the GSI characterization chart and its correlation with Q-value parameters were consulted, resulting in a GSI value of 45 being deemed appropriate. Subsequently, the Hoek and Brown parameters for the study section are derived as follows:

Table 2: Values of Hoek and Brown Parameters

Parameters	Undamaged Zone	Damaged Zone
GSI	45	45
mb	0.842	0.437
s	0.00022181	0.0006534
a	0.5081	0.5081

3.3 Evaluation of Rock Stresses

In-situ stresses in rock mass are caused by overlaying layers, plate tectonics, and topographic impacts. To evaluate the in-situ stress conditions, commonly employed techniques include methods such as hydraulic fracturing and 3D over coring. We examine analyzing similar nature projects because we lack measured data for the selected location. 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.

The Earth's gravitational force gives rise to two distinct types of stress components: horizontal and vertical. In situations where the surface is horizontal, the vertical gravitational stress at a depth of z can be expressed as follows:

$$\sigma_v = \gamma * z \tag{5}$$

In an elastic rock mass with a Poisson's ratio denoted as ν, the horizontal stress caused by the influence of gravity is given by:

$$\sigma_h = \nu/(1-\nu)\gamma * z \tag{6}$$

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 total horizontal stress can be determined by:

$$\sigma_H = \nu/(1-\nu)\gamma * z + \sigma_{tec} \tag{7}$$

Where, σv and σh are the vertical and horizontal stresses respectively in MPa, σtec is the tectonic stress, γ is the specific weight of rock mass in MN/m^3, and z is overburden depth in meters.

Since RS2 is a software program designed for two-dimensional analysis, it's necessary to project the horizontal stresses onto the relevant cross-section within the model. This can be achieved by using equations that are derived from an equilibrium state in a two-dimensional stress plane [7].

$$\sigma_\alpha = \sigma_H cos^2\alpha + \sigma_h sin^2\alpha \tag{8}$$

$$\sigma'_\alpha = \sigma_h cos^2\alpha + \sigma_H sin^2\alpha \tag{9}$$

In this context, $\sigma\alpha$ represents the in-plane horizontal stress, while $\sigma'\alpha$ stands for the out-plane horizontal stress, both measured in MPa. The angle α is the angle between the axis of length of the cavern and the direction of the tectonic stress.

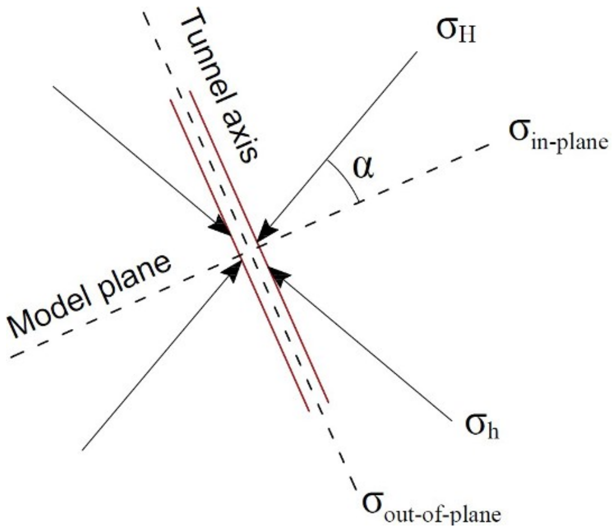

Figure 3: Illustration of the use of equations 8 and 9

Table 3 shows the in-situ stress values evaluated which are used in the 2D-numerical model.

Table 3: Determination of In-situ Stress

Input Parameters			
Overburden	h	217.62	m
Poisson's Ratio	ν	0.30	
Tectonic Stress	σ_{tec}	8.2	MPa
Trend of Tectonic Stress	θ_{tec}	N10E	
Trend of Cavern	θ_c	N344E	
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	σ_α	9.14	MPa
Out of plane	σ'_α	4.10	MPa
In-plane Stress ratio	K	1.55	
Out of plane Stress ratio	k	0.696	

4. Stability Assessment of Powerhouse Cavern

Depending on the type of failure, multiple methods are used to assess the stability of cavern. In the context of underground caverns, there are two distinct types of failures that can occur in both the roof and walls: structurally controlled failure and stress-induced failure. To conduct stability evaluation for the powerhouse cavern, empirical approaches, semi-analytical methods, and numerical modeling were used.

4.1 Empirical Methods

The Q-system support chart is used for determining the preliminary rock support necessary for the powerhouse cavern.

Table 4: Q-system Recommended Support for Roof and Wall

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

4.2 Prediction of Failure mode

Table 5: Failure Mode Prediction

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

Correlating the values in Table 5 given by [8], falling or sliding of blocks and wedges will occur in the powerhouse cavern.

4.3 Semi-Analytical Methods

The plastic deformation was analyzed using semi-analytical methods namely (Hoek & Marinos, 2000) and (Panthi & Shrestha, 2018). Table 6 highlights the squeezing predictions, indicating that there is no squeezing however there could be some support issues.

Table 6: Squeezing Prediction using Semi-Analytical methods

Hoek & Marinos (2000)		Panthi & Shrestha (2018)			
Strain % without support	Squeezing Condition	ϵ_{ic}%	ϵ_{fc}%	Strain%	Squeezing Condition
0.0944	Few support problems	0.489	0.924	< 2	Few support problems

4.4 Numerical Modelling

For the numerical modelling of the powerhouse cavern, the numerical methodologies such as Unwedge and RS2 software have been used.

4.4.1 Unwedge Analysis

The necessary jointing parameters crucial for analyzing structural instability are depicted in Figure 4. The cavern's longitudinal axis is aligned in the N344°E direction. To account for the inherent uncertainties associated with factors like Joint Properties, Field Stress, Water Pressure, Bolt Properties, a Probabilistic Unwedge analysis is carried out. This analysis aims to assess the potential outcomes on the most critical wedges within the structure. It takes into account various factors, including the maximum support pressure required, the deepest point of the wedges, the minimum factor of safety, and the probability of failure. These considerations collectively help in evaluating the stability and potential risks associated with the structure, while accounting for the uncertainties mentioned earlier.

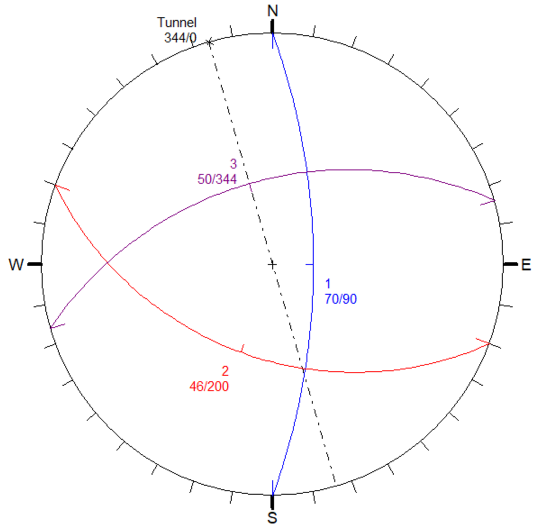

Figure 4: Stereonet plotting of major joints with cavern alignment

The Unwedge analysis reveals some critical findings. Firstly, it indicates that the maximum support pressure required to maintain stability is 0.059 MPa. Secondly, the analysis identifies that the deepest point of the wedges within the structure extends to a depth of 3.84 m. Additionally, it highlights that the weight of the wedges in the roof area falls within the range of 0.011 to 2.035 MN. Furthermore, Figure 5 represents the minimum factor of safety concerning the potential failure of wedges in the cavern's roof obtained from probabilistic analysis in Unwedge.

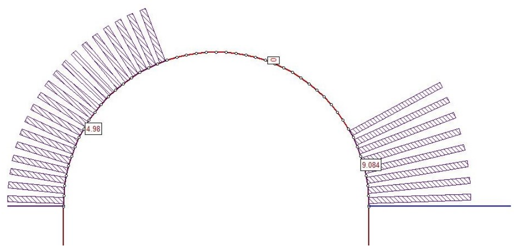

Figure 5: Minimum factor of safety at each segment of roof

Similarly, in figure 6, the probability of failure for each individual roof segment can be observed. This probability of failure represents the proportion of wedges that have failed when compared to the total number of samples or wedges analyzed. It is essentially a measure of the likelihood or chance that a given wedge within the structure may fail based on the analysis conducted.

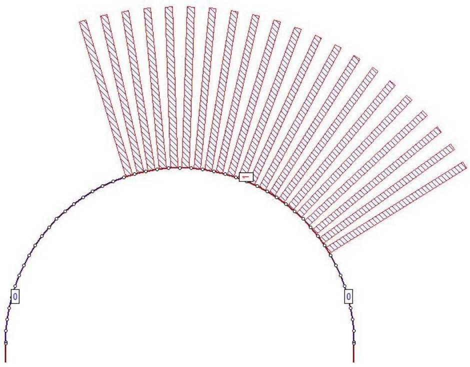

Figure 6: Probability of failure for each segment of roof

In most cases, an underground support system is designed using a combination of rock bolt and shotcrete. 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 use of rock bolts and shotcrete raised the safety factor of the most crucial wedges and reduced the likelihood of failure to zero, confirming the efficiency of the applied support.

4.4.2 RS2 Analysis

A finite element numerical modeling was employed for the analysis. Specifically, RS-2D [9] is a two-dimensional Finite Element Method (FEM) program designed for applications in rock engineering. It allows for the efficient creation and analysis of complex models, particularly those involving multiple stages. RS2 was used to model and analyze the stability of the powerhouse cavern. The numerical modeling in RS2 is conducted as a plane strain analysis, and Gaussian elimination serves as the solver type for the calculations.

Valley Model A 2D topographical model is being used to address a specific analytical problem. This model focuses on a single cross-section that runs perpendicular to the length of a powerhouse cavern. To create this model, topographical data from the hydropower project's working drawings are imported and used. Figure 7 illustrates the topographical representation of this cross-section. In this model, certain constraints are applied to mimic real-world conditions. The model's lower boundary is fixed or constrained in both the X and Y directions, ensuring it cannot move. On the other hand, the lateral boundaries are constrained only in the X direction, preventing movement perpendicular to them. However, the upper surface is left unrestricted, allowing it to move freely both horizontally and vertically.

Additionally, a gravity-type field stress is incorporated into the model, taking into account the actual elevation variations present in the real ground surface. The material used in the model is assumed to be elastic, allowing for the examination of stress within the rock mass without causing material failure. The unit weight of the shale in this model is set at 27 KN/m^3. The modeling approach used here is referred to as planar strain analysis, and it utilizes a Gaussian eliminator as the solver type. Furthermore, the table 3 provides a summary of the in-situ stress ratio, both within and out of the plane, that is employed in the model to represent real-world conditions.

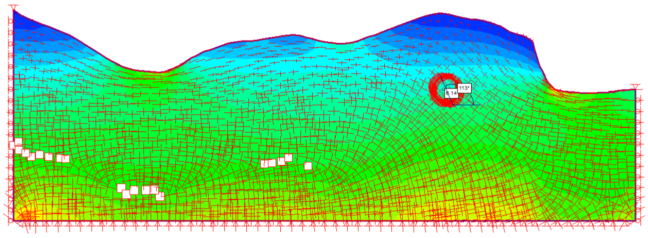

Figure 7: Modeling the stress conditions within the valley scenario (σ_1)

The table 7 displays the outcomes derived from the valley model, including the maximum, minimum, and intermediate horizontal stresses, as well as the direction of maximum stress present at the powerhouse cavern location.

Table 7: 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

Powerhouse Cavern Model

Model Setup in RS2 The vertical cross-sectional representation of the cavern is a simplified version of the original geometry, omitting the bus-bar tunnels and draft tube to facilitate modeling. Typically, large-scale caverns are excavated in multiple phases. However, since the primary focus here is on assessing overall stability, the number of excavation stages has been reduced according to the original excavation plan. The specific count and sequence of model stages are depicted in the accompanying figure 8.

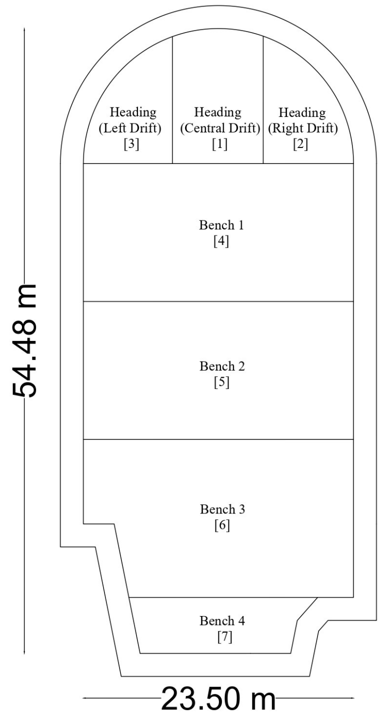

Figure 8: Excavation stages of Powerhouse Cavern

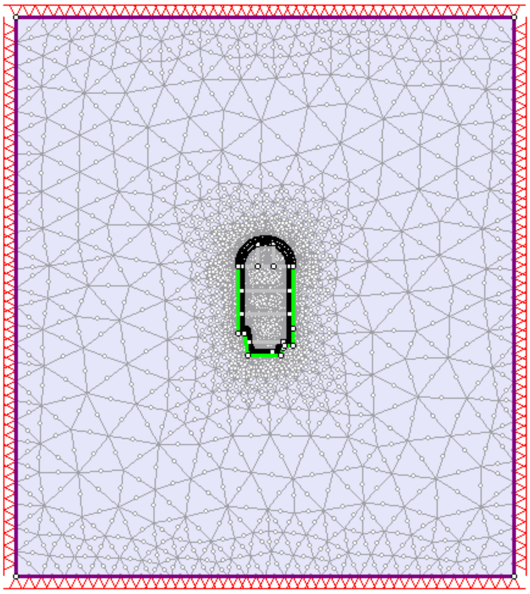

Figure 9: 2D Model of powerhouse cavern in RS2

The external boundary of the model is a rectangular box with an expansion factor of 4. This factor has been chosen to ensure that any potential end effects are adequately avoided. Figure 9 also shows the cavern of 54.48m ×23.5m cross-section modelled in RS2 as described above. The obtained results from Valley Model in Table 7 was used in the 2D model. Given that the main objective of this model is to create a slender shotcrete lining, typically measuring about 0.2m in thickness, so creating a mesh is essential, and in this process, the vertices should be positioned at intervals roughly equivalent to half the thickness of the shotcrete lining.[10]. Forces will be poorly distributed in beam elements with widely spaced vertices utilized in modeling the shotcrete lining [10]. According to [10], a mesh composed of six-nodded triangular elements has proven to yield satisfactory outcomes. However, it's crucial to exercise caution when determining the vertex spacing along the excavation perimeter. The model also incorporated a 2m thick damaged zone around the periphery of cavern geometry with reduced strength.

Modeling and Support Considerations Rock support must be strategically deployed to ensure the stability of the excavation at every stage, including the final phase. In the computer simulation, ground relaxation was mimicked by applying a evenly distributed load to the excavation surface during each excavation stage. Prior to installing the support system, a 50% ground relaxation allowance was considered for each stage of excavation [11].

Comprehending the various excavation phases, the behavior of the rock mass displacement, and the interplay of support measures is paramount in crafting an effective support system design. The analysis suggests that initiating bolt application at an early stage would establish a sturdy foundation and prevent the dislodging of rock blocks.

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. The rock mass displacement varies with excavation depth and the time required for stress redistribution. Therefore, a more suitable approach is to apply shotcrete in layers, aligning it with the construction schedule as determined by numerical modeling. Often, a thin shotcrete layer is placed alongside the reinforcement. This layer can be applied over securely attached wire mesh to prevent small rock fragments from slipping through the gaps between the reinforcing elements. To avoid damage from ongoing deformations as the cavern is excavated below, the final shotcrete layer should ideally be laid as late as feasible. The features of the final shotcrete layer can be tailored to meet a range of requirements. The incorporation of steel or polypropylene fibers, silica fume, and other chemical plasticizers or retardants, as indicated in [12] work, illustrates that with the right mix design, shotcrete having strength up to 60 MPa may be achieved [10]. The modeling was completed using the approach outlined above, and the results are detailed in the sections that follow. The support procedure utilized for the analyses in RS2 is presented in Table 8.

Table 8: Excavation Stage and Rock Support procedure

Stage	Parts	Stress Relaxation	Rock Bolt		Shotcrete	
			6m	12m	50mm	300mm
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]

5. Results and Discussions

The figure 10 illustrates the displacement observed within the powerhouse cavern subsequent to the implementation of structural support measures. The stability of the cavern's roof is effectively maintained due to the utilization of a bolt system and an initial layer of shotcrete. Based on the support capacity curve, while the initial support mechanisms put in place have provided some level of stability, there is still potential for improvement hence, the need of additional layer of shotcrete. Consequently, both in the physical construction and in the model, it is imperative to apply a second layer of shotcrete before proceeding with the next excavation phase.

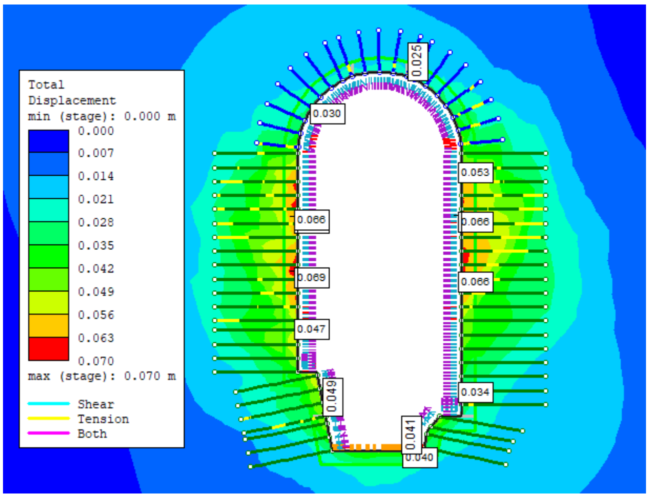

Figure 10: Total displacement and yielded elements at the final excavation stage

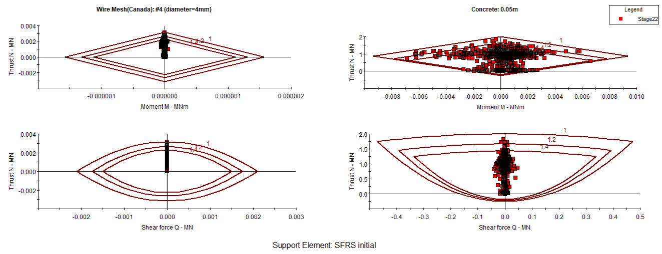

Figure 11: Support Capacity Plot of Initial shotcrete elements

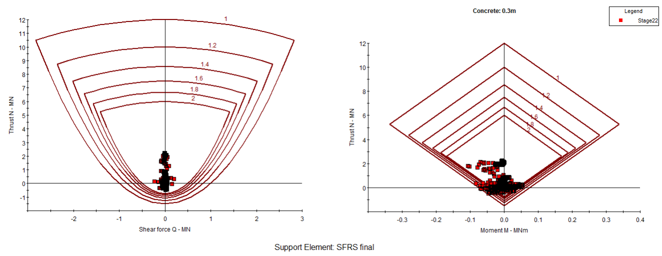

Figure 12: Support Capacity Plot of Final shotcrete elements

In the final excavation stage, after the completion of the last benching and support activities, the cavern is fortified using a system of bolts. These bolts consist of two types: 6 meters in length with a spacing of 2 by 1.5 meters for roof and 12 meters in length with the same spacing for the wall of cavern. Additionally, two layers of shotcrete are applied, initially with a thickness of 50 millimeters, followed by a final layer with a thickness of 300 millimeters. It is noteworthy that the maximum observed displacement in the cavern walls amounted to 70 millimeters.

6. 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 technique - Q-system was initially employed to estimate the necessary rock support. This preliminary information served as a crucial input for the subsequent two-dimensional numerical analysis. The numerical analysis, being more intricate, 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 a 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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References

[1] Krishna Kanta Panthi and Bjørn Nilsen. Uncertainty analysis of tunnel squeezing for two tunnel cases from nepal himalaya. *International Journal of Rock Mechanics and Mining Sciences*, 44(1):67–76, 2007.

[2] Krishna Kanta Panthi. Underground space for infrastructure development and engineering geological challenges in tunneling in himalayas. *Hydro Nepal Journal of Water Energy and Environment*, 2008.

[3] Evert Hoek and Mark S Diederichs. Empirical estimation of rock mass modulus. *International journal of rock mechanics and mining sciences*, 43(2):203–215, 2006.

[4] Evert Hoek. *Practical Rock Engineering*. www.rocscience.com, 2007.

[5] M Cai and PK Kaiser. In-situ rock spalling strength near excavation boundaries. *Rock mechanics and rock engineering*, 47:659–675, 2014.

[6] Krishna Kanta Panthi et al. Effectiveness of post-injection grouting in controlling leakage: A case study. *Hydro Nepal, Journal of Water, Energy and Environment, Issue*, (8), 2011.

[7] Charlie C. Li. Applied rock mechanics. *Norwegian University of Science and Technology*, 2015.

[8] CD Martin, PK Kaiser, and DR McCreath. Hoek-brown parameters for predicting the depth of brittle failure around tunnels. *Canadian Geotechnical Journal*, 36(1):136–151, 1999.

[9] Rocscience Inc. Rocscience rs2 user guide. 2022.

[10] E Hoek. Cavern reinforcement and lining design. prepared for rocnews. 2011.

[11] Gyanendra Lal Shrestha. *Rock Engineering Handbook on Design of Tunnel and Other Underground Structures*. 2021.

[12] T Franzén. Shotcrete for underground support: a state-of-the-art report with focus on steel-fibre reinforcement. *Tunnelling and Underground Space Technology*, 7(4):383–391, 1992.