Difficult Ground Conditions demand for an Observational Approach for a Powerhouse Cavern in the Himalayas

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ABSTRACT: The powerhouse cavern excavation for Tanahu Hydropower Project (140MW), in Nepal, was successfully completed in July 2022. The site is geologically located in the Lesser Himalayan Zone. During cavern excavation, moderately to highly foliated Slate with phyllitic Slate intercalations and localised sheared zones were encountered. To cope with difficult ground conditions, the design of rock support was not only based on 3D-modelling but rather adopted an observational approach. Thus, during excavation a thorough monitoring system was implemented consisting of multi-point borehole extensometers, load cells, convergence stations and piezometers. Data collection and evaluation of results were conducted on a daily basis during the whole excavation process. Aiming to mitigate occurring deformations during the excavation stage, the rock support measures were modified accordingly. Furthermore, to control overstressing, loads from pre-stressed double-corrosion protected anchors required stress release. The timely response to monitoring results was of paramount importance for the successful excavation of the powerhouse cavern.

Keywords: observational method, powerhouse cavern, pre-stressed anchors, instrumentation, phyllitic Slates.

1 INTRODUCTION (GENERAL SITE GEOLOGY)

The powerhouse cavern of Tanahu Hydropower Project (height 45m, width 22m, length 89m) has been entirely excavated in the Slates of Benighat series, a sub-unit of Upper Nawakot Group which is part of the Lesser Himalayan Sequence. The formation consists of dark bluish grey to black slightly to moderately weathered, highly cleaved Slates and Phyllites. The excavation stage revealed weak to moderately strong, thin to thinly foliated fresh Slates. The excavated formation is mainly argillaceous with subordinate bands of siliceous fine-grained Quartzite, as well as localised intercalations of shear zones. The majority of shear zones exists parallel or sub-parallel to the foliation, with a gradually increasing thickness (between 10mm and 80cm), from the North to South end-wall of the cavern. Areas of very poor rock-mass with distinct cleavage and thin foliation/lamination were advanced by mechanical means. The moderately strong rock-mass (UCS: 20-60 MPa) and its high degree of anisotropy (Tsidzi & Kwami 1990) in conjunction with weak interlayers and the aforementioned shear zones resulted to a challenging excavation process during which large deformation and excessive loads in anchors (higher than their design load) were encountered.

2 INVESTIGATION & DESIGN

2.1 *Field and laboratory testing*

To assess the geological regime, a 288m-long exploratory drift was initially excavated and evaluated based on the RMR classification system, where the rock mass quality corresponds to 48.6 % class-III, 37.5% class-IV, and 13.89% class-V. For the particular stretch of the powerhouse cavern, rock-mass is predominantly classified under class-III. Several investigations were conducted before the detailed design and during the design phase of the cavern, and according to the retrieved core samples and in-situ tests, the preliminary parameters of the rock mass are summarised in Table 1.

Table 1. Geomechanical parameters of powerhouse cavern (investigation and design stage).

Parameter	Unit	Value	Comment / Method
Unit weight	KN/m^3	26.3	Laboratory
Young's modulus	GPa	40	Laboratory
Poisson's ratio	%	0.25	Laboratory
Permeability	Lugeon	0.7-5.2	In-situ water pressure test
Tensile strength	MPa	5.0	Brazilian; normal to foliation

2.2 *Rock Mass Parameters, In-situ Stresses and Discontinuities*

The rock mass parameters were determined according to test results (laboratory & field) with consideration of relevant literature (Hoek 1990, Hoek 2007, Marinos & Hoek 2001, Hoek et al. 2002), as well as according to the engineering practice and experience. The recommended values for the whole range of anticipated rock mass qualities are presented in Table 2, below.

Table 2. Recommended geomechanical parameters

Parameter	Rock class				
	I	II	III	IV	V
Unconfined compressive strength (UCS)			38.0		
Density (KN/m^3)	27.0	27.0	27.0	26.0	25.0
Angle of internal friction (φ_{peak})/(φ_{res})	45/36	38/30	38/30	28/22	20/16
Cohesion (c_{peak})/(c_{res})	2.5/1.3	2.0/1.0	1.7/0.9	0.8/0.4	0.2/0.1
Dilation angle (ψ)	30	19	19	9.3	0
Poisson's ration	0.25	0.25	0.25	0.30	0.35
Modulus of elasticity (intact rock)	8.0	6.0	3.0	1.2	0.4
Deformation modulus (GPa)	4.0	3.0	1.5	0.4	0.2

The in-situ stress in the cavern derives from two types of testing; a) hydrofracturing, and b) stress relief method (borehole slotter). The difference in the direction of the major principal stress σ_H (010°to 030°) as well as the minimum principal stress σ_h (280° to 300°) is only in the order of 10° to 20°, and consequently in line with each other. The absolute values of the major principal stress based on the stress relief method is within the range of 27.2-30.2MPa: 2.1~2.3 times higher than the values measured with the hydraulic fracturing method (13.0MPa). The axis of the powerhouse cavern intersects with the direction of the maximum principal stress (σ_1) with an acute angle.

Three major prominent discontinuity sets were identified with some random joint sets. Foliation is the predominant set (J_1) mainly dipping SW (28-45/215-232), a second joint set (J_2) generally dipping NE (45-60/035-050), and a third set (J_3) dipping ESE to SE (55-68/100-130) (Figure 1).

Based on the kinematic geostructural analysis of the North-wall, a planar sliding probability along foliation was considered. However, the intersection of J_2 and J_3 forms potential critical wedges, provided that the maximum principal stress directs towards the South-wall. No significant risk was identified for the East and West-wall.

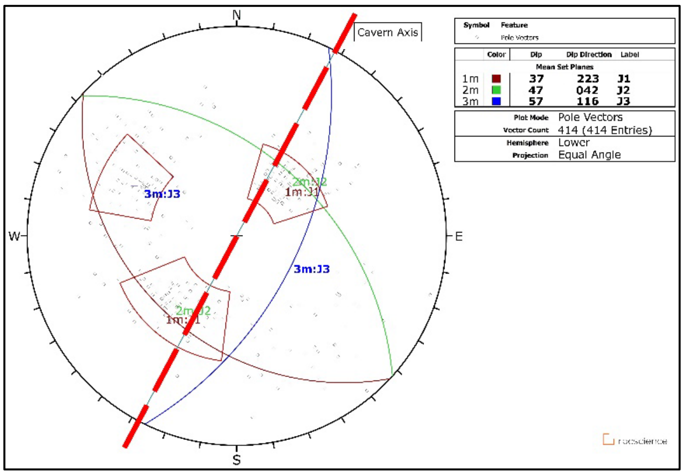

Figure 1. Mean planes of measured discontinuities in the powerhouse cavern (plot produced with DIPS 7.0).

3 POWERHOUSE EXCAVATION

3.1 Excavation and rock support

The pre-defined rock support in the powerhouse cavern consists of:

- 20cm steel-fibre reinforced shotcrete (SFRS) at the crown and 15cm SFRS for the walls,
- 32mm diameter rock bolts, 6m-long, in 1.5X1.5m staggered patterns in the crown and 8m-long in 2X2m staggered patterns for the walls,
- 47mm mono-bar double-corrosion protected (DCP) anchors 10m-long in 3x4m pattern in the crown, 25m-long in 3X3m pattern for sidewalls and 20m-long in 3X3m pattern for the end-walls.

The excavation adopted a typical scheme for large hydropower caverns, starting from a pilot tunnel, widening the roof section followed by benching phase down to the pump sump. The bench excavation was conducted from the central area in 3m steps leaving a 3m-wide berm at each sidewall. The excavation was generally performed by drill & blast. However, weak rock sections were advanced by mechanical means. The first blast for the cavern took place on 28.08.2020 and the final blast for the pump sump was conducted on 03.07.2022. The total excavated volume of the cavern is 78,168 m³.

3.2 Monitoring

The excavation of the cavern has been monitored by an extensive program comprising convergence sections, multi-point borehole extensometers, pre-stressed DCP-anchors equipped with load cells, and piezometers. For conciseness, in the following, only extensometer and load cell data are given.

In the cavern roof, 6 extensometers were installed from the pilot tunnel before widening the roof section. In total 22 extensometers were installed during the excavation stages, 6 in the roof, 5 at each sidewall, 4 at the South and 2 at the North-wall. Furthermore, 87 DCP-anchors were equipped with load cells. Thus, the dense monitoring network of extensometers and load cells allowed for consistent data collection and thorough evaluation. Readings were taken on a daily basis for the majority of instruments during the entire excavation process.

4 OBSERVATIONAL APPROACH

4.1 Encountered geological conditions

In general, the rock mass conditions encountered during the cavern excavation coincide with the geological regime of the investigation adit. However, at several locations, especially at the West (upstream) wall and South-wall, thick shear bands (up to 80cm thickness) are seated within zones of weak and thinly foliated phyllitic Slates (Figure 2).

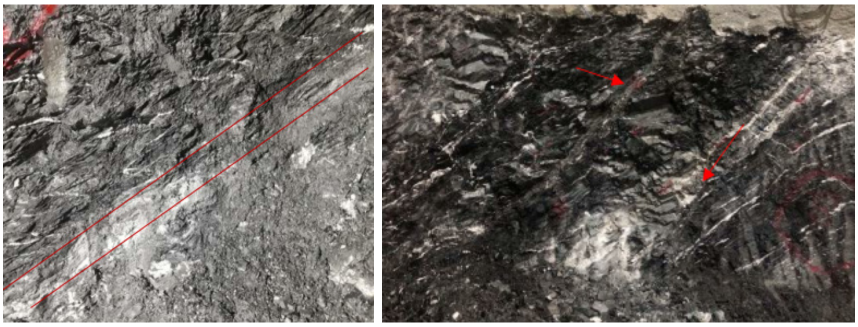

Figure 2. Undulated thinly foliated Slates with shear bands (80cm thickness) and micro-folding.

Foliation dips 30^{0}-45° towards the excavation at the North-wall (Figure 3), which potentially entails stability risk due to planar sliding. However, only blocky Slates without sheared or weakness zones were encountered at the North-wall. In terms of stability, foliation did not cause any stability issues, in contrast to the observed sheared zones and in areas of thinly foliated weak Slates.

Thus, the South end-wall and the southern part of the upstream sidewall turned out to be the most critical areas of the powerhouse excavation due to the occurrence of sheared zones. Although foliation and shear zones dip favourably towards South, the highest displacements and load increase were recorded at the South end-wall.

Figure 3. Blocky Slates with foliation dipping towards the excavation at North-wall.

4.2 Extensometer and load-cell readings

To reflect the decreasing trend of displacements from the excavation boundary to the extensometer fix point, all extensometer data were first converted into actual displacement data. Furthermore, the actual displacement data facilitate the assessment of each extensometer point and provide better clarity for the extent of the plastified zone (Figure 4).

For all DCP-anchors equipped with load cell, the measured load increase has been back-calculated into displacements. Thus, the following figures correspond to load cell displacements instead of loads. The figures refer to data retrieved until mid-October 2022, approximately 3 months after the completion of the cavern excavation.

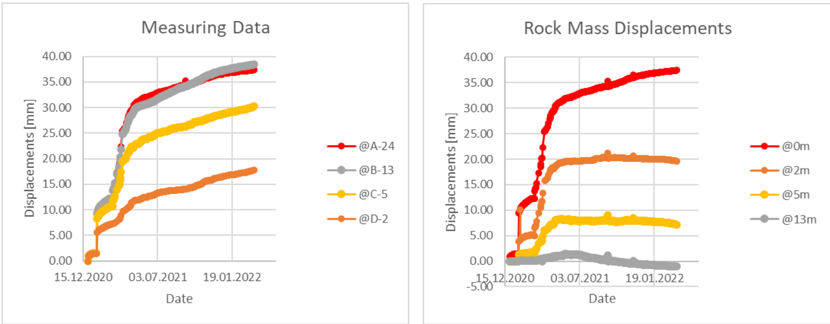

Figure 4. Measured data (left) and actual displacements (right); 0m refers to the excavation boundary.

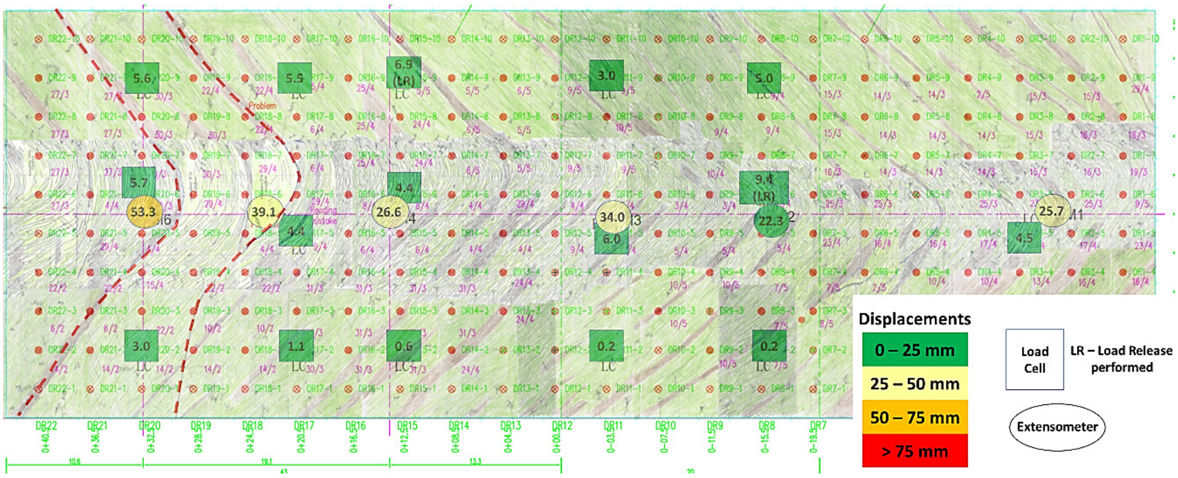

Figure 5. Displacements in the cavern roof with geological mapping overlay.

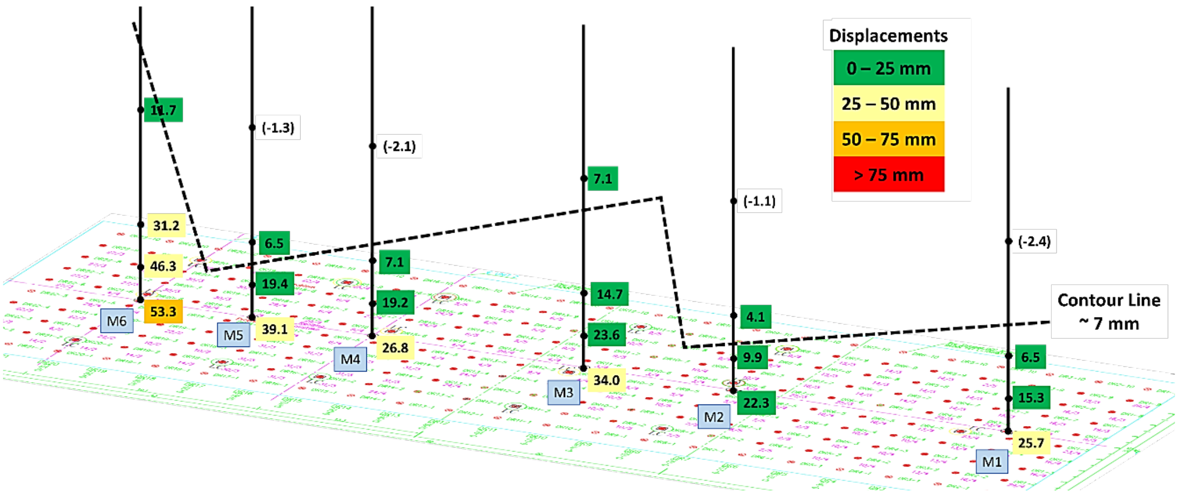

Figure 6. Displacements along extensometer in the cavern roof (dashed line: approx. contour line of 7 mm crown settlement).

The cavern roof revealed increasing displacements towards the South-wall which clearly relates to the occurrence of two thick shear bands which truncate this part of the cavern (Figure 5). Interestingly, the displacement distribution along the roof extensometers is not uniform, but rather follows a zig-zag pattern which probably relates to the southward dipping foliation (Figure 6).

The displacements at the West (upstream) sidewall follow the same pattern as in the roof with increasing displacements towards the South-wall, and an apparent correlation to the observed shear bands (Figure 7). In addition, the extensometer displacements and the back-calculated DCP-anchor displacements generally appear to be in line with each other.

With respect to the recorded displacements, the South-wall constitutes the most critical area. Due to large displacements, basically all load cells indicated exceedance of the design load, requiring load release for all DCP-anchors. Considering a continuous increase of displacements for the upmost extensometer (even during longer breaks of the excavation works), a number of additional DCP-anchors were installed in the upper reaches of the end-wall. In contrast to the systematic anchor support, the supplementary anchors were installed approximately perpendicular to the foliation. Supplementary steel beams were erected in the middle reaches of the cavern, acting as waler and link between anchors. The additional anchors as well as the waler proved to be effective, resulting in diminishing displacement rates.

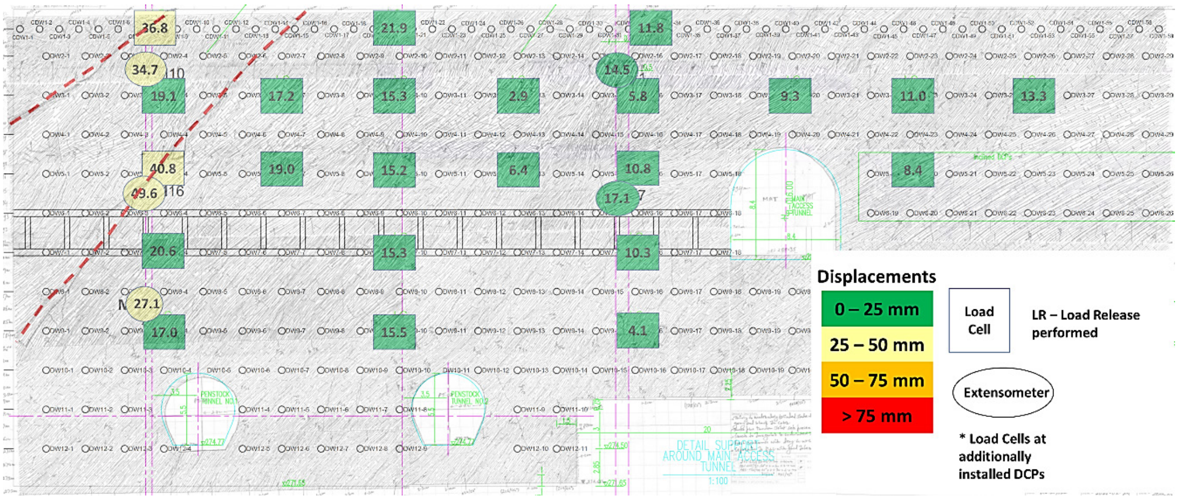

Figure 7. Displacements at W-wall (u/s) with geological mapping (dashed red lines illustrate shear zones).

5 CONCLUSION

The largest displacements (up to 58mm) and highest load-cell increase occurred at the South end wall of the cavern. Consequently, all DCP anchors necessitated destressing; some of them twice. The observed displacements could be linked to sheared zones that truncate the end-wall and/or are associated with very weak rock-mass conditions. The observational approach proved to be a suitable and practicable method for the excavation of a large underground cavern in difficult and anisotropic ground conditions. A comprehensive and dense monitoring program, with the consistent and reliable recording, along with timely evaluation of monitoring data is a vital precondition for the successful implementation of a large underground structure. Furthermore, the harmonious and tight collaboration between all involved parties (Owner, Engineer, Designer and Contractor) is essential for ad-hoc decision-making and the timely response to possibly encountered adverse rock conditions.

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