

Numerical Modelling And Instrumentation For Underground Excavations

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Design of underground excavations is extremely complex because the geological structures and their characteristics are not known to the extent desirable. Simple instrumentation data can provide valuable information about its behaviour when the excavation starts. Numerical Modelling coupled with the back analysis of instrumentation data seems to be the only rational way to proceed. Example of Tehri Underground Powerhouse is briefly described below to illustrate the point.

Introduction

Unlike several other civil engineering structures, where the design is rather straight forward, the design and construction of underground excavations is fairly complex. The complexities are compounded by the facts that the basic data required for the design is either not known or known with several question marks. The geological strata through which the excavations are made are not known precisely even after the excavations get completed. So, is the case of material characteristics, not only the properties of various rock masses but also the characteristics of the interfaces, and the in situ stresses, ground water conditions.

Due to complexities described above it is not possible to design and construct the underground excavations in one go. The design has to be modified as the excavation progresses. The basic philosophy has to be - design as you excavate. The design



Excavation of Core Trench of Tehri Dam

methods used in the past were based on thumb rules or empirical formulations. Starting from the times of Karl Terzaghi, when the rock loads were determined on the basis of rock cover and the type of rock. The classification given by Terzaghi was very broad and more qualitative than quantitative.

Bieniawski (1973) and Barton (1974) gave the two systems of rock mass classification which have been used quite extensively. These systems are known as the Rock Mass Rating System (RMR) and the Norwegian Geotechnical Institute Quality of Rock Mass System (NGI Q). Both the systems go on to correlate the quality of rock mass with the type and extent of support system required for the underground excavation.

The basic data on which the correlation is based was taken



Dam under construction



Chute Spillway under construction



Intake structure under construction

from the existing tunnels and had been provided with more supports than required. It also did not take the time of installation of support system into account. The existing in situ stresses also have been taken into account in a very indirect way.

The only rational method available for the analysis and design of support system is, therefore, based on numerical modelling technique wherein one can take almost all the factors into account. This is particularly true about the underground power houses which have a non-standard geometrical shape, for which closed form solutions are not available.

Underground excavations tend to close under the in situ stresses of the surrounding rock mass and depending on the ground characteristics, this may lead to either reduction of the excavated area or collapse, either of which is not desirable. Therefore, a basic requirement for the success of the excavation of an underground opening is to monitor its deformation during excavation and to control it before it reaches the limits of instability.

Modern methods of underground excavations are based on the principle of controlled deformation rather than no deformation or too much deformation. This is achieved by establishing a detailed monitoring system which not only records the deformations during excavations, but also indicates the need for



A view of the completed works from downstream side

additional supports if the convergence exceeds certain limits. Incidentally, it also tells us about the adequacy of the support system being deployed for the particular reach which depends upon several factors such as in situ stresses, rock mass characteristics, method of excavation etc..

Monitoring of deformations, support pressures and other parameters like pore water pressure, subsidence etc., is crucial for deciding about the optimum support system and provide an economical solution for stable underground excavations.

To illustrate what has been stated above, a case study of Tehri Underground Power House is presented.

Design Of Underground Caverns At Tehri

The first stage of Tehri Hydro Power Project comprises a 260 m high earth and rock-fill dam across river Bhagirathi and an underground power house complex on the left abutment to generate 1000 MW. The project is located in the Himalayan region within complex geological formations. The design process involved continuous evaluation of rock mass parameters, detailed 2D and 3D numerical modelling and monitoring program. Hoek and Brown failure criterion was used to estimate the rock mass strength parameters for the model analysis and the model results



Another view of dam on completion



Dam after completion of construction

validated with the field measurements. FLAC 2D (Itasca 1995a) was used for design of pattern support system. Shear zones and major shear planes were modelled in 3D discontinuum analysis with 3DEC (Itasca, 1995b). Instrumentation scheme was planned and implemented to calibrate the model. This case study demonstrates the role of advanced numerical modelling technique in the analysis and design of the caverns.

Power House Complex

The power house complex consists of two main parallel caverns namely the Machine Hall (MH) and the Transformer Hall (TH) located about 370 m below the surface. The MH cavern is 188 m long, 22m wide and 47 m high. There are four turbine pits 16 m deep on the floor of the machine hall. The TH cavern is 161 m long, 18.5 m wide and 36 m high located upstream of the MH with a 41.75 m rock pillar between them. In addition, there are other excavations such as pressure tunnels, draft tubes, bus duct tunnels and adits joining the main caverns and drainage galleries.

The Tehri Hydropower Project is located in the Lesser Himalayas. The rock formation in the power house area is mainly massive to thinly bedded Phyllitic Quartzite with caverns oriented perpendicular to foliation strike direction. There are four major joint sets and several shear planes with varying thickness (Navani, 1996). There is a shear zone of about 2 - 5 m thick, dipping along the foliation intersecting the caverns. The 3D model of excavation and geological features are shown in Fig. 1.

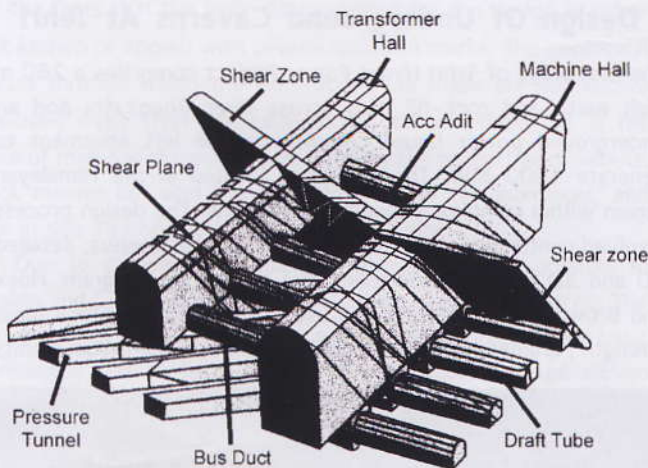


Fig 1: 3DEC Model of Excavation and Geological Features

Numerical Model

The input parameters for the model were estimated based on the methodology proposed by Hoek and Brown (1977). With the progress of excavation, rock mass properties were updated and support design was modified. The final rock mass properties used for the analysis are given in the Table I which were based on the Geological Strength Index = 56 (obtained from visual inspection and geological mapping), and Unconfined Compressive Strength

= 60 MPa and m^1 (Hoek Brown Parameter) = 10 (determined from laboratory triaxial tests).

Table I: Rock Mass Parameters

	E	c	Φ	σ^1
Rock Mass	8.66 GPa	2.75 MPa	33°	0.22 MPa
Shear Zone	4.0 GPa	1.5 MPa	30°	0.10 MPa
Shear Plane		1.0 MPa	28°	

The in situ stresses were measured at the crown by hydrofracturing test. It was established that the horizontal to vertical stress ratio along the caverns was 0.526 and across the caverns was 0.314, while the vertical stress was equivalent to the overburden rock mass.

The power house complex with MH and TH was analyzed in 2D to establish pattern support design. In this analysis, the construction sequence of the excavation and support installation was modelled considering the rock mass to be continuum with Mohr Coulomb non linear material behaviour. The effect of rock bolt was considered in the support interaction analysis neglecting the shotcrete. The length and spacing of rock bolts were optimized by a series of analysis, which also involved updating of rock mass properties during construction. The results of the analysis showed that the failure zone on the roof was about 3 m for both MH and TH, and on the walls it extended to about 12 - 13 meters for MH and about 9 - 11 meters for TH at the mid-height. The displacement on the roof and mid-height of MH was about 2.5 cm and 2.25 cm and that of TH was it was 0.9 cm and 1.5 cm. The final support design consisted of pre-tensioned rock bolts: on the roof 25 mm dia, 6 m and 10 m long for MH and 6 m and 8 m long for the TH, on walls 32 mm dia of varying length from 9 m to 15 m for MH and of varying length 8 m to 13 m for TH. The axial forces developed in the bolts were within the bolt capacity. The excavation sequence, deformed boundary, failure region, and rock bolts with axial forces are shown in figure 2.

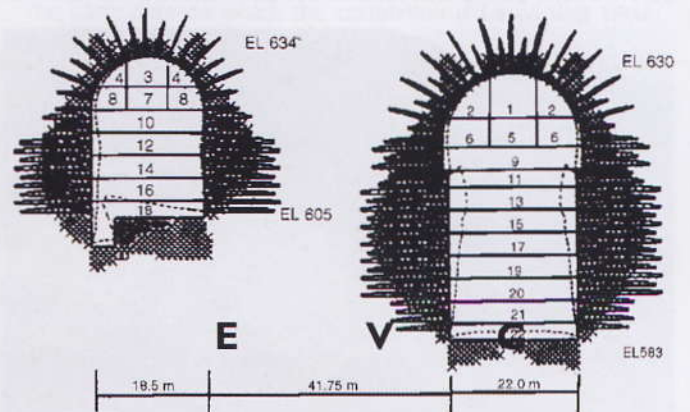


Fig 2: Failure Region and Rock Bolts with Axial Forces

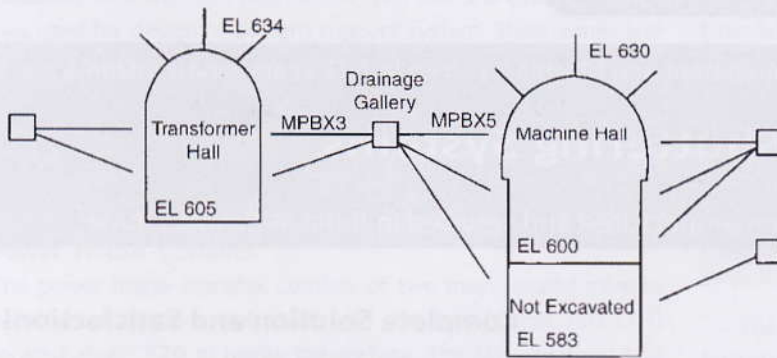


Fig 3: Layout of Instrumentation

The power house complex was analyzed in 3D with busduct, draft tube and pressure tunnels joining the MH and TH caverns. The geological features mainly the folded shear zone, and about 19 shear planes were explicitly modelled as shown in figure 1. Discontinuum analysis was performed to understand the interaction of multiple excavations with geological features and identify the regions requiring enhanced support.

Model Calibration

Instrumentation was planned and installed at appropriate locations and time during excavation so that maximum information could

be obtained for the calibration of the model. Rock deformation was measured by mechanical type MPBX. The extensometers were installed on the roof of both the caverns from inside, i.e. after the excavation of the crown advanced to the desired location, however, monitoring usually becomes difficult with benching.

The MPBX on the walls were installed and monitored from the drainage galleries before the excavations reached the instrumented levels. The wall was monitored un-interrupted by the construction activity. A layout of the instrumented cavern is shown in figure 3. Caverns were instrumented at three sections.

The wall instruments were installed in a phased manner when excavation level of MH was EL 609 and TH at EL 618 and the exact sequence of excavation and installation of the MPBX was modelled using FLAC. The comparison of the calculated and measured deformation from the cavern wall into the rock mass for MPBX 3 and MPBX 5 is shown in figures 4 and 5. It can be seen that the results are in close agreement indicating proper validation of estimated rock mass parameters and support design.

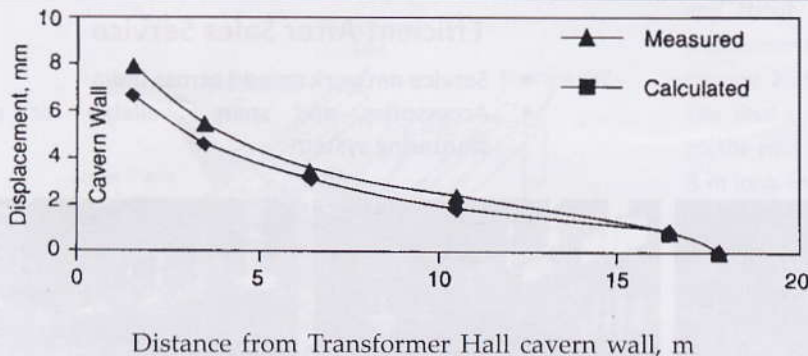


Fig 4: Calculated and Measured Data-MPBX 3

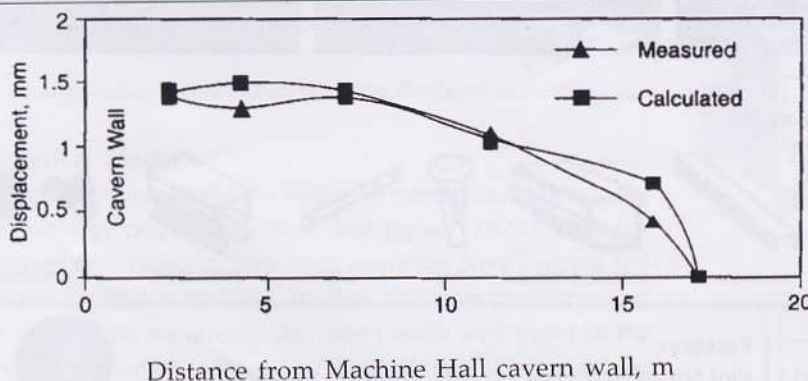


Fig 5: Calculated and Measured Data-MPBX 5

Conclusions

Numerical modelling provides a tool to the designer such that various parameters affecting the stresses and deformations of underground openings can be taken into account. It is possible to simulate various shapes, sequence of construction and various support systems to arrive at the optimum designs.

However, it is essential that some measurement data should be made available so that the models can be verified and calibrated.

References

- Hoek, E and Brown, E.T., 1997, Practical estimates of rock mass strength, Int. J. Rock Mech. & Min. Sci. & Geomech Abstracts, 34(8) : 1165-1186.
- Itasca Consulting Group, 1995a, 1995b FLAC, Fast Lagrangian Analysis of Continua, Version 3.3, and 3DEC Version 1.5 Minneapolis, Minnesota, USA