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Rock mass investigations for underground openings and 2D FEM analysis -A case study

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Abstract

The Koyna Hydro-Electric Project, Stage-IV, in India envisages construction of an underground powerhouse with an installed capacity of 1000MW. Due to complex topography of the region, rock mass investigation work was required to be carried out. The geological report indicated presence of alternate layers of volcanic breccia and basalt. Field and laboratory studies were carried out to measure the rock mass properties, e.g. in situ stress, deformation modulus, compressive and tensile strength, shear parameters, Poisson's ratio, density, etc. in the area where openings were made and in the surrounding areas. The measured rock mass parameters were used to carry out 2D finite element elastic and plastic analysis of the four openings to check their stability with the help of Phase2, FEM program developed and supplied by Rocscience Inc. Toronto. The effect of providing rock bolts and shotcrete was also studied. By adopting actual deformation modulus at its location for different zones of rock mass the plastic FEM analysis shows stable openings with deformations equivalent to the extensometer results.

Introduction

Underground powerhouse at Hydro Electric Power Projects generally consists of underground caverns to accommodate turbines and generators/transformers. The stability of underground openings depends upon stresses and properties of the rock mass, design geometry, rate of excavation etc. The evaluation of stability of an underground structure calls for determination of the regional geology and its influence on the stress and displacement distribution. Any rational procedure for designing underground openings includes laboratory, field and theoretical studies of the rock behavior. One of such design requirements of underground openings is the stability analysis by Finite Element Method, in which the stress distribution and the displacement at various points around the openings are to be evaluated along with providing the necessary supports, if required. Present analysis is carried out by 2D FEM; CWPRS has the facility to carry out the three dimensional BEM and FEM analysis.

Details of the Underground Openings

The173m long underground openings at Koyna Hydroelectric project, Koyna, Maharashtra, India, are depicted in Fig. 1. The height, width and length of the Machine Hall opening are 50.14m, 20.6m and 145m, respectively. The Valve house opening is 13.15m 7.0m 145.0m in size and is located at a distance of 15.5m on the right side of the Machine Hall cavern. The Collection Gallery, which is of 10.6m 10.8m 173m size, is located at a distance of 18.7m on the right side of the Machine Hall. The Transformer Hall cavern is of 23.5m 20m 173m and is located at a distance of 45.0 m on the right (extreme) side of the Machine Hall. The Powerhouse openings are non-symmetrical and their lengths are not very large but are parallel to each other.

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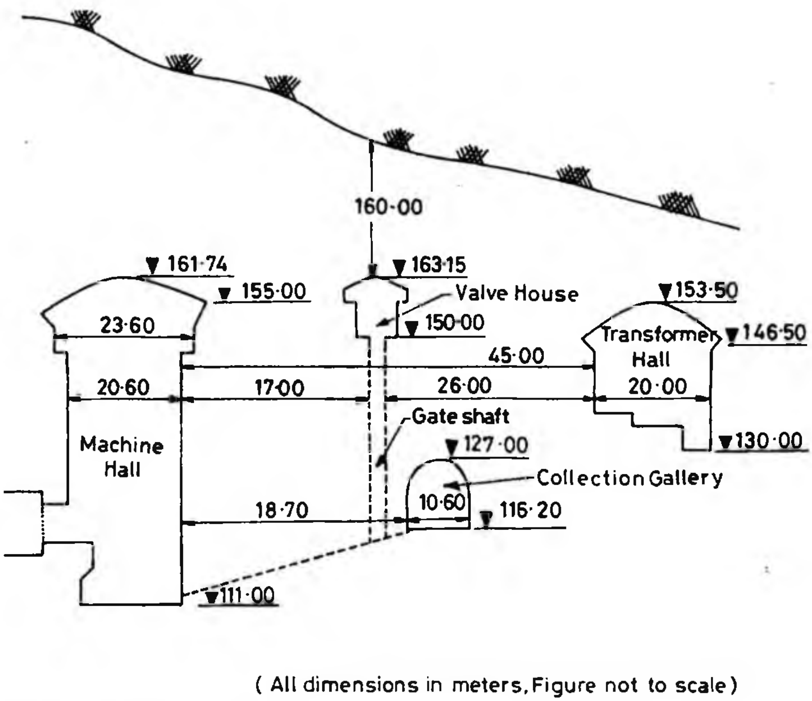

Fig. 1: Schematic diagram of the openings at Koyna Hydroelectric project

Geological Features

The rock mass comprises horizontal and vertical brecciated rock horizons, at several locations, in the main Amygdaloidal Basalt rock, as depicted in Fig. 2. The Volcanic Breccia occurring in between the compact Amygdaloidal Basalt generally exhibits undulating top surfaces. It also contains some red Tactylyte at several locations. The average overburden thickness on the site can be assumed to be equal to 160 m.

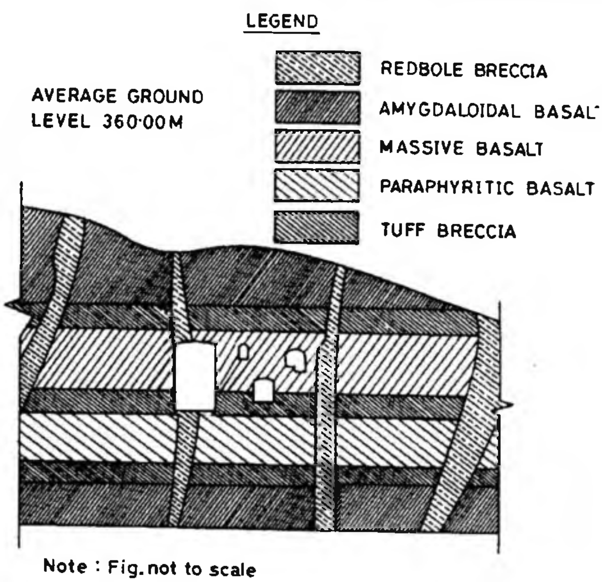

Fig. 2: Idealized geological cross-section of the powerhouse area.

As per the geological report and lithologs of borehole the rock mass was selected for the analysis was divided into about 23 groups. From these zones samples were collected and tested for different parameters of rock mass.

In Situ Stresses

When an opening is introduced in the rock mass, the natural state of stress is disturbed locally as the rock mass attains a new equilibrium (Cornet, 1996). The stress around an opening resulting from various man-made activities is termed as the induced stress, which depends on the original state of stress. This original state of stress is referred as *in situ* stress. *In situ* stresses have been measured in the walls of underground galleries by flat jack (IS: 7292-1974) and in boreholes (ISRM, 1984) by hydraulic fracturing method. They are complementary to each other, each offering different advantages and disadvantages. The stress measurement techniques perturb the rock in order to create a response that is theoretically modeled to estimate part of the *in situ* stress tensor. In the flat jack test, the rock is partly unloaded by cutting a slot and then reloaded; the in situ stress normal to the slot is related to the pressure required to nullify the displacement that occurs because of slot cutting (Ramegowda, 1990).

Flat jack tests were conducted at 12 locations as shown in Fig. 3 by cutting a thin slot into rock surface by drilling a series of overlapping holes. The slot cut in the horizontal direction would yield stress, P , tangential to the boundary of the opening and the slot cut vertically would yield stress, Ph, parallel to the axis of the openings at the respective test locations. The results obtained are listed in Table 1. The average vertical (σ_v) and horizontal (σ_h) stresses were 6.86 and 4.80 MPa, respectively (CWPRS, 1991). The value of *in situ* stress ratio K^R (ratio of σ_h to σ_v) is 0.7.

Estimation of *In Situ* Rock Mass Properties

The performance of underground openings depends largely on the properties of the rock

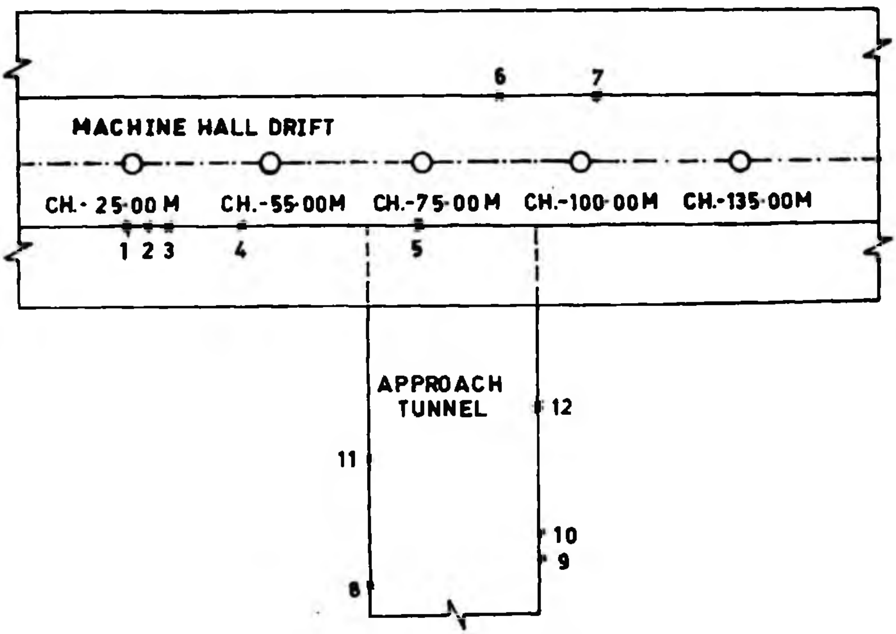

Fig. 3: The plan showing flat jack test locations.

Table 1: Flat jack test results

Location	Chainage (m)	Test Point	Direction	E_m (GPa)	Induced Stresses (M Pa)	
					P_θ	P_h
Machine Hall Drift	25.50	1	Horizontal	22.06	15.6	-
	29.00	2	Vertical	16.86	-	4.1
	32.00	3	Horizontal	15.49	6.5	-
	42.00	4	Horizontal	12.00	-	-
	76.30	5	Horizontal	22.06	4.7	-
	84.30	6	Horizontal	22.06	3.6	-
	102.15	7	Horizontal	13.50	10.7	-
Approach Tunnel	1000.00	8	Horizontal	22.50	8.9	-
	1017.00	9	Horizontal	16.86	10.1	-
	1021.75	10	Vertical	22.50	-	4.1
	1035.00	11	Horizontal	15.49	6.7	-
	1045.50	12	Horizontal	18.60	14.8	-

mass, which are determined by *in situ* and laboratory testing. The need and the importance of these rock mass properties increases as a function of the complexities involved in the rock mass system as well as in the methods of analysis. Some of the rock mass properties relevant to the present study and their evaluation procedure (Lama and Vutukari, 1978) are briefly described as follows.

Deformability of the Rock Mass

The static modulus of deformation of the rock mass Em characterizes its deformability. The Em is influenced by the extent and orientation of the discontinuities that are present in the rock mass.

In situ Tests

Large-scale *in situ* tests are necessary to measure and evaluate E_m of the rock mass. Flat jack in the available drifts and plate-jacking test on surfaces have been extensively used to measure the modulus of deformation. The modulus of deformation is determined by the flat jack tests by canceling the convergence occurred in the slot; load is applied with the help of hydraulic pump. At the fixed stress intervals, the deformations are noted for the loading and unloading cycles

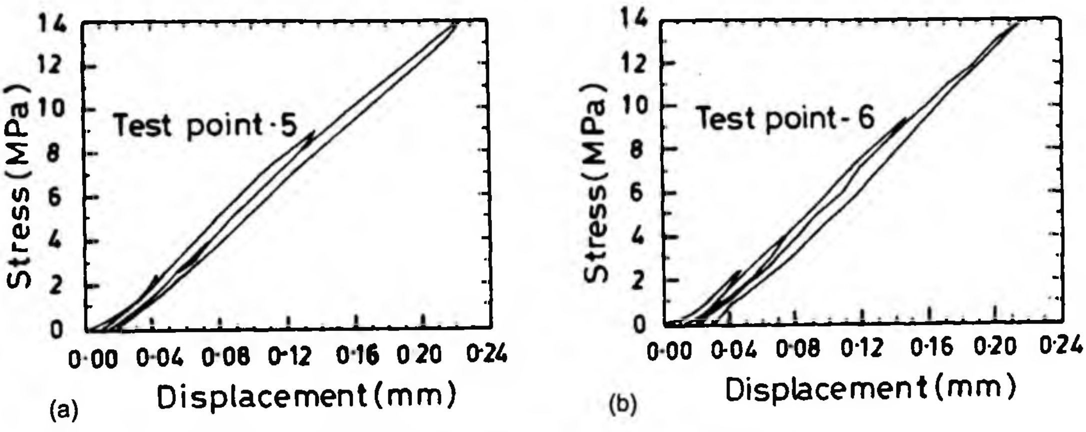

Fig. 4: Stress-deformation behavior of the rock mass (using the flat jack test).

and from the stress deformation values modulus of deformation is determined (IS: 7292-1974). The plots of stress deformation curves for the points 5 and 6 are depicted in Fig. 4. E_m values at some locations (CWPRS, 1993) are presented in Table 2.

Shear Strength of the Rock Mass

For determining shear strength parameters of the rock joint surfaces, rock blocks are constructed on representative grade of the rock mass. For evaluating shear strength of the interfaces/intact rock, in situ tests are conducted on rock blocks of size varying from 60cm 30cm 30cm to 150cm 100cm 100cm, carved out of the rock with the help of hand operated tools. The in situ shear tests for jointed rock mass were conducted as per (IS:7746-1975). The results are presented in the Table 2.

Estimation of Rock Material Properties

Field tests were conducted in different zones, defined by the geological investigations. Simultaneously, boreholes were made at the selected locations and rock cores (NX size) were collected from different depths. These rock cores were tested for determining shear strength parameters, compressive and tensile strengths, elastic modulus and Poisson's ratio. Stress-strain relationships for these rocks are depicted in Fig. 5 (a) and (b), respectively. By determining the correlation coefficients of 1.5 between the results of field and laboratory tests for different zones, deformation modulus and cohesion are determined for different zones of the rock mass, which could not be determined by field tests. The rock cores were also tested for porosity and density. The results of these tests are also presented in Table 2.

The overall average value of unconfined compressive strength for the Basalt and Breccia were varying from 45.2MPa to 58.2MPa and 19.3MPa to 35.3MPa, respectively. Hence, a mean value of 53.6MPa and 29.90MPa was chosen for Basalt and Breccia, respectively. The mean Poisson's ratios, v', for these rocks and the whole rock mass were 0.13 and 0.18, and 0.15 respectively. The mean value of uniaxial compressive strength, for the whole rock mass was found to be 40GPa. The mean value of tensile strength of these rocks was found as 4.15MPa and 1.60MPa. The mean deformation modulus, E'_m, values were 5.2GPa and 22.06GPa for Breccia and Amygdaloidal Basalt, respectively, and for the whole rock mass was determined as 14GPa. Mean density, , of 26.5 kN/m^3 was assigned to these rock masses. For the sake of completeness, the determined rock mass properties with the above defined methods and further used in the present analyses are presented in Table 2.

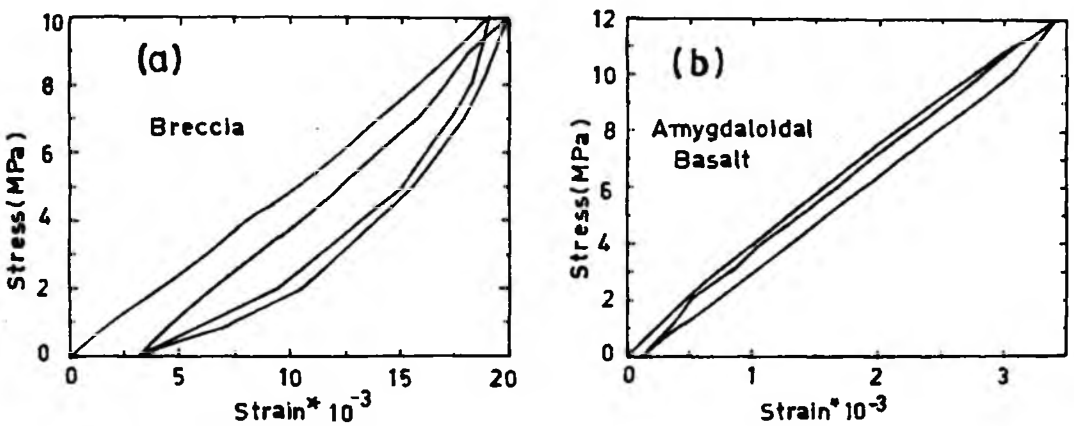

Fig. 5: Stress-strain behavior of the rock material.

Table 2: The Rock mass properties used in the FEM analysis

Element Group	Deformation Modulus, E_m (GPa)	Poisson's Ratio, ν	Cohesion, c (MPa)	Angle of friction, ϕ (°)
1	16.86	0.18	0.6	41
2	5.20	0.13	0.2	35
3	16.86	0.18	0.6	41
4	8.73	0.13	0.3	36
5	22.06	0.18	0.6	41
6	8.73	0.13	0.3	36
7	22.06	0.18	0.6	41
8	22.06	0.18	0.6	41
9	5.20	0.13	0.2	35
10	18.60	0.18	0.5	40
11	8.73	0.13	0.3	36
12	8.73	0.13	0.3	36
13	18.60	0.18	0.6	41
14	8.73	0.13	0.3	36
15	18.60	0.18	0.6	41
16	8.73	0.13	0.5	40
17	18.60	0.18	0.6	41
18	8.73	0.13	0.3	36
19	18.60	0.18	0.6	41
20	8.73	0.13	0.3	36
21	8.73	0.13	0.3	36
22	18.60	0.18	0.6	41
23	18.60	0.18	0.6	41

Measurement of Deformations

Extensometers were used for measuring deformations at the arch of Machine Hall and Transformer Hall caverns. These caverns were excavated from top to bottom and borehole extensometers were installed from August 1991 to January 1993. Deformations were measured using electrical readout units from August 1991 to June 1995. Multipoint borehole extensometers were used to determine the depth of loosened rock mass and deformation of the rock mass at different depths at the selected points due to loosening of the rock mass during the excavation of underground openings. The vertical borehole extensometers were installed at the center of the Machine Hall and inclined extensometers were installed at 5m distance from the right wall, at different chainages from the start of the opening. The deformations at the surface of the openings were measured with the help of tape extensometers. The measured cumulative deformations in Machine Hall and Transformer Hall varied from 1.80cm to 2.50cm.

There was time lag of about three to eleven months between excavation of openings and installation of the extensometers for measuring deformations due to site conditions. The corrections required in the deformations due to delay in installation of extensometers were taken into consideration while setting the initial readings of the extensometers.

Finite Element Analysis

The deformation results of the FEM analysis for various type of material behavior for the underground openings at Koyna Stage-IV Project were compared with the field extensometer measurements. CWPRS has SOLVIA FEM programs developed by SOLVIA Engineering AB, Sweden for 2D and 3D analysis, Examine 3D and Phase², developed and supplied by Rocscience Inc. Toronto. Phase2 was employed for the present FEM analysis with the assumed conditions listed below.

i) Since the openings were located at an overall depth of about 160m and much away from slopes of the hill, the directions of the principal stresses were assumed to be vertical and horizontal at the center of opening.

ii) Although, cavities were situated in an inhomogeneous medium, individual elements were considered to be homogeneous.

iii) Stresses in the direction normal to the plane under consideration were ignored, as their influence would be negligible.

iv) The stresses were unique and the solution was independent of the sequence of excavation for a linear, time independent and elastic material.

v) The excavation of the caverns were simulated sequentially in the following steps

Step - 1 : No opening in the rock mass (0% excavation),

Step - 2 : Excavation of the machine hall,

Step - 3 : Excavation of the machine hall and valve house,

Step - 4 : Excavation of the machine hall, valve house and collection gallery,

Step - 5 : Excavation of all the openings.

For validating the type of analysis deformation, obtained from the FEM analysis for elastic and plastic behavior, the results were compared with the field extensometer measurements.

2D Elastic Analysis with Average Properties of Rock Mass

The rock mass of 300m x 360m (by keeping a cover of about 4 times of the size of openings), which is assumed to be linearly elastic, is discretized by Two Dimensional Triangular Elements. The weak zones and shear zones in the rock mass are represented by equivalent continuum material properties (Singh, 1973). The density of the rock mass was taken as 26.5 KN/m^3. Further, the ratio of *in situ* stress $_h/_v$, determined by flat jack tests, was taken as 0.70, which was adopted from primary stress $_v$ and σ_h, measured in vertical and horizontal directions, respectively. The left and right sides were free to move in Y direction. The bottom side was

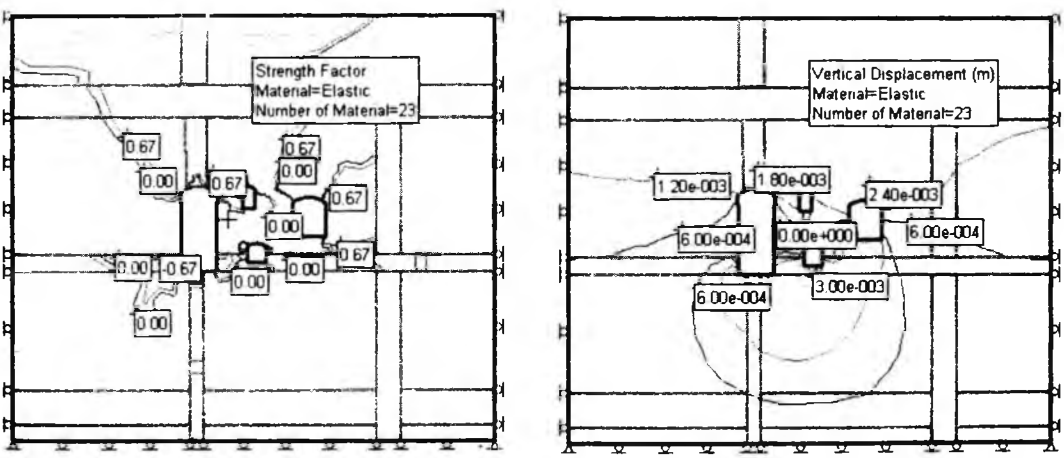

Fig. 6: Elastic analysis with average properties of Rock mass

free to move in X direction. Top surface was considered as free and bottom ends were pinned. By adopting average E_m value as 14GPa and Poisson's ratio of 0.16, FEM analysis was carried out for all the five stages. The strength factor and vertical displacement results at Stage 5 are shown in Fig. 6. The vertical displacement was 0.30cm, much lesser than the measured displacement of 1.80cm by extensometers when all four openings were excavated. So 2D elastic analysis with average properties of rock mass was not found to be correct analysis for underground openings.

2D Elastic Analysis with 23 Properties of Rock Mass

In this analysis, the rock mass of 300m x 360m, is divided into 23 groups as Em values are varying from 5.20GPa to 22.06GPa. The rock mass, which is assumed to be linearly elastic, is discretized by Two Dimensional Triangular Elements. The properties like density, in-situ stress, fixity conditions were adopted as in 6.1. By adopting Em values and Poisson's ratio from Table 2, FEM analysis was carried out for all the five stages. The strength factor and vertical displacement results at Stage5 are shown in Fig.7. Maximum vertical displacement is 0.24cm much lesser than the measured displacement of 1.80cm by extensometers when all four openings were excavated. So, 2D elastic analysis with 23 properties of rock mass has not been found to be the correct analysis for underground openings.

2D Plastic Analysis with 23 Properties of Rock Mass

In this analysis, the rock mass of 300m x 360m, is divided into 23 groups as per Em values, which is assumed to behave as plastic. The rock mass with Em values equal to 11GPa or less would behave in plastic manner (Borsetto and Ibacchi, 1979). The rock mass is discretized by Two Dimensional Triangular Elements. When the induced stresses in the rock mass due to creation of openings exceed the *in situ* stresses and are more than the strength of the rock mass, then plastic zones may develop in the surrounding areas of openings Aydan et al. (1996). A Mohr Coulomb failure criterion has been adopted for the plastic analysis. The properties like density, *in situ* stress, fixity conditions are adopted as described earlier. By adopting E_m values, Poisson's ratio and shear parameters from Table 2, plastic FEM analysis was carried out for all the five stages. The strength factor, vertical displacement and yielded element results are shown in Fig.8. Maximum vertical displacement was 1.84cm much closer to the measured displacement of 1.80cm by extensometers when all four

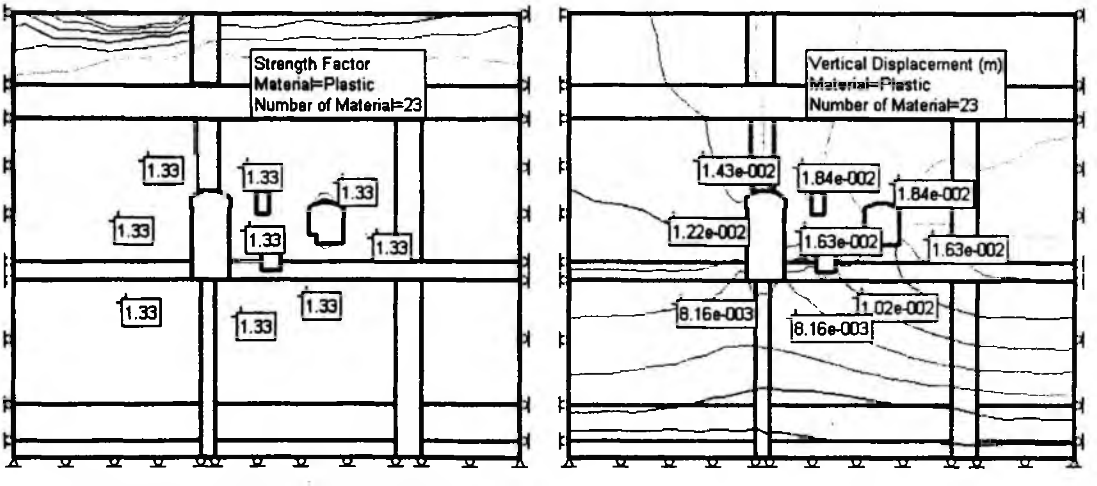

Fig. 7: 2D Elastic analysis with 23 properties of Rock mass

openings were excavated. So 2D plastic analysis with 23 properties of rock mass was found to be correct analysis for underground openings.

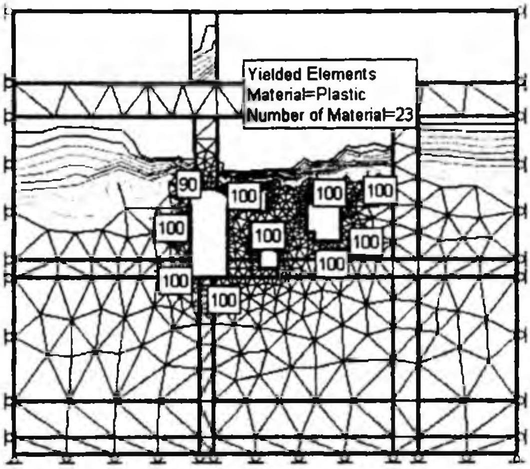

Fig. 8: 2D Plastic analysis with 23 properties of Rock mass

Rock Reinforcement

From the analysis results as shown above, Fig. 8 shows the strength factor of 1.33. Strength factor represents the ratio of available rock mass strength to induced stress, at a given point. From the yielded element plot yielded/failure zone is decided. It is seen from Fig. 8 that there are 90 percent yielded elements on the top left corner of machine hall and 100 percent in the other areas. In fully grouted bolts the load is transferred through the grout to the rock and the deformation of the rock mass and the reinforcement cannot be separated. The load is distributed over a limited distance from the rock joint (approximately 5-20 bolt diameters). To reduce the yielded zone, 10m long fully bonded bolts of 25mm diameter are provided on 8x8m grid in the roof portion of the machine and transformer hall and plastic analysis is carried out for all the five stages. The other properties of steel bolts adopted are E_m =211GPa with yielding stress of 7MPa. There is improvement in the strength factor at the top of transformer hall and collection gallery (Fig. 9). Further the percentages of yielded elements have further reduced (Fig. 9).

Shotcrete Lining

The two main openings have been lined with 10cm thick layer of steel fibre reinforced shotcrete (SFRS). The other properties of shotcrete adopted are Em =30GPa with compressive stress, peak and residual, of 35MPa and 5MPa. There is improvement in the strength factor at the top of transformer hall and collection gallery (Fig.10). Further the percentages of yielded elements have further reduced (Fig.10) at the top portion of the openings.

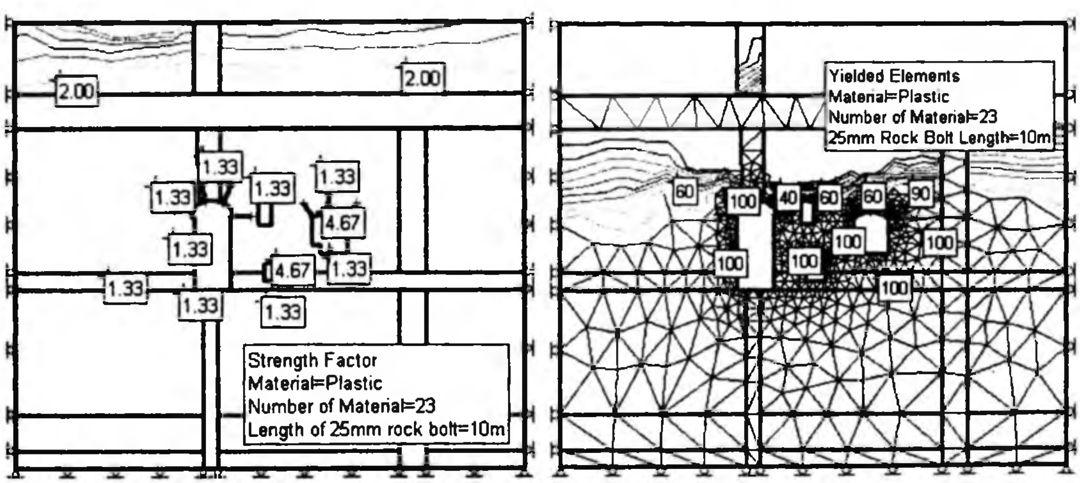

Fig. 9: 2D Plastic analysis with 23 properties of Rock mass with rock bolts

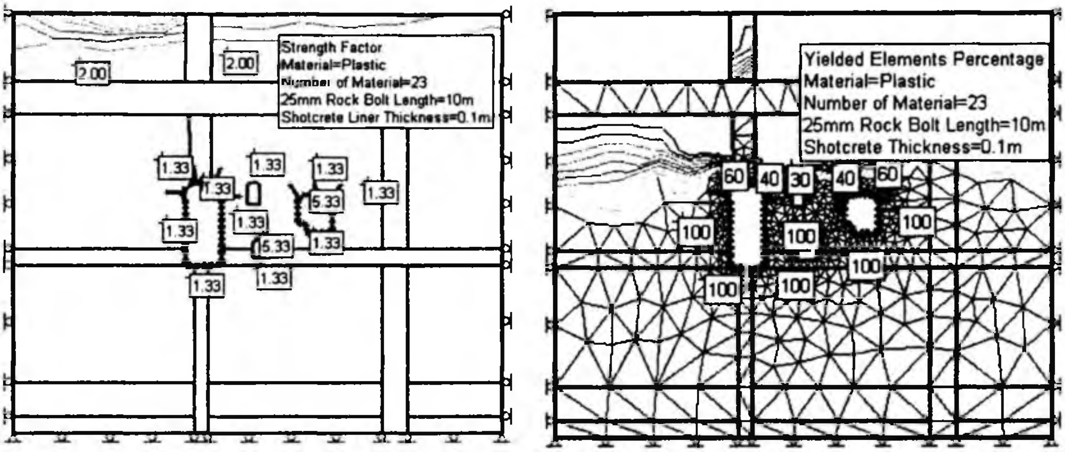

Fig. 10: 2D plastic analysis with 23 properties of rock mass with rock bolts and SFRS lining.

Discussion

The secondary reinforcement is provided with 2 to 3m long 20mm bolts at 2 to 3m spacing with wire mesh before providing shotcrete for safety measures. There is no need to provide secondary reinforcement from support point of view.

The other parameters, which can be determined, are stresses σ_1 and σ_3, horizontal displacement, maximum shear stress and strain. The shape of the caverns is such that there is no axial symmetry in any direction and length to width ratio is almost equal to 8. The accuracy may be about 80percent when the system of underground openings is analyzed with 2D analysis as compared to 3D analysis.

Conclusions

Present study deals with 2D finite element analyses of a set of underground openings that exist at Koyna Hydroelectric Project, Koyna, Maharashtra, India. Linear elastic and plastic analysis have been considered to determine its deformations and strength factors. Based on the study, the following major conclusions can be derived.

1. The study brings out the fact that for the rock mass with widely varying Em, in different zones, use of aggregate representative value of E_m is not correct. Hence, the effect of weak zones in the rock mass can be understood well by adopting the actual E_m values of different zones and their location in the model.
2. A comparison of the computed deformations vis-à-vis those obtained from the in situ measurements indicate that the rock mass with plastic analysis is noted to yield correct deformation values.
3. When support measures like rock bolts and shotcrete are provided then there is minor improvement in the strength factor and reduction of deformations but reduction in percentage of yielded elements is clearly seen.

Acknowledgement

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