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2d Finite Element Analysis of Underground Openings using Three Models for Rock Mass Behaviour

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Introduction

The need for better performance, safety and reliability of structural components have led to an increasing interest in the development of numerical solutions for underground openings to predict their lifetime performance. The strategy adopted being the use of realistic rock mass parameters instead of approximate or average values. The limitation in modelling is expected due to the difficulties associated with obtaining realistic input data. Therefore, in the recent past, numerical simulation has been preferred to the modelling (Giraud et al., 1993). The description of rock mass has shown constant progress using both empirical methods based on hundreds of case histories and more rational approaches based mainly on laboratory or in-situ testing. To design an economical support system, the designer has to evaluate as accurately as possible, the stress distribution around the structure, taking into consideration the diverse material properties, presence of discontinuities, inhomogeniety of the rock medium and in-situ stress conditions.

A critical appraisal of the available literature indicates that most of the work is done for axisymmetric openings where it is easy to achieve convergence for a mathematical algorithm employed (Carrubba and

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Cortellazzo, 1996). While carrying out the analysis, emphasis is given to the shape and size of the openings and elastic properties of the rock mass (Hoffman and Ehgartner, 1993). In some cases, the non-linear parameters of the rock mass have been considered, but comparison with in-situ measurements has not been done to validate the obtained results (You et al., 1996). The rock mass generally has high degree of variability in rock types and properties and may also contain extensive geologic features, such as dykes and faults, or set of discontinuities such as bedding planes, joints, etc. As such the computational algorithm (or the model) should be capable of representing the rock mass conditions accommodating all these features. In addition to this, non-linear constitutive behaviour and potentially large strain deformations must also be considered. In such a situation, Finite Element Method has been found to be an efficient tool, capable of handling such complexities associated with different engineering problems (Bathe, 1980).

Most of the past studies on stability analysis of the underground openings (Sakurai, 1977; Sharan, 1989; Zheng and Khodaverdian, 1996; Aydan et al., 1996; Picha et al., 1996; Richards et al., 1996; Grasso et al., 1996; Faiella and Garino, 1996; Sloan et al., 1996; etc.) are based on rock mass parameters which are generally assumed or are representative values for the rock mass. However, to obtain results for practical design of underground openings, realistic behaviour of materials, and an appropriate model of analysis need to be adopted.

To verify the relative suitability of various models for the design of underground openings, incorporating practical points of view, 2-D elastic, nonlinear elastic and elasto-plastic analyses have been carried out, in this paper, for a set of four underground openings that exist at Koyna Hydro Electric project, Maharashtra, India. The in-situ deformations have been measured by borehole extensometers and are compared with the computed deformations.

Description of Cavities and the Geology

The Koyna Hydropower House openings analysed in this paper are shown in Fig.1. These openings are located in Amygdaloidal Basalt rock. The height, width and length of the Machine hall cavern are 50.14 m, 20.60 m and 145.00 m, respectively. The valve house cavern of size 13.15 m × 7.0 m × 145.0 m, is located at a distance of 15.50 m on the right side of the machine hall cavity. The collection gallery of size 10.6 m × 10.80 m × 173.0 m is located at a distance of 18.70 m on the right side of the machine hall cavity. The transformer hall cavity is of dimension 23.50 m × 20.0 m × 173.0 m and is located at a distance of 45.0 m on the right (extreme) side of the Machine hall. The rock mass comprises of horizontal and vertical brecciated rock horizons, at several locations, in the main Amygdaloidal Basalt rock.

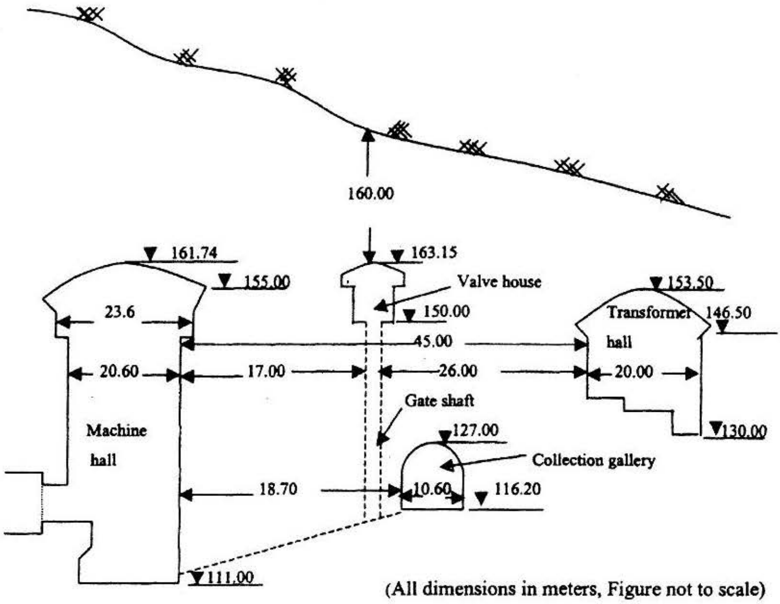

FIGURE 1 : Details of the Opening at Koyna Hydro Power Project

The Volcanic Breccia occurring in between the compact basalt generally exhibits undulating top surface. It also contains some red tactylyte at several locations. The average overburden thickness is about 160 m.

In-Situ Investigations

Investigations have been conducted to determine in-situ stress, modulus of deformation, and compressive and tensile stresses in the rock mass (Specific note No. 2873). Flat jack tests (IS:7292-1974) were conducted to determine in-situ stress and modulus of deformation at 12 locations as shown in Fig.2, 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_θ, tangential to the boundary of the opening and the slot cut vertically would yield stress, P_H, parallel to the axis of the openings at the respective test locations. The results obtained are described in Table 1. The average vertical (σ_V) and horizontal (σ_h) in-situ stresses are found to be 6.86 and 4.80 MPa, respectively. The modulus of deformation, for the rock, has been determined from the stress displacement curves depicted in Fig.3.

Laboratory tests have been conducted (Specific note No. 3035) on NX size cylindrical rock samples, collected from different zones of underground openings, to evaluate mechanical properties (viz., density, static modulus of elasticity, unconfined compressive strength, tensile strength, Poisson's ratio,

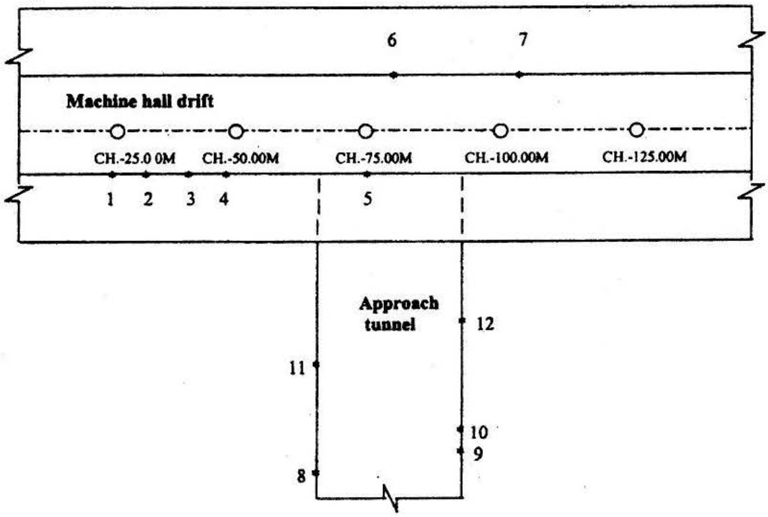

FIGURE 2 : Plan showing Flat Jack Test Locations

TABLE 1 : Flat Jack Test Results for Koyna Hydroelectric Project

Location	Chainage in m	Test Point	Directions	E_M (GPa)	Induced Stresses (MPa)	
					P_0	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	–

cohesion and angle of friction) of the Amygdaloidal Basalt and Breccia. These properties are given in Table 2 and have been used for finite element analysis.

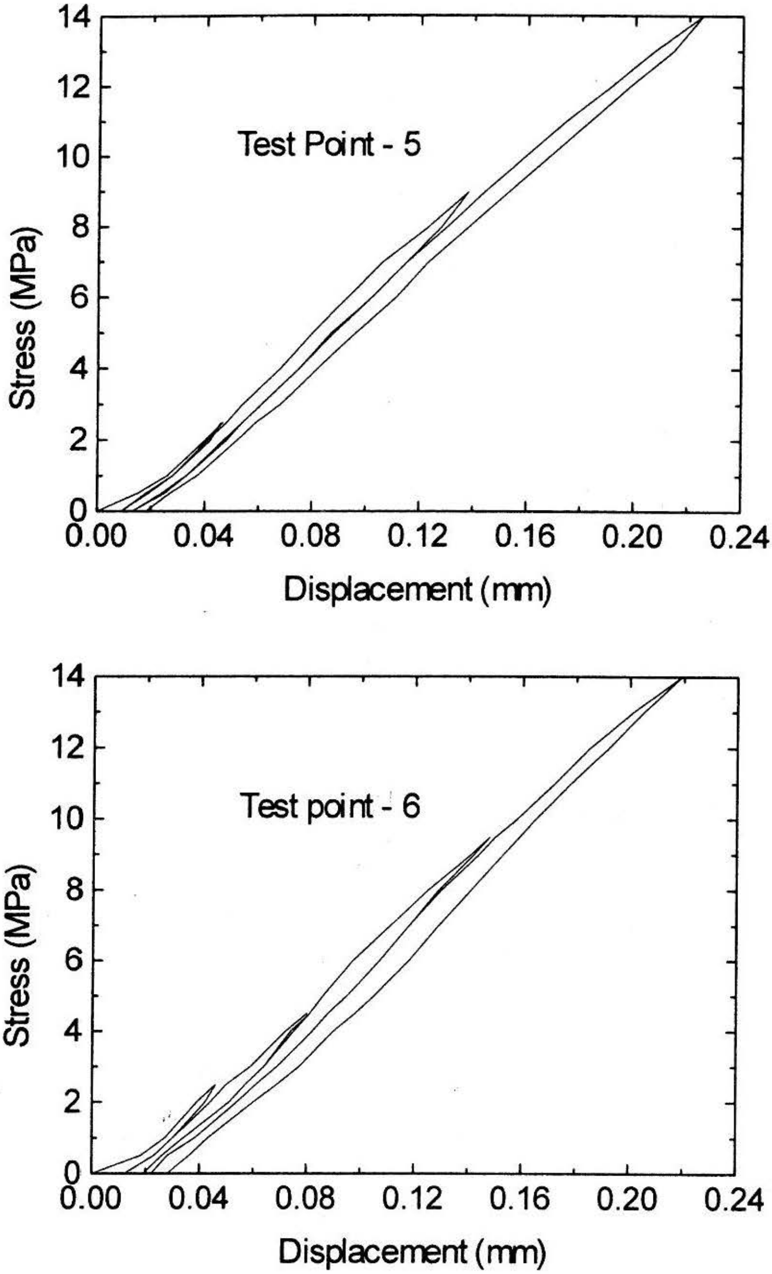

FIGURE 3 : Stress-Strain Behaviour of Rock Mass b Flat Jack Method for Test Points 5 and 6

TABLE 2 : The Rock Mass Properties Used for the FEM Analysis

Element Group	Linear Elastic Analysis		Non-Linear Elastic Analysis: E_m (GPa) at stress (MPa)						Elasto-Plastic Analysis	
Element Group	E_m (GPa)	ν	2.0	4.0	6.0	8.0	10.0	12.0	c (MPa)	ϕ (°)
1	16.86	0.18	—	—	—	—	—	16.86	—	—
2	5.20	0.13	—	—	—	—	—	5.20	—	—
3	16.86	0.18	—	—	—	—	—	16.86	—	—
4	8.73	0.13	8.13	8.13	8.33	8.33	8.73	8.73	0.3	36
5	22.06	0.18	15.49	17.90	20.20	21.57	22.06	22.06	0.6	41
6	8.73	0.13	8.13	8.13	8.33	8.33	8.73	8.73	0.3	36
7	22.06	0.18	15.49	17.90	20.20	21.57	21.77	22.06	0.6	41
8	22.06	0.18	15.49	17.90	20.20	21.53	21.77	22.06	0.6	41
9	5.20	0.13	4.31	4.71	4.90	5.00	5.20	5.20	0.2	35
10	18.60	0.18	2.73	20.40	20.30	19.60	18.60	18.60	0.5	40
11	8.73	0.13	8.13	8.13	8.33	8.33	8.73	8.73	0.3	36
12	8.73	0.13	8.13	8.13	8.33	8.33.	8.73	8.73	0.3	36
13	18.60	0.18	2.73	20.40	20.30	19.60	18.60	18.60	0.6	41
14	8.73	0.13	8.13	8.13	8.33	8.33	8.73	8.73	0.3	36
15	18.60	0.18	2.73	20.40	20.30	19.60	18.60	18.60	0.6	41
16	8.73	0.13	8.13	8.13	8.33	8.33	8.73	8.73	0.5	40
17	18.60	0.18	2.73	20.40	20.30	19.60	18.60	18.60	0.6	41
18	8.73	0.13	—	—	—	—	—	8.73	—	—
19	18.60	0.18	—	—	—	—	—	18.60	—	—
20	8.73	0.13	—	—	—	—	—	8.73	—	—
21	8.73	0.13	—	—	—	—	—	8.73	—	—
22	18.60	0.18	—	—	—	—	—	18.60	—	—
23	18.60	0.18	—	—	—	—	—	18.60	—	—

Models for Rock Mass Behaviour

The intact rock may deform elastically, or may undergo significant plastic deformation, causing joints to open, close or slip, resulting in instability of an opening. Three different types of material behaviour have been adopted for describing the rock properties, (viz., linear elastic, non-linear elastic and elasto-plastic). The relative performance of the FEM solutions incorporating these rock behaviours has been compared vis-à-vis the field deformation data of the rock mass. The observed and calculated displacements are compared for the same loading conditions.

Linear Elastic Analysis

In the linear elastic behaviour, the displacements are considered to be proportional to the applied load. Due to the assumed material isotropy, the stiffness can be treated as a function of material properties (like modulus of elasticity, E) and the geometry of the structure. For the plane-strain condition, the constitutive equations can be expressed in terms of the plane components of the stress and strain tensors (Hinton and Owen, 1977).

Non-linear Elastic Analysis

In the linear elastic analysis, the stress-strain relationship is normally considered as a straight line. But, in reality, the behaviour of rock mass is never found to be linear even within elastic limits (with application of small loads). Therefore, to get accurate estimate of the elastic deformations it is essential to consider the actual non-linear stress-strain relationship. Due to this, the equations of static equilibrium depicting non-linear form are required to be solved by suitable iterative procedure. However, for the sake of simplicity, in the present study the load is incremented from zero to its final value in six small steps (2.0 to 12.0 MPa) as depicted in Table 2.The stress-strain relationship is approximated to be linear in each load increment. (i.e. a piecewise linearization of non-linear behaviour of rock mass has been adopted).

Elasto-plastic Analysis

In the elasto-plastic material behaviour, it is necessary to define a yield criterion, which specifies the onset of plastic flow, and the stress-strain relationship for which an appropriate flow rule has to be used to determine the plastic strain. In the present study, it is assumed that the rock mass obeys Drucker-Prager yield criterion:

$$(J_2')^{1/2} + \alpha J_1 - k = 0 \qquad (1)$$

where, J_1 and J_2' are the stress invariants and constants α and k are defined as:

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)}, \qquad k = \frac{6c\sin\phi}{\sqrt{3}(3-\sin\phi)} \tag{2}$$

where, c and ϕ are the cohesion and the angle of friction, respectively.

FEM Analysis for Underground Openings

Solvia 90, of Solvia Engineering AB, Sweden has been employed for analysis of the openings in the rock mass. 2-D FEM analysis is carried out using 2-D solid plane strain eight noded isoparametric elements. In the Finite Element method (FEM), the continuum rock mass is discretised suitably and the faults and shear zones in the rock mass are represented by an equivalent continuum material properties that approximately reflect their influence on the deformation and the strength of the rock mass as a whole. Depending upon the characteristic properties, the continuum is discretised into 23 zones, each representing a group of elements, which are connected to each other through their nodes at the interface. As depicted in Fig.4, the rock mass of 300 m × 360 m, has been discretised with the help of 1420 nodes and 467

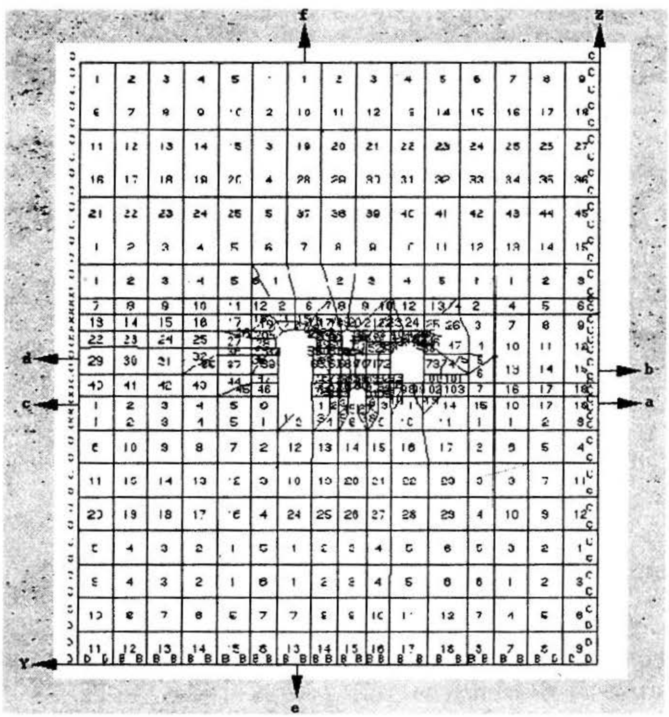

FIGURE 4 : The Discretisation of the Rock Mass with Openings

elements (divided into 23 element groups). The directions a to f indicated by arrows represent the various directions along which the stress factors are plotted. On the two vertical boundaries of the rock mass, Z direction, only vertical deformation is permitted and that on the base of the rock mass only horizontal deformation (Y direction) is permitted, whereas at the bottom corners of the rock mass both horizontal and vertical deformations are assumed to be zero. The adopted values for rock mass parameters for these element groups are presented in Table 2. The density of rock mass has been taken as 26.5 kN/m^3.

Since the openings are located at an overall depth of about 160 m and far away from the hill slope, the directions of the principal stresses are assumed to be vertical and horizontal at the center of the openings. Stresses in the direction normal to plane under consideration have been ignored, as their influence would be negligible (Hojem and Cook, 1968). The effect of other small openings in the rock mass such as openings for bus duct passage, penstock, tailrace tunnel etc. is localised and as such has been ignored. These openings are excavated sequentially in the following five steps:

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

Step 2: Excavation from top of the opening has progressed to 23%,

Step 3: Excavation from top of the opening has progressed to 74%,

Step 4: Excavation from top of the opening has progressed to 95%, and

Step 5: Excavation from top of the opening has progressed to 100%.

For realizing the openings, the elimination of a certain set of elements is considered at different steps. Various excavation steps have been simulated by a sequence of calculative steps in which the currently investigated excavation step is always the starting point.

For linearly elastic rock mass, the entire load has been applied in one step for each of the excavation steps. There will not be any changes in the deformations and stresses due to excavation of the openings, in one step or four steps, for elastic cases.

For non-linear elastic behaviour, the stress-strain curve is approximated by several linear segments, which account for the non-linearity. Each segment is described by different material constants. The rock mass properties used for curve description models are the material density, γ, Poisson's ratio, ν, elastic modulus, E_m, volumetric strain, ε_ν, bulk modulus, and shear modulus, G are given in Table 2. Normal stiffness reduction factor, k_n = 0.0001, shear stiffness reduction factor, K_s = 0.000099 are adopted (Solvia-Post 90, Users Manual for Stress Analysis).

When stresses in the rock mass due to creation of the openings exceed the in-situ stresses and are more than the strength of rock mass, then plastic zones develop in the surrounding areas of openings. When the tangential stresses around an opening are greater than about one-half the compressive strength, cracks may begin to form and plastic analysis is required to be conducted. The rock mass properties used for carrying out the elasto-plastic analysis are the cohesion, c, angle of internal friction, ϕ, elastic modulus, E_m, Poisson's ratio, ν, yield function parameters, α and k (Table 2).

Results and Discussion

Table 3 presents the results of nodal deformations for 10 nodal points at which the field deformation results are available. These locations along with the node numbers (used in FEM discretization) are depicted in Fig.5. Table 3 also presents the results for the non-linear elastic and elasto-plastic analysis. From the table it can be seen that the linear elastic analysis, for the Machine Hall and Transformer Hall, yields displacements for the nodes, which are approximately 68 to 77% less as compared to the values recorded in the field. The field deformations have been suitably corrected to account for the deformations caused due to the delay in installation of extensometers after making the openings. It may be noted that there was a time lag varying from

TABLE 3 : Deformation (cm) of the Machine and Transformer Hall Openings

Node	Depth of the Node (m)	Type of the Analysis			Corrected Extensometer Values
		Linear Elastic	Non-linear Elastic	Elasto-Plastic	
		Machine Hall			
105	25.00	0.55	0.59	1.08	1.88
120	10.00	0.63	0.67	1.11	2.13
141	5.00	0.63	0.79	1.27	2.62
142	0.00	0.79	0.95	1.43	2.65
106	25.00	0.60	0.71	1.40	1.92
121	10.00	0.63	0.67	1.41	2.25
144	5.00	0.67	0.71	1.40	2.53
143	0.0	0.63	0.90	1.46	2.74
		Transformer Hall			
159	3.00	0.78	0.95	2.10	2.97
178	0.00	0.93	1.11	2.15	3.23

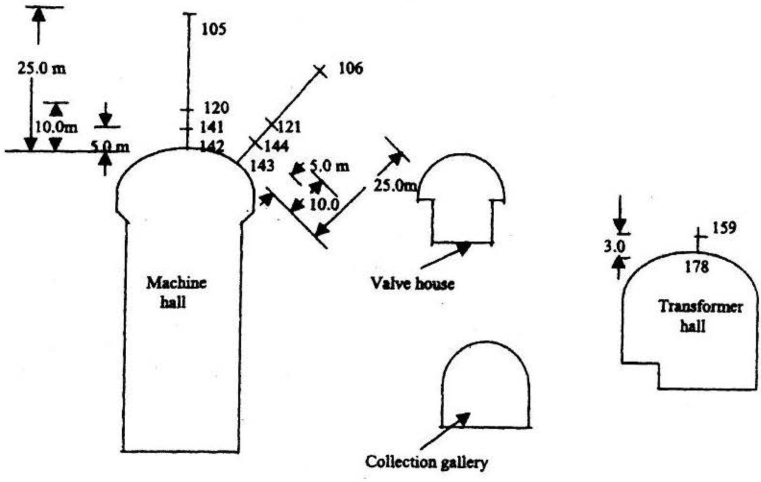

FIGURE 5 : Location of Various Nodes

three to eleven months between the excavation of openings and installation of the extensometers. To account for this, the deformations, which might have occurred during the delay of installation of extensometers, have been calculated with the help of Q-method, which takes into account the size of openings and overburden for correcting the field observations (Barton et al., 1974).

From Table 3 it can be noticed that for non-linear elastic analysis, the under estimation of the results of nodal displacements vary between 63 and 71% of the field extensometer results. However, when elasto-plastic analysis is used, the same is improved and varies from 27 to 51% only. From the deformations obtained by different analyses, it is seen that rock mass exhibits elasto-plastic behaviour and are closer to field extensometer results.

For linear elastic analysis, the maximum and minimum values of major principal stresses have been calculated, corresponding to each excavation step, and the obtained results are presented in Table 4. The results indicate that when compared to the in-situ stress condition (i.e. Step 1), the maximum value of major principal stress increases by about 22% in Step 2 and practically remains constant for Steps 3, 4 and 5. However, the tensile stresses are noticed to increase progressively from 0.95 to 1.84 MPa, for excavation Steps 2 to 5. The variation in stresses may be attributed to the change in the shape and size of the openings at each step. However, the maximum stress is noticed to occur at Step 2 of the excavation process (where the openings have been just created) and may be attributed to the presence of sharp corners. However, in subsequent excavation steps an increase in size of the opening does not exhibit any significant effect on the stresses.

TABLE 4 : Maximum and Minimum Values of the Major Principal Stresses (Mpa) due to Self Weight of the Rock Mass (In-situ Stress Ratio = 0.70)

Excavation Step	Major Principal Stresses (MPa) for the type of analysis as					
	Linear Elastic		Non-Linear Elastic		Non-Linear Plastic	
	Max.	Min.	Max.	Min.	Max.	Min.
1	14.63	0.14	14.67	0.14	15.23	0.15
2	17.81	−0.95	16.37	0.69	20.94	−2.04
3	14.61	−1.00	14.67	−0.67	18.49	−1.23
4	14.65	−1.29	14.69	−0.79	22.02	−2.20
5	14.67	−1.84	14.70	−1.27	16.46	−1.53

For non-linear elastic behaviour, the obtained distribution of major principal stresses (presented in Table 4) is noticed to be similar to that obtained for linear elastic analysis. However, the obtained results improve significantly when elasto-plastic behaviour is used and when compared with the in-situ stress condition (Step 1), the maximum major principal stress is found to vary by approximately 37%, 21%, 44% and 8 %, for Steps 2, 3, 4 and 5 respectively. It can be noticed that similar to the elastic analyses, the maximum major principal stress increases significantly for Step 2. This may be attributed to the excavation of top portion of the Machine and Transformer halls, and Valve house (which have sharp corners and as such exhibit unfavourable effect on the stress). However, Step 3 corresponds to excavation in the vertical direction, in a rectangular shape, which helps in reduction of stresses. For Step 4, a substantial increase in the stresses may be attributed mainly to the effect of creation of the Collection gallery in the vicinity of the Machine hall. However, this does not pose any stability problems. For deciding the correct sequence of excavation and appropriate distance between the openings, further investigations are being made. Even after completing Step 5, the stresses are found to be a little higher than the in-situ stress conditions. This may be due to the plastic behaviour of the rock mass.

Major principal yield stress (σ_1) and Minor principal yield stress (σ_3), are plotted after complete excavation of all the four openings, i.e. at step-5, along six directions around the Machine hall. These directions designated as (a) to (f) are respectively along the bottom right side, central right side, bottom left side, central left side, vertical downward and vertical upward elements of the Machine hall (location shown in Fig.4). The stresses are plotted as ratios of vertical insitu stress (σ_v), as σ_1/σ_v , and σ_3/σ_v , verses ratio r/a of distance from center of Machine hall (r) and half span of Machine

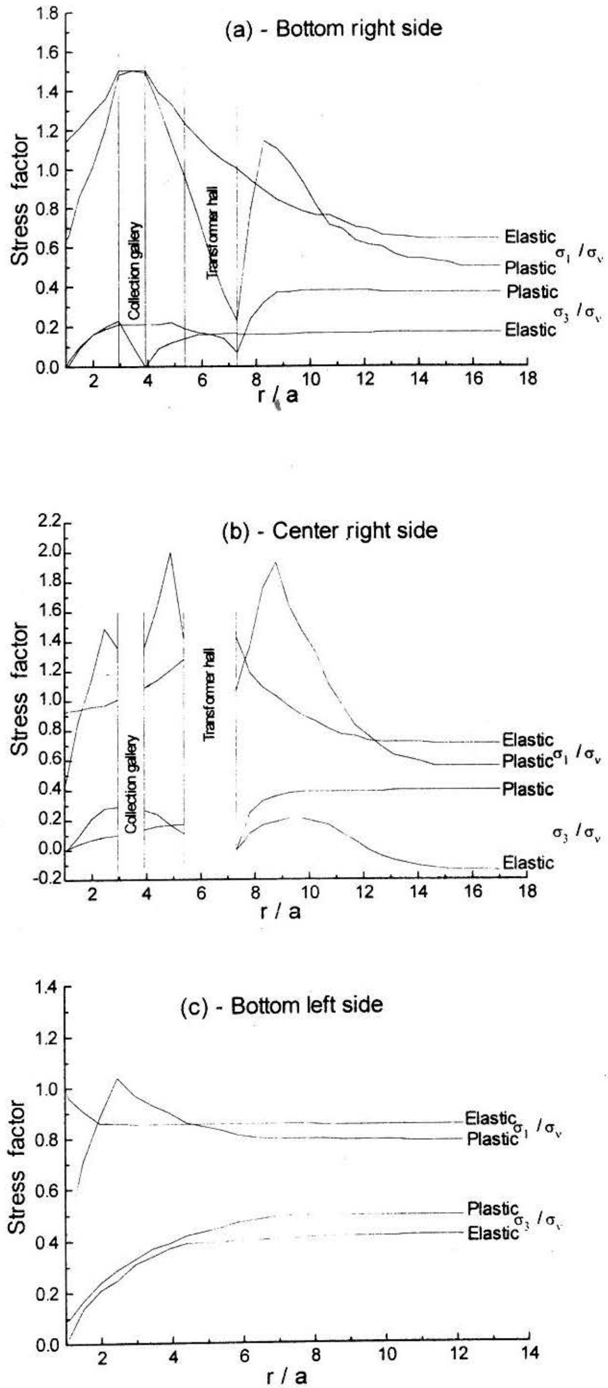

FIGURE 6 : Stress Distribution Around the Machine Hall Opening

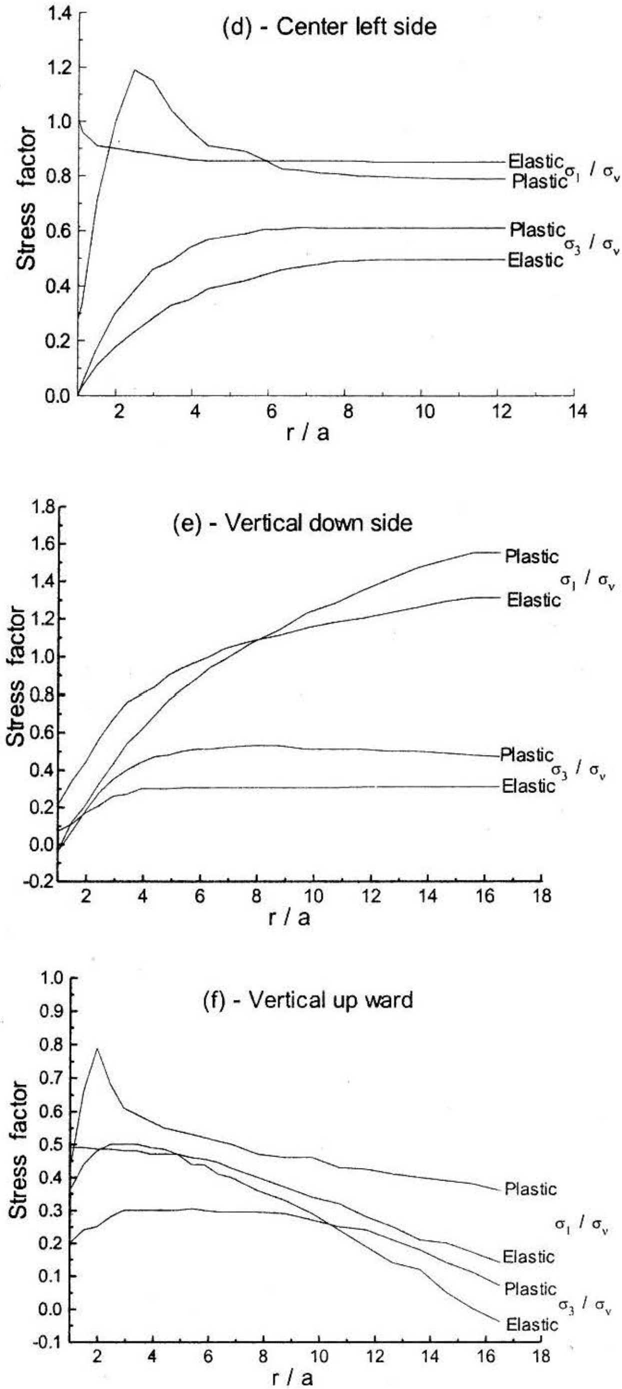

FIGURE 6 : Continued

hall (a), termed as distance factor. The results of the two stress ratios for the six directions described above are shown in Figs.6(a) to 6(f), where Figs.6(a) and 6(b) are for right side of Machine hall, Figs.6(c) and 6(d) are for left side of Machine hall and Figs.6(e) and 6(f) are for vertical downside and upside of the Machine hall. The stress factors in these figures are plotted for elastic and elasto-plastic behaviour only, because the results for the elastic analysis were found to be similar to the non-linear elastic analysis.

From Figs.6(a) to 6(f), it is seen that the stress factor σ_1/σ_v, near the boundary of the Machine hall is considerably less for elasto-plastic analysis as compared to the elastic analysis. The zone of influence for the same stress factor is spread over a larger area in elasto-plastic analysis as compared to the elastic analysis. However, along the bottom right side and center right side directions, the effects of encountering other openings; viz., Collection gallery and Transformer hall, on the behaviour of stress distribution is manifested in a more realistic way in the elasto-plastic material behaviour. The large variations in the stress factor due to the presence of different openings are represented more accurately in the elasto-plastic analysis. Also, along the other four directions considered for plotting the stress factors where no other openings were made, the stress factors near the surface of the Machine hall opening are seen to be quite different for the elasto-plastic analysis, particularly for the stress ratio σ_1/σ_v. The stress factors are attaining their peak values at a distance factor of 2.0 to 2.5. The stress behaviour in the elements just below the Machine hall opening shows that rock mass below the Machine hall has been affected at the surface only due to occurrence of minor tensile stresses, whereas below a certain depth stresses are due to the existing rock mass. Along the bottom and central elements in the right and left and vertical downward direction of the Machine hall, the stress factors are found to stabilize at different distance factors. However, along the vertical upward direction, the stress factor is seen to be continuously decaying due to the reduction in the height of the overburden. The stress factor σ_3/σ_v is found almost zero at the surface of the opening except at the top surface of the Machine hall, which means some support is required to be provided to make the top surface stress free. On different sides of the Machine hall no similarity is found in the values of stress factor and location of peak stresses is also found at different distance factors. It is due to non-symmetrical openings and non-hydraulic in-situ stress conditions.

Conclusions

The major conclusions of the present study can be summarised as follows:

1. Linear elastic, non-linear elastic and elasto-plastic FEM analyses using 2D approximation have been conducted for underground openings at

Koyna H.E. Project, India and the results compared with the field deformation measurements.

2. The elasto-plastic analysis shows the best agreement with the field deformation results, but the results are found to be somewhat lower than the field deformation values. The corresponding results on the major principal stresses can be thus considered the best from the elasto-plastic analysis.

3. The effect of making more openings in the surrounding areas of machine hall can be better understood by elasto-plastic analysis.

4. In the multi-stage excavation, the stresses at the end of a partial stage are higher than those at the end of the complete excavation.

Acknowledgement

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Notation

The following symbols are used in this paper:

σ_1	=	major principal yield stress,
σ_3	=	minor principal yield stress,
σ_v	=	average vertical in-situ stresses,
σ_h	=	average horizontal in-situ stresses,
ϕ	=	angle of internal friction of the material,
γ	=	material density,
ν	=	Poisson's ratio,
ε	=	volumetric strain,
c	=	cohesion of the material,
E	=	modulus of elasticity,
E_m	=	deformation modulus,
E_M	=	static modulus of deformation,
K_n	=	normal stiffness reduction factor,
K_s	=	shear stiffness reduction factor,
P_θ	=	induced stress, tangential to the boundary of the opening,
P_H	=	induced stress, parallel to the axis of the opening,
r	=	distance from center of the Machine hall,
a	=	half span of the Machine hall