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Measurement of In-Situ Stresses and its Application in Rock Engineering

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ABSTRACT

The common applications of in-situ stresses in hydroelectric projects are the design of underground openings, permeability and recharge of aquifers. Only an assessment of stresses in rock will allow the application of rock strength determination for a rational design of excavations in rock. For designing underground opening, measurement of in-situ stresses is very useful. The knowledge of prevailing in-situ stresses helps in the alignment and the design of support system. There are various methods of determining in-situ stresses in the rock mass. When an underground opening is created, stress readjustment occurs due to development of the induced stresses around the opening. So the stresses measured after the excavations are the real stresses affecting its stability. It is experienced that stresses determined by flat jacks give reasonably good results and found economical. Flat jack tests are easy to conduct and interpret. Flat jack test studies have been carried at many underground openings all over India and some results are presented. Determination of rock mass strength is also explained which should always be more than the stresses for self-supported openings. The tunnels which are designed through the rock masses having strength less than the in-situ stresses are to be fully supported by external supports.

Keywords: Flat jack test; Cavern; Underground opening; In-situ stress; Induced stress

1. INTRODUCTION

Stability of underground openings depends upon the stress prevailing in the rock mass. High internal stresses may lead to popping of rock often resulting into heavy rock bursts in strong rock mass. On the other hand, in-situ stresses are useful in the case of water pressure tunnels where it increases tensile strength of rock mass by confining the preexisting cracks. Thus, prior knowledge of in-situ stress condition in rock mass enables the engineers to design for ensuring the safety and stability of underground openings. Due to aforementioned reasons, prior information of in-situ stresses becomes indispensable for designing an underground opening. The present study deals with the measurement methods of in-situ stress and provides measured values for further use of the researchers.

2. ORIGIN OF STRESSES IN ROCK

In the earth crust, state of stress is defined by gravitational, tectonic, residual and thermal stresses. The gravitational force field has the peculiarity that it cannot be separated from the material bodies, which generate it. The tectonic force field is significantly more complex than the gravitational force field and is related to the non-uniform distribution of the rates of tectonic movement and rate of strain of the crust. Development of induced stresses takes place due to stress changes in the manmade excavations.

2.1 Gravitational Component of Stress Field

This includes the analysis of a situation when the stress state of rock mass is governed only by the gravity forces. At any point M (Fig.1) in an elastic homogeneous-isotropic rock mass beneath the surface of earth, the vertical component of the gravitational stress field is equal to the product of mean volumetric weight of the overlying rock mass multiplied by the depth of point (Eq. 1). Under the influence of vertical gravitational force, the element flattens in the vertical direction and tends to expand horizontally. The surrounding rock resists the lateral deformation; as a result, horizontal compressive stresses develop (Goodman, 1988) as shown by Eq. 2.

Vertical stress, $\sigma_3 = \gamma H$ (1)

Horizontal stresses, $\sigma_{1,2} = \gamma H\nu/(1-\nu)$ (2)

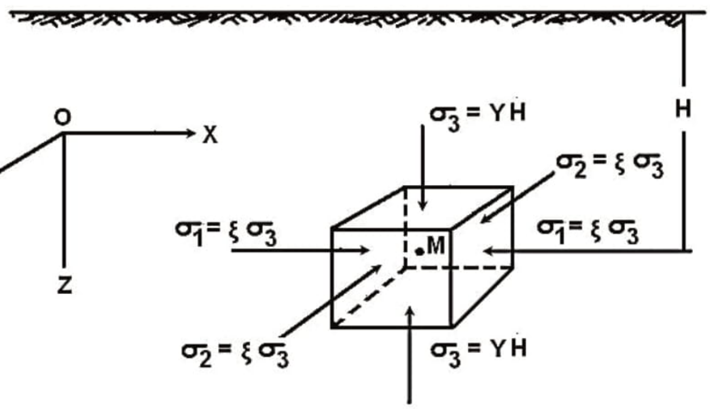

Fig. 1 - Computation of stresses in rock mass (Goodman, 1988)

2.2 Tectonic Component of Stress Field

The tectonic component of stress field results from the continuous redistribution in space of the tectonic movement and crustal deformation rate. The tectonic stress field is much less uniform than the gravitational stress field. Measurements have shown that the tectonic component of total stress field i.e., the active stress generated by lateral compression of crust, can be considered horizontal. Its orientation in plan depends generally on the orientation of tectonic structures/deep fractures in the crust. In many cases, it has been observed that at shallow depths, tectonic stresses are significantly greater than the gravitational stresses.

2.3 Residual Stresses

Residual stresses are self-equilibrating stresses locked in by the rock fabric and outsides of rock fabric are free of stress.

2.4 Thermal Stresses

Thermal stresses are due to cooling or heating of rock. They occur close to the earth's surface due to exposure to the sun or as the result of the heating of the interior of the earth due to radioactivity or other geological processes.

2.5 Induced Stresses

These stresses are result of excavation activity and therefore are of great concern in the underground excavation design.

3. TECHNIQUES FOR MEASUREMENT OF IN-SITU STRESSES

In-situ stresses can be measured in boreholes, on outcrops, and in the walls of underground galleries as well as can be back calculated from displacements measured underground. Three of the best-known and presently most used techniques are hydraulic fracturing, the flat jack method and over coring. These techniques are complementary to each other, each offering different advantages and disadvantages. All stress measurement techniques perturb the rock in order to create a response that can then be measured and analyzed. In the hydraulic fracturing technique, the rock is cracked by pumping water into a borehole, the known tensile strength of the rock and the inferred concentration of stress at the well bore are processed to yield the initial stresses in the plane perpendicular to the borehole. 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 null the displacement that occurs as result of slot cutting. In the over coring test, the rock is completely unloaded by drilling out a sample, while radial displacements or surface strains of the rock are monitored in a central, parallel borehole. Analysis using an unloaded thick walled cylinder model yields stress in the plane perpendicular to the borehole. In each case, stress is inferred, but displacements are actually measured. Precisions are seldom great and the results are usually considered satisfactory if they are internally consistent and yield values believed to be correct to within about 0.3MPa. The main problem associated with all these techniques is that the measurement are conducted in a region that has been disturbed in the process of gaining access for the measurement; this paradox is handled by accounting for the effect of the disturbance in the analytical technique.

The flat jack stresses are measured after the making of the opening whereas the stresses measured by hydraulic fracturing or over coring methods are not in the existing openings. So, the stresses measured by flat jack may be more useful for the design of support system because the measured stresses will be the actual existing stresses developed due to the size, shape and depth and existing tectonic forces. The actual measured stresses will be after the readjustment of the stresses after making of the opening.

3.1 Hydraulic Fracturing Method

The hydraulic fracturing method (ISRM, 1987) makes it possible to estimate the stresses in the rock at considerable depth using borehole.

3.2 Over-Coring Technique

In this method, strains are measured while causing a change in the stress field and then relate deformation properties to stress. The deformations measured are related to the initial stresses (ISRM, 1987).

3.3 Flat Jack Method

The present study deals with flat jack test for in-situ stress measurements. The methods described above are suitable for measuring stress either before or after the opening is made in a zone. Stress measurement using this method is conducted on the walls of the underground openings thus measuring the induced stresses from which in-situ stresses are deduced. While carrying out geological investigations, access is required to be made to the area of interest in the form of drifts or galleries. These openings can be very well used for carrying out stress measurements by this method.

Flat jack method is also known as stress compensation method. It does not require any special or costly equipment training and can be performed rather very quickly. The reliability of results is high and more number of tests can be performed in the given location within the same amount of time and money in order to achieve a higher degree of confidence. The method involves the use of flat hydraulic jacks, consisting of two steel plates welded around their edges and a nipple for introducing oil into the intervening space (Fig. 3). A flat jack, a deformeter, reference pins, a standard distance bar and a hydraulic pump with pressure gauges are the main equipments required for the test (BIS, 7292-1974).

The deformeter consists of a demountable mechanical dial gauge of sensitivity either 0.001mm. Steel pins fixed at two reference points or gauge points on both sides of the slot for a distance of about 25.4 cm with grove at one end to accommodate tip of deformeter. These pins are used to measure the change in distance between the gauge points either after slot is made or during the pressurization of the embedded flat jack in the slot. The flat jack, deformeter, reference pins and standard distance bar is shown vide Fig. 2.

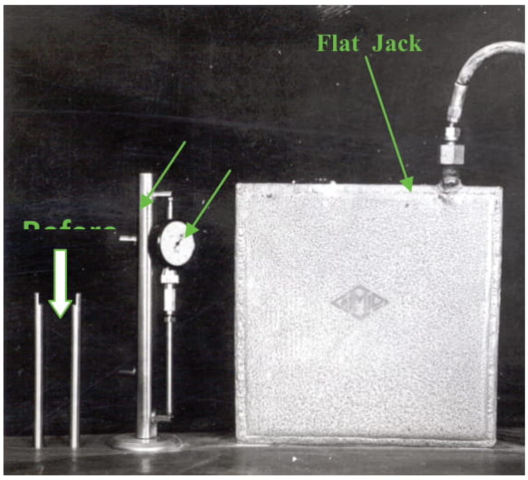

Fig. 2 - Equipments used in flat jack test

The separation of the points is typically 25.40 cm but must conform to the gauge length of available extensometers. Then a deep slot is extracted perpendicular to the rock face between the reference points (Fig.3);

A serious limitation of the method is that the measured stress lies in the region of disturbance of the gallery introduced for the purpose of taking the measurement. If the gallery is carefully executed, this disturbance might be calculated by conducting an independent stress concentration investigation. In general, if the stresses normal to the plane of the jack are determined at three points around the section of the gallery, yielding values for the surface of the opening near the surface at these points, the initial stresses in the plane perpendicular to the gallery can be calculated (ISRM, 1979). The measured in-situ stresses at some important projects are given in Table 1.

3.3.1 Flat jack test calculations

The flat jack provides the stresses at the surface of rock, which are affected by the stress concentration and are called, induced stress which are a function of primary stresses and shape of the opening (Herget, 1988).The method consists of cutting a thin slot approximately elliptical in shape into the rock surface by drilling a series of overlapping holes or by using a special diamond saw. A view of the slot in the horizontal and vertical direction cut by jack hammer drilling at the rock surface is shown vide Figs. 3 and 4.

Fig. 3 - Slot cut in horizontal direction to measure vertical induced stress

Fig. 4 - Slot cut in vertical direction to measure induced stress along the tunnel axis

This process relieves the rock surface of the stress originally existing across it. Because of the stress relief, the sides of the slot converge. The amount of convergence, which depends upon the stresses in rock and its elastic properties given by the Eq. 3 (BIS, 7292-1974),

$$E_m = \frac{2PC_o}{w}\left[(1-\nu)\left(a_o + \frac{Y}{C_o}\right) + \frac{1+\nu}{a_o}\right] \tag{3}$$

where $a_o = \sqrt{1 + \frac{Y^2}{c_o^2}}$

w = amount of convergence between two points spaced at equal distance Y,
$2c_o$ = length of flat jack,
P = pressure applied,
E_m = elastic modulus of rock and
υ = Poisson's ratio.

The convergence of the slot is measured from the difference in present reading between the two points across the slot fixed in the rock and that of the reading prior to cutting the slot. A view of the measurement of distance by deformeter between two references pins after the slot is made is shown in Fig. 5. The flat jack is then embedded tightly in the slot by grouting or mortar, filling the gap between the jack and the slot. View of Flat jacks embedded in slot in rock mass both in horizontal and vertical position to measure stresses tangential and parallel to the axis of the opening are shown vide Fig. 6.

The jack is then pressurized by a hydraulic pump until the displacement that took place after the slot is cut, is cancelled. The cancellation pressure is then very nearly equal to the pressure existed in the normal to the plane of slot before the slot is cut, provided the length of the slot and the jack are same. A view of Flat Jack test in progress is shown vide Fig. 7.

However, in actual tests, the length of the slot may be bigger than the jack size and the slot may not be of uniform width. Further, the stress acting in the plane parallel to the major axis of the slot also affects the contraction of the slot. The Eq. 4 as given below that accounts for the effect of all the above can be used for evaluating the stresses (BIS, 7292-1974).

$$PK_1 = P_\theta K_2 + P_h K_3 \tag{4}$$

where P is the flat jack cancellation pressure, P_θ is the induced stress tangential to the boundary of the opening and P_h is the induced stress parallel to the axis of the opening.

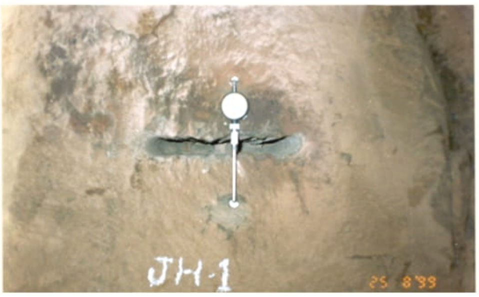

Fig. 5 - Measurement of distance between reference pins by deformeter

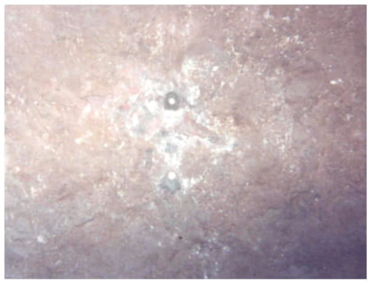

Fig. 6 - Flat jacks embedded in the slot by cement mortar

Fig.7 - Flat jack test in progress

To account for the effect of slot size, constants K_1, K_2, K_3 which are determined using BIS:7292-1974 and are expressed by Eqs. 5, 6 and 7 as follows:

$$K_1 = C_o[(1-\nu)\left(a_o - \frac{Y}{C_o}\right) + \frac{1+\nu}{a_o}] \tag{5}$$

$$K_2 = (C + Y_o)\left(\frac{1+\nu}{a}\right) + \left(a - \frac{Y}{C}\right)[C(1-\nu) - 2\nu Y_o] \tag{6}$$

$$K_3 = Y_o[(2\nu)\left(a - \frac{Y}{C}\right) - \frac{1+\nu}{a}] \tag{7}$$

where $a_o = \sqrt{1+\frac{Y^2}{C_o^{\,2}}}$ *and* $a = \sqrt{1+\frac{Y^2}{C^2}}$, $2C_o$ is length of flat jack, ν is Poisson's ratio, $2C$ is length of slot, $2Y$ is distance between two reference pins and $2Y_o$ is width of slot.

Stresses induced on the boundary of the opening are P_θ and P_h. Since these induced stresses are a function of the in-situ stresses and the shape of the opening, it is possible to evaluate the in-situ stresses from Eqs. 8, 9 and 10 (BIS, 7292-1974).

$$P_\theta = K_v\sigma_v + K_h\sigma_{h1} \quad (8)$$

$$P_h = \sigma_{h2} + \nu(K_v - 1)\sigma_v + \nu(K_h - 1)\sigma_{h1} \quad (9)$$

$$P_L = \sigma_{h1} + \nu(K_v - 1)\sigma_v + \nu(K_h - 1)\sigma_{h2} \quad (10)$$

Stress on the boundary of the opening and tangential to the boundary is P_θ, Stress on the boundary of the opening and parallel to the axis of the opening is P_h, Stress on the boundary and perpendicular to the axis of the opening is P_L, Stress concentration factor due to in-situ vertical stress is K_v, and Stress concentration factor due to in-situ horizontal stress is K_h.

With the knowledge of measured P_θ, P_h and P_L values and stress concentration factors, it is possible to evaluate σ_v and σ_h. For regular geometrical shape such as circle, *D* shape, rectangle and square shapes, stress concentration factors can be obtained from available monographs. For other shapes mathematical models could obtain the same. In case where it is not possible to measure both P_h and P_L, the horizontal stresses σ_{h1} and σ_{h2} are assumed to be nearly equal and σ_v and σ_h are evaluated using Eqs. 8 and 9

For circular openings, it is possible to estimate the magnitude of the principal stresses σ_v and σ_h in a horizontal plane using the Eq. 11 (Goodman, 1988).

$$\begin{Bmatrix} P_\theta \\ P_h \end{Bmatrix} = \begin{bmatrix} -1 & 3 \\ 3 & -1 \end{bmatrix} \begin{Bmatrix} \sigma_h \\ \sigma_v \end{Bmatrix} \quad (11)$$

3.3.2 Merits and demerits of the method and results

The method is simple and many tests can be performed easily and at low cost. Knowledge of elastic properties of rock is not absolutely necessary for stress evaluation. Further, it can be seen that elastic properties E_m and υ, if required can be determined by the test.

The stresses using flat jack are measured after creation of the opening. Hence, the measured stresses may be more useful for the designs of support system. Thus obtained value of stresses can also be used for the numerical analysis for the stability analysis of the underground openings.

The limitations are that since the tests have to be made near the surface, the in-situ stresses unaffected by the excavation of the tunnel cannot be measured directly. Besides the measured stresses may not be the principal stresses. But by careful site selection and use of correction factors to account for slot and flat jack mismatch and the rigidity of the flat jack, it is possible to obtain reasonably accurate results as given in Table 1.

4. STRENGTH OF ROCK MASS

Evaluation of strength of rock mass is dependent on many factors. By combining the strength of rock material and discontinuities and their respective influence on the properties, an empirical approach (Hoek and Brown, 1980) has been adopted to find the strength of rock mass. Rock mass classification parameters based on rock mass rating (RMR), (Bieniawaski, 1984) is also used to assess some of the parameters of the empirical approach. The generalized Hoek and Brown failure criterion for jointed rock masses is defined by relation in Eq. 12:

Table1 - Results of in-situ stress measurements by flat jack method

CWPRS Report	Project	Rock type	H in m (σ_v in MPa)	σ_{vi} (MPa)	σ_{hi} (MPa)	σ_{hi}/σ_{vi} (K)
1888 (1979)	Kalinadi H.E Project Karnataka	Greywacke	92 (2.48) 96 (2.59) 84 (2.27) 205 (5.54)	11.4 17.7 6.0 4.95	13.4 15.9 7.2 6.14	1.18 0.90 1.20 1.24
2360 (1987)	Idamalayar project,T. N	Granite Gneiss	70 (1.89) 50 (1.35)	8.5 8.3	12.3 8.8	1.45 1.06
2429 (1987)	Kadamparai Project,T. N	Granite Gneiss	180 (4.86) 125 (3.38)	18.0 11.9	23.0 20.1	1.28 1.69
2555 (1988)	Kuttiyadi Tunnel, Kerala	Gneiss	50 (1.35) 230 (6.21) 28 (0.76)	9.0 8.6 5.7	8.2 7.8 8.4	0.91 0.91 1.47
2558 (1988)	Lower Periyar, kerala	Charnokite Gneiss	131 (3.54) 121 (3.27) 70 (1.89)	10.4 5.1 3.9	5.6 3.8 2.2	0.54 0.75 0.56
2696 (1990)	Varahi H.E.Project Karnatka	Granite Gneiss	80 (2.16) 260 (7.02) 230 (6.21)	15.3 17.7 18.1	11.6 14.6 10.3	0.76 0.82 0.57
2874 (1991)	Kakkad H. E. Project, Kerla	Hard Charnokite	34 (0.92)	3.20	2.40	0.75
2936 (1992)	Bhira H.E Station, Maharashtra	Amygdo-loidal Basalt	46 (1.24)	5.0	5.0	1.0
2983 (1992)	Srisailam H.E.Project A.P.	Quartzite and Shale	64 (1.73) 91 (2.46) 100 (2.70)	2.3 2.5 2.8	3.5 1.6 1.7	1.52 0.64 0.61
3447 (1997)	Indira Sagar Pproject, M.P	Quartzite and fine grained sand stone	40 (1.08) 55 (1.49)	4.32 4.38	6.53 4.60	1.30 1.05
3464 (1997)	Koyna H.E. Project Stage IV Maharashtra	Basalt inte-rmixed with volcanic Breccia	145 (3.92) 77 (2.08) 57 (1.54) 101 (2.73)	7.30 1.30 1.80 0.60	4.70 0.90 1.70 0.25	0.64 0.70 0.90 0.40
3973 (2003)	Ghatghar H.E. Project, Maharashtra	Basalt with porphyritic texture	35 (0.95)	4.10	5.60	1.36

σ_{vi} - induced vertical in-situ stress; σ_{hi} - induced horizontal in-situ stress

$$\sigma_1 = \sigma_3 + \sigma_C \left[m_b \frac{\sigma_3}{\sigma_1} + s\right]^a \tag{12}$$

where σ_1 and σ_3 are the maximum and minimum effective stresses at failure, m_b is the value of the Hoek and Brown constant for the jointed rock mass, s and a are constants, which depend upon the characteristics of the rock mass, and σ_c is the uniaxial compressive strength of the intact rock pieces.

The strength of jointed rock mass depends on the properties of the intact rock pieces and also upon the freedom of these pieces to slide and rotate under different stress conditions. The geometrical shape of the intact rock pieces as well as the condition of the surfaces separating the pieces control this freedom. Angular rock pieces with clean, rough discontinuity surface will result in a much stronger rock mass than one which contains rounded particles surrounded by weathered and altered material. The Geological Strength Index, (GSI) introduced by (Hoek et. al., 1998) provides a system for estimating the reduction in rock mass strength for different geological conditions. Once the GSI, has been estimated, the parameters which describe the rock mass strength characteristics, are calculated as explained in equations (Hoek et al., 1998):

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \tag{13}$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \tag{14}$$

$$a = 0.65 - \frac{GSI}{200} \tag{15}$$

The unconfined strength of rock mass is given by,

$$\sigma_1 = \sqrt{s\sigma_c^2} \qquad (i.e, \sigma_3 = 0)$$

The basic aim of any underground excavation design is to utilize the rock itself as the principal structural material, creating as little as possible disturbance due to the excavation process. The stress field in the rock mass is disturbed by the creation of an underground opening, and in some cases, this disturbance induce stresses which are high enough to exceed the strength of the rock mass. In such cases, failure of the rock mass adjacent to the excavation boundary can lead to instability, which may take the form of gradual closure of the excavation. In the context of stress analysis applied to rock engineering problems, it is useful to have a complete strength envelope for the rock mass in terms of minor (σ_3) and major (σ_1) principal stresses. The stability of an underground opening depends upon the behavior of the entire rock mass surrounding the opening. In considering the behavior of different systems in transition between intact rock and a heavily jointed rock mass, the quantity and quality of experimental data decrease rapidly as one move from the intact rock samples to the rock mass. Experimental difficulties increase significantly in tests on specimens containing one set of discontinuities and become very serious when two or more sets of discontinuities are present. Full-scale tests on heavily jointed rock masses are extremely difficult because of the experimental problems of preparing and loading the samples and are very expensive because of very scale of the operation. Taking all these factors into account, an empirical criterion, developed by (Hoek and Brown, 1980 and 1997) has been used to determine the strength of rock mass at Koyna H.E. Project site.

The rock parameters have been determined by using CWPRS specific note Nos. 2873 & 3035 and Hoek and Brown relations, required for determining the strength of the amygdaloidal basalt and breccia rock mass (Table 2).

Table 2 - Rock properties for the rock mass at Koyna H.E. Project

Rock Properties	***Type of Rock***	
	Amygdaloidal Basalt	Breccia
Intact rock material strength, σ_c (MPa)	60.0	29.9
Hoek and Brown Constant, m_i	20	18
Geological Strength Index, GSI	55	44

The average vertical σ_v and horizontal σ_h stresses are found to be 7.30 and 4.70 MPa, respectively. As the underground openings are located at a very higher depth, σ_v is assumed as σ_v and σ_h is assumed as σ_3. Strength of rock mass at σ_3 = 4.70MPa for amygdaloidal basalt is 38.74MPa and for breccia is 23.50MPa, which is found more than the in-situ stresses. So in the Head race tunnel no supports have been provided.

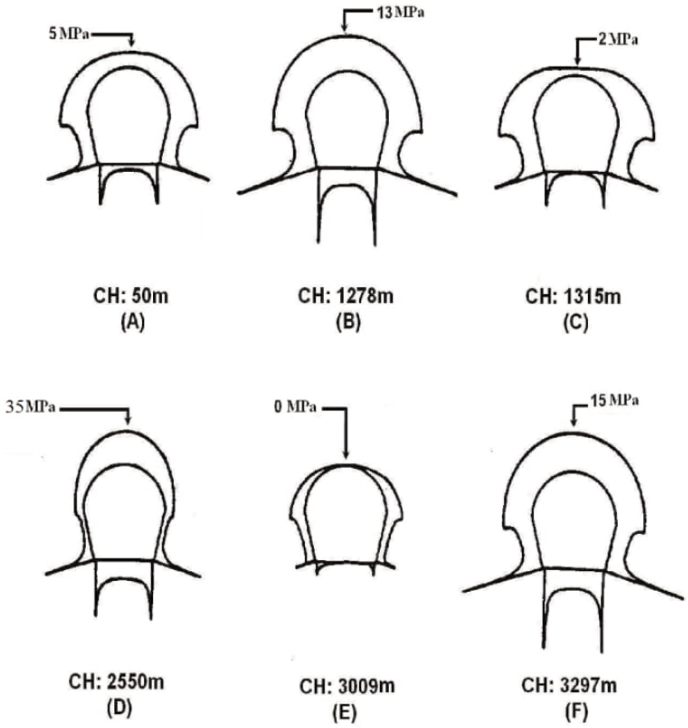

Fig. 8 - Measured induced stresses around Head Race Tunnel at Koyna Stage-IV H.E. Project (Not to scale)

5. APPLICATIONS OF IN-SITU STRESS FOR TUNNELS

When an opening is made in the ground, the pre-existing in-situ stress controlled equilibrium is altered and a new state of stress is induced (Fig. 8) around the boundary of the opening. This new state of stress can render the rock mass around the opening unstable if the strength of the rock mass is less than the induced stress intensity, which is a function of in-situ stress and shape of the opening. In-situ stress knowledge helps in choosing the orientation for a cavern, one hope to avoid aligning the long dimension perpendicular to the greatest principal stress, if the initial stresses are very high, the shape will have to be selected large1y to minimize the stress concentrations. Pressure tunne1s and penstocks can be constructed and operated in rocks without any 1ining if virgin stresses are greater than the internal water pressure.

Fig.8 shows induced stresses existing at different chainages measured by flat jack tests in the 4055 m long HRT of 9.5 m dia. horse-shoe type, at Koyna Stage-IV H.E. Project. The stresses are measured at different chainages, which are selected as per the variation of the overburden in 4055 m long HRT. The overburden height is varying from 30 m to 145 m. It is seen that except at chainage 2550 m, where HRT cover is about 30 m (less than the four times dia. of the opening), the maximum value of induced stress i.e. 35.50 MPa has been found. n this region rock mass is Basalt, which has rock mass strength of 38.74 MPa. There is not much difference in values of induced stresses and rock mass strength. So in this less cover region reinforced concrete lining has been designed. At other places plane concrete lining has been adopted. So measuring induced stresses by flat jack gives clear idea for requirement of design supports.

6. DISCUSSIONS

From the results presented in Table-1, it is observed that the ratio of horizontal to vertical in-situ stresses (K) varies from 0.40 to 1.52 up to depth of 260 m. From the world wide data available (Fig.9) the same is found to vary from 0.5-3.5 for depths up to 500m. There is no evidence of the influence of rock type or the overburden on the K values (Fig.9). The vertical stress is found to agree with the overburden stress in the case of stratified and volcanic rock formations. In the case of metamorphic rocks, the vertical stress is found to be in excess of the overburden stresses (Fig.10), it is also found to be heterogeneous within a given site, contrary to conclusions drawn by (Hoek and Brown, 1978) equating the vertical stress to overburden stress. The vertical stresses are higher than the overburden stress particularly at depths less than 500m (Fig. 11).

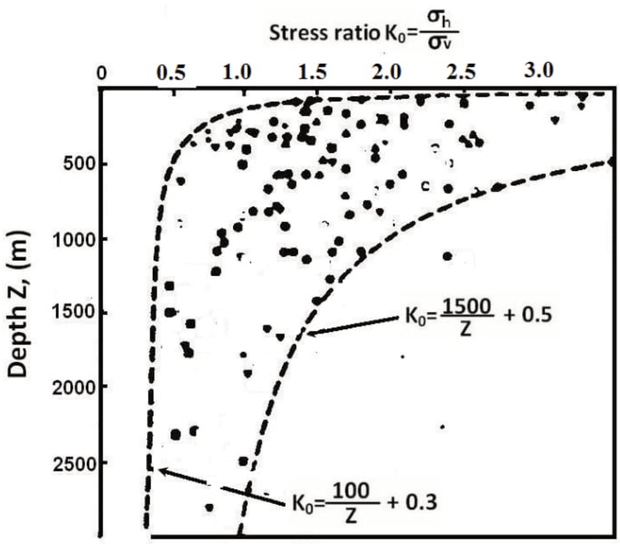

Fig. 9 - Variation of measured horizontal to vertical Stress with depth (Hoek and Brown 1978).

Based on the stress measurement data:

$\sigma_{h\ min}$ = (2.0± 0.016Z)
$\sigma_{h\ max}$ = (7.5+0.024Z)
σ_h = (0.025Z)

where $\sigma_{h\ min}$ and $\sigma_{h\ max}$ are the minimum and maximum horizontal stresses in MPa and Z is depth in meters.

Probably due to such conclusions, it is generally accepted that the vertical stress more or less agrees with the overburden stress. Although, it is clearly evident from a closer examination of Figs.10 and 11, that the conclusions are far from true for depth less than 500m. Mostly, the underground openings in Civil Engineering projects like power tunnel, pressure shafts, surge shafts, powerhouse, storage caverns etc. are situated within 500m depth. Including the data available from India, it is observed that no viable relation could be drawn between the depth and stress (Fig.12) and it would be safe to conclude that in majority of the cases, the in-situ vertical stress is in excess of the overburden stress (Table 1). So, for stress based design approach it is essential to measure the stresses relevant to a given site.

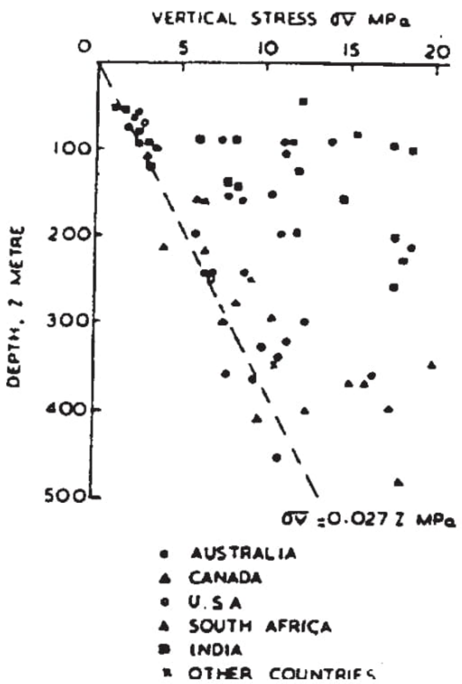

Fig. 10 - Variation of measured vertical stress with depth up to 500m

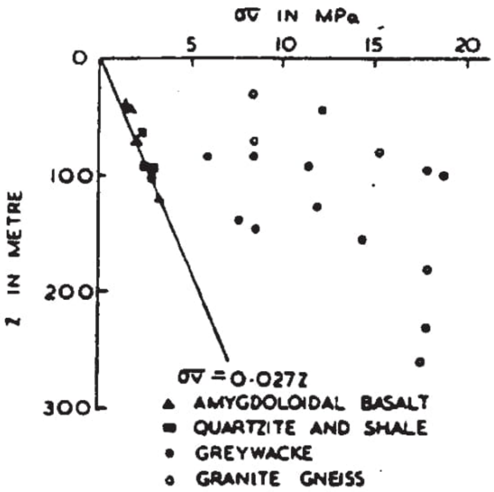

Fig. 11 - Variation of measured vertical stress with depth up to 300

The vertical stresses based on overburden are calculated and are shown in Table 1. The vertical stresses due to overburden are found to be much less than the measured vertical stresses by flat jack tests. The measured flat jack stresses are plotted in Fig. 13. So it is very much comparable with the world wide data that for depths up to 500 m measured stresses are always more than the overburden stresses.

7. CONCLUSIONS

The rock mass on the boundary of the opening is under biaxial stress field with the tangential stress being the major of the two stress components especially in the case of long opening where plain strain conditions are applicable. The strength of the rock mass on the boundary is controlled by the major principal stress in a biaxial stress field. If this stress exceeds the strength of rock mass under identical stress field, zones of instability would develop around the boundary. In the rock mass away from the boundary, the tangential stress would reduce and the radial stress would increase thereby improving the strength of rock mass. The cause of instability is therefore the removal of confining pressure on the boundary. Having identified the zones of instability, it is possible to allow the rock mass to support itself by providing required support. By changing the shape of the opening it is possible to alter the induced stress intensity. The required σ_3 can be provided by means of rock bolts, anchors, shotcrete, concrete lining and steel supports.

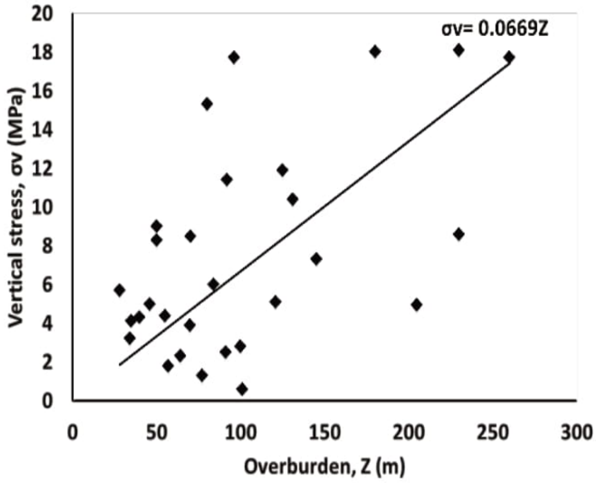

Fig.12 - Variation of measured vertical Stress with depth up to 260 m

For carrying out an exercise of the type discussed above, the pre requisites are; knowledge of in-situ stresses and strength of rock mass. The induced stresses around the opening can be easily calculated by means of Photo-elastic or numerical modus like FEM or BEM. The strength criterion can be incorporated into these modules to identify the zones of instability.

From the flat jack test results it is also concluded that existing stresses are site specific. Measurement of in-situ stresses becomes essential for proper design of stability measures for any underground opening and flat jack tests results can be used for the proper design of underground openings.

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References

Bieniawski, Z. T. (1984). Rock mechanics design in mining and tunnelling. Pub A. A. Balkema, Rotterdam.

BIS (1974). Code of practice for in-situ determination of rock properties by flat jack, BIS-7292.

CWPRS (1979). Rock mechanics studies for Supa dam site Kalinadi hydroelectric project, Karnataka, Technical Report 1888.

CWPRS (1986). In-situ stresses along the water conductor system of Idamalayar project, Kerala, Technical Report 2360.

CWPRS (1987). Deformabilit of rock mass and in-situ stresses at access tunnel and tail race tunnel junction, Kadamparai project, Tamil Nadu, Technical Report 2429.

CWPRS (1988). In-situ stresses, Deformability and engineering properties at Kuttiyadi tunnel, Kerla, Technical Report 2555.

CWPRS (1988). In-situ stresses and deformability of rock mass power tunnel lower periyar hydro project Kerala, Technical Report 2558.

CWPRS (1990). 2D and 3D analysis of underground power house at Varahi H.E. Project, Karnataka, Technical Report 2696.

CWPRS (1991). Deformability of rock Mass, in-situ stresses and engineering properties of rock material of Koyna H.E. project (Stage-IV), Maharashtra, Technical Report 2873.

CWPRS (1991). Deformability of rock mass, in-situ stresses and engineering properties of rock material, pressure shaft, Kakkad hydroelectric project , Kerala, Technical Report 2874.

CWPRS (1992). Deformability of rock mass in-situ stress and engineering properties of rock material, Bhira Generating Station extension (old tunnel) Maharashtra, Technical Report 2936.

CWPRS (1992). In-situ stresses and deformability of rock mass at underground power house site, Srisailam hydroelectric project, Andhra Pradesh, Technical Report 2983.

CWPRS (1992). Engineering properties of rock material, Koyna H.E. project (Stage-IV), Maharashtra, Technical Report 3035.

CWPRS (1997). Rock mechanics studies for the powerhouse area of Indira Sagar project, Madhya Pradesh, Technical Report 3447.

CWPRS (1997). Rock mechanics studies for headrace tunnel, Koyna hydroelectric Project, stage IV, Maharashtra, Technical Report 3464.

CWPRS (2003). Measurement of in-situ stresses and deformability of rock mass at various locations of Ghatghar hydroelectric Project, Thane, Maharashtra, Technical Report 3973.

Goodman, R.E. (1988). Introduction of Rock Mechanics. John Willy and Sons.

Herget, G (1988). Stresses in Rock. A.A. Balkema, Rotterdam, Brookfield.

Hoek, E and Brown, E.T. (1980). Empirical strength criterion for rock masses. ASCE, Vol. 106, No. GT9, 1013-1035.

Hoek, E and Brown, E.T. (1978). Trends in relationship between measured in-situ stresses and depth. Int J Rock Mech Min Sci, 15(4): 211-215.

Hoek, E., Martinos, P. and Benissi, M. (1998). Applicability of the geological strength index classification for very weak and sheared rock masses Bul. Eng Geol Env, 57:151-160.

ISRM (1979). Suggested methods for determining in-situ deformability of rock, Int J Rock Mech Min Sci, 6(3):131-141.

ISRM (1987). Suggested methods for rock stress determination. Int J Rock Mech Min Sci, 24(11):53-74.