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Determination and Applications of Deformation Modulus for Rock Mass

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ABSTRACT

Widely varying stiffness of rock formations pose problems of uneven settlements and consequent distress in the foundation. Safety of structures built on soft foundations need to be examined in relation to the moduli values of the material of the structure and that of the foundation. Foundation in the homogeneous rock mass are preferred than the foundations in the inhomogeneous rock mass due to the problem of differential settlements in the inhomogeneous rock mass. The commonly used methods for determining deformation modulus have been described in detail. In situ plate load (PLT) and flat jack (FJT) tests were conducted at different dam sites and underground openings in different regions of India. An attempt has been made for determining the deformation modulus of different types of rocks available in different regions of India. These can be used for preliminary assessment of deformation modulus of different type of rock masses. The average deformation modulus (E_m) determined by PLT and FJT is used for classifying the rock mass. The E_m values determined by FJT as compared to that obtained by PLT are higher for massive and excellent rock mass because flat jack is of half the size (30 x 30 cm) of PLT plate in which lesser number of joints may be present. Applications of deformation modulus for different types of structures have been discussed. Effect of deformation modulus of rock mass, for transfer of internal pressure in the tunnel to the rock mass and stress distribution in lining has been presented.

Keywords: Elasticity; Deformation modulus; Inhomogeneous rock mass; Intact rock; Jointed rock mass; Plate load test

1. INTRODUCTION

The deformations in rock concern engineers even when there is little risk of rock failure because locally rock deformations, can raise stress within the structure. There are many situations in which rock deformations should be determined. To design pressure tunnels, one should know the expansion of the lining under operating pressure, as well as the amount of recovery when pressure is lowered. The same is true of arch dams pressing against their abutments. Tall buildings on rock may transmit sufficient load to their foundations that rock settlements becomes significant for design. For long span, prestressed roof structures and bridges, anchorages in rock, structures pushing against rock, or gravity blocks seated on rock, knowledge of the rock deformations is basic to design details. For any excavation that is monitored, the expectable deformations should be calculated to provide a framework with which to interpret the measurements. It is not sufficient to characterize rock deformability by elastic constants alone, as many rocks are non-elastic. Elasticity refers to the property of reversibility of deformation in response to load. Many fresh hard rocks are elastic when considered as laboratory specimen. But on the field scale, where the rock can be expected to contain fissures, fractures, bedding planes, contacts and zones of altered rock and clays with plastic properties, most rocks do not exhibit perfect elasticity. So, when deformability is measured in jointed rocks, it is called deformation modulus (E_m), whereas deformability measured for intact rocks (without any joints) is called elastic modulus (E). In structures concrete is found to be more elastic than rock mass, as joints in the rock may open or close in different loading and unloading cycles. Some sites have been considered unacceptable for concrete dams because of large hysteresis even though the deformation modulus of the rock itself was considered reasonable.

2. DEFORMABILITY OF ROCK MASS AND ITS MEASUREMENT

The deformations of linearly elastic isotropic solids can be calculated for known increments in stress if only two material constants are specified. These are Young's modulus (E) and Poisson's ratio (ν). The quantities E and ν can be determined directly from tests where known stress is applied and strains are measured. Many rock masses are anisotropic, that is, directional to their behavior, due to regular bedding or jointing or oriented fabric or microstructure that makes the rock itself anisotropic.

Deformability is one of the most important parameters, governing the behaviour of rock mass regardless of the structure type. The structure may be concrete gravity dam, arch dam, lining in tunnels or settlement analysis of heavy bridge piers, where the deformation modulus has an important role to play in assessing the stability.

Since rock masses are heterogeneous, tests conducted on smaller rock specimens in the laboratory, do not produce the deformation data, which could be applied in the field of geotechnical problems. This has led researchers to conduct the large scale in situ tests for the determination of modulus of deformation of the rock mass. It is important to determine the rock mass deformability characteristics, especially in the direction of loading with short and long term time-dependent loading effects.

Stress-strain relationship can be observed in static and dynamic tests conducted in the laboratory or in the field. The most widely used testing procedures for deformability measurements are laboratory compression tests, wave velocity measurements in the lab or field, field loading tests using flat jacks or plate bearing apparatus and borehole expansion tests.

2.1 In situ Experimental Methods

Once the rock mass is characterized into different zones on the basis of geological classifications and borehole data, the next step is to conduct the in situ tests to obtain reasonably reliable estimate of the deformation properties of each of these zones. The emphasis is to be given more for number of simple tests rather than one or two costly tests. Plate load test, flexible dilatometer test with radial displacement measurements, flat jack test, radial jacking test, Goodman jack test and seismic refraction test are commonly used as the in situ deformability tests.

2.1.1 Plate load test

Test locations are finalized in consultation with the geologist based on the available level surface and subsurface geological data. In general, deformability tests are conducted in exploratory drifts, adits, tunnels, and in trenches at the surface of the foundation grade rock of dam with the help of anchor loading arrangement. The deformability of rock may be measured in the field by loading a rock surface and monitoring the resulting deformation.

Further for the jointed and intact rock different test procedures are adopted. For the intact rock rigid pad method (Fig.1) is adopted, whereas for jointed rock, flexible pad method is used (Fig.2). The site is selected carefully to exclude loose, highly fractured rock that might be unrepresentative of the rock condition. A relatively flat rock surface is sculptured and leveled with mortar to receive circular or square bearing plates 60 cm to 1m in diameter.

Fig. 1: Plate load test setup for deformation modulus with rigid plate method

Fig. 2: Plate load test setup for deformation modulus with flexible plate method

The depth of the rock volume affected is proportional to the diameter of the loaded area, so it is desirable to choose large bearing plate, but it proves difficult to apply loads greater than 200 tons so it may be necessary to reduce the plate size in order to achieve desired contact pressure levels. The load is applied in small increments on carefully prepared flat surface, by means of hydraulic/screw jacks through a rigid or flexible finite plate, the resultant deformations are measured at the surface as well as at different depths below the loaded surface using dial gauges, and multiple point borehole extensometers when deformations at different depths are to be determined.

2.1.2 Flexible dilatometer test with radial displacement measurements

This test uses an expanding probe to exert pressure on the walls of a drill hole. The resulting drillhole expansion (dilation) is measured directly by a displacement transducer in the probe. Deformability characteristics of the rock mass at the dilatometer locations are calculated from the relation between pressure and dilation. A dilatometer probe as shown in Fig.3, includes a high-pressure flexible membrane, mounted on a core, such that the membrane can be inflated to press against the drill hole wall.

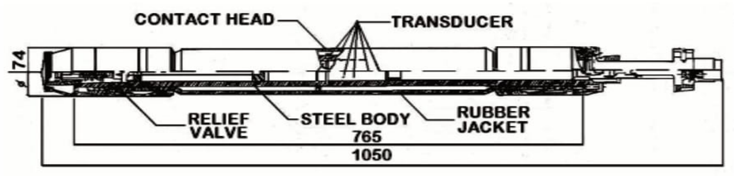

Fig.3: Diagram showing dilatometer

At the maximum test pressure, the applied pressure is to be maintained constant during at least 10 minutes. Readings of dilation verses time at constant pressure are tabulated to determine creep rates. Dilation and pressure readings are noted down during unloading and three cycles of loading and unloading are required to be completed for each testing point.

2.1.3 Flat jack test

Flat jack tests are the most useful non-destructive tests used for determining rock/masonry structural properties. It does not rely upon correlation to laboratory tests.

The test is based on the principle of partial stress release and involves the local elimination of stresses, followed by controlled stress compensation.

The reference field of displacements is first determined by measuring distances between gauge points fixed to the surface of the rock/masonry (Fig.4). Then, a slot is cut in a plane normal to the direction of measured stresses (Fig.5). This allows deformations in a direction normal to the slot. Distances between gauge points decrease after making of a slot. Cutting the slot causes partial stress relief in rock/masonry above and below. Then distance after converging of pins is measured (Fig.5) with the help of dial gauge.

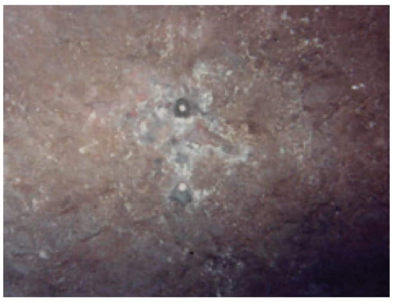

Fig. 4: Fixation of distance measuring pins at testing point

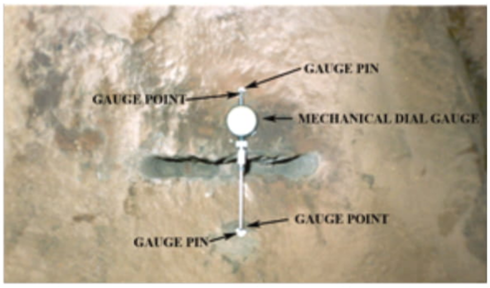

Fig.5: Distance measuring by dial gauge between two pins after making a slot

Afterwards, a thin flat jack (Fig. 6) is introduced into the slot and tightly grouted. With the aid of this device, pressure (compressive stress) is applied to the slot surfaces in the rock/masonry (Fig. 7).

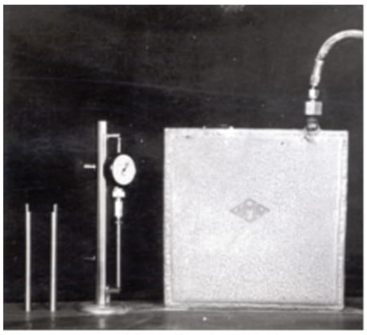

Fig.6: A set of flat jack, dial gauge and pins

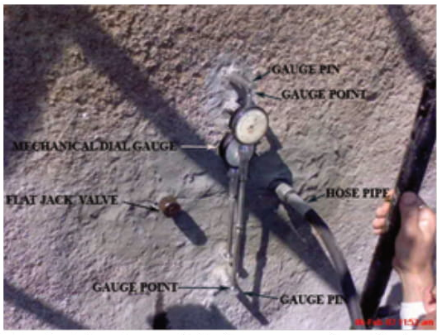

Fig.7: Placing of dial gauge after fixing flat jack between two pins

This causes a partial restoration of the initial displacement field, which at some point reach (approximately) to the previously measured distance between pins. The necessary pressure is called cancellation pressure and can be related to the compressive stress in the direction normal to the slot. The hydraulic pressure in the flat-jack, necessary to restore the undamaged state is higher than the actual stress.

This is caused by the inherent stiffness of the flat-jack, which resists expansions when the jack is pressurized. Another factor that contributes to this effect is the difference between the area of the jack and the area of the slot (the latter being greater then the former). Both these factors are taken to be in account when interpreting test results.

The test, as described above, is based on the following assumptions: the stress in place of the test is compressive; the rock/masonry surrounding the slot is homogenous; the rock/masonry deforms symmetrically around the slot; the state of stresses in the place of the measurement is uniform; the stress applied to rock/masonry by the flat-jack is uniform; and the value of stresses (compared to compressive strength) allows the rock/masonry to work in an elastic regime.

2.1.3.1 Evaluation of deformation modulus, E_m

When slot is made in the rock/masonry wall, stress originally existing across it relieves the rock/masonry surface of the existing stresses. Because of the stress relief, the sides of the slot converge. The amount of convergence, which depends upon the stresses in the rock/masonry and its elastic properties, is used for determining the deformation modulus, which is given by the Eq. 1 (ISRM,1986).

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

and a_o is determined as $\sqrt{1+\frac{Y^2}{C_o^2}}$

where, w is the amount of convergence between two points spaced at equal distance Y from the plane of the slot along the center line normal to its plane due to stress P, E_m is the modulus of deformation, $2C_o$ is the length of flat jack, and υ is Poisson's ratio.

2.1.4 Radial jacking test

Radial jacking test measures the deformability of a rock mass by subjecting a test chamber of circular cross section to uniformly distributed radial loading; the consequent rock displacements are measured, from which elastic or deformation moduli are calculated (ISRM, 1979). The test loads large volume of rock so that the results may be taken to closely represent the true properties of rock mass, taking into account the influence of joints and fissures. It is extremely costly test.

2.1.5 Goodman jack test

Goodman jack (Goodman, 1988) is a borehole probe transmitting hydraulic pressure to rock walls through moveable rigid bearing plates for the measurement of wall deformation as a function of applied load in NX borehole. E_m is computed as the tangent modulus along the slope of the linear portion of this plot of the field data by Eq. 2.

$$E_m = 0.86\,\mathrm{K(V)}\frac{\Delta \mathrm{Q.d}}{\Delta \mathrm{U_d}} \tag{2}$$

where
ΔQ = incremental change in plate pressure in MPa,
ΔU_d = incremental change in displacement in mm,
d = borehole diameter in mm, and
K(V) = jack constant, depending upon Poisson's ratio of material being tested.

2.1.6 Seismic refraction test

The in situ test using a seismic refraction technique is fast and convenient method for getting in situ determinations of E_m that are representative of large volume of material. Accurate determination of the velocities of the P and S waves is the first requirement for effective dynamic testing. Since these velocities are directly related to the elastic constants of the medium through which they pass, they may be used to calculate dynamic modulus, E_u. Dynamic elastic modulus is the important rock parameters for describing the influence of earthquakes and explosion waves. The relationships used for the determination of dynamic elastic modulus using compression and shear wave velocities are given in Eqs. 3, 4 & 5 (IS:10782,1983):

$$E_u = V_s^2.2.\rho(1+\nu) \tag{3}$$

$$E_u = V_p^2.\ \rho(1-2\nu)(1+\nu)/(1-\nu) \tag{4}$$

$$E_u = (V_p\text{-}2\ V_s^2)/2(V_p^2\text{-}\ V_s^2) \tag{5}$$

where V_p and V_s are compression (P) and shear (S) wave velocities, ρ and ν are density and Poisson's ratio respectively normally determined in laboratory.

An explosive source is used to generate waves at a point on the ground surface and the arrival of waves at different points on the surface is observed. The wave velocity for each layer is given by the inverse of the slope of the time vs. distance plot.

2.2 Laboratory Methods

Laboratory tests are carried out on rock specimens of regular size for intact rocks.

2.2.1 Static elastic modulus

Cylindrical samples with a length to diameter ratio (L/D) of 2.0 to 3.0 are prepared such that the circular end faces are plane and perpendicular to the axis of the cylinder. The load is applied continuously in small increments such that maximum stress is applied within 5-10 minutes of loading or stress rate shall be within limits of 0.5–1.0 MPa/sec. By knowing the axial strain and the corresponding stress, the value of static modulus of elasticity between any two-stress values and also at any particular stress value are evaluated as per I. S. Code 9221:1979.

2.2.2 Laboratory dynamic elastic modulus (E_u)

The technique for dynamic elastic modulus (E_u) involves accurate measurement of travel time of elastic waves through a known length of rock core, which is propagated. This involves accurate measurement of travel-time of elastic waves through a known length of rock core by PUNDIT (Portable Ultrasonic Non Destructive Indicating Tester), which enables to compute velocities of the wave propagation. A mechanical impulse imparted to a solid elastic body generates a group of body waves, viz. compression and shear waves. These waves travel in the elastic solid medium with a certain velocity, from which the elastic properties such as dynamic elastic modulus, of the solids could be computed. The transmitter generated electronic pulses of frequency of 54 kHz, are applied to the rock specimen, to produce elastic compression and shear waves. The velocity of these waves through rock specimen is used to compute dynamic modulus (E_u).

2.3 Empirical Methods

2.3.1 Bharat Singh co-relation

The correlation as suggested by Singh (1973) and adopted by U.S. Corps of Engineers, is a very useful relationship, for determining the deformation modulus of rock mass. The modulus reduction factor (MRF) is given as:

$$\mathrm{MRF} = \frac{1}{1 + A.n} = \frac{E_m}{E} \tag{6}$$

where

E_m = static modulus of deformation obtained from in situ tests,
E = elastic modulus of rock material obtained from laboratory tests,
n = average number of joints per meter in the direction of load in plate load test, and
A = 0.60 m for continuous joints or loose bedding planes in unweathered rock mass, 0.25 m for discontinuous joints in unweathered rock mass and 0.05 m for unweathered cleavage planes but separated.

2.3.2 Bieniawski co-relation

Rock mass modulus could be predicted approximately using rock mass rating (RMR) obtained from Bieniawski (1978) geomechanics classification system.

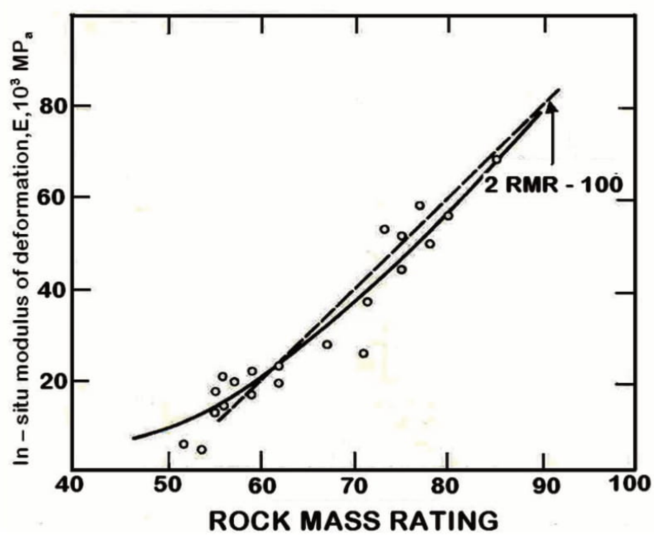

Fig. 8: Relationship between rock mass rating and rock mass deformability (Bieniawski, 1978)

Figure 8 shows in situ values of modulus of deformation, determined by various large scale field tests at number of sites, plotted against the rock mass rating higher than 55, the data points are fitting approximately by Eq. 7.

$$E = 2\,(RMR) - 100 \qquad (7)$$

Bieniawski (1984) gave another co-relation as in Fig. 9 which shows relationship between geomechanics classification and ratio of static deformation modulus of rock mass (E_m) to that of rock material (E). The correlation obtained from the graph (Fig.9) is $E_m/E=0.086\, e^{0.021RMR}$.

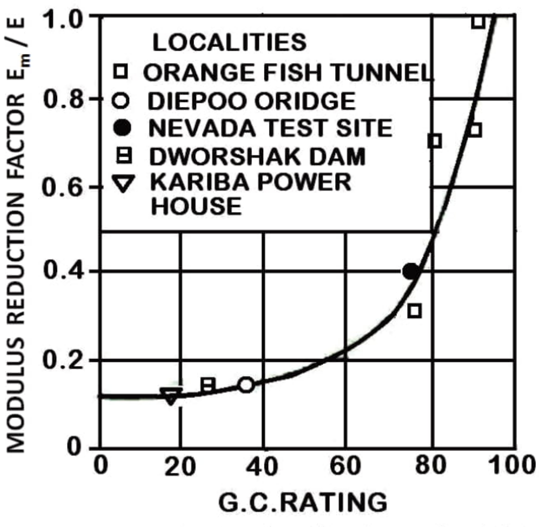

Fig. 9: Relationship between geomechanics classification rating (RMR) and ratio of static deformation modulus of rock mass (E_m) to that of rock material (E)

2.3.3 Serafim and Pereira co-relation

Serafim and Pereira (1983) have proposed a relation between E_m and rock mass rating (RMR) as per Eq. 8.

$$E_m = 10^{\left(\frac{RMR-10}{40}\right)} \tag{8}$$

This relation is useful for a preliminary assessment of E_m values. However, it is found to be inadequate in case where rock mass is soft and weak.

3. APPLICATION OF MODULUS OF DEFORMATION

The modulus of deformation**,** E_m obtained from the secant modulus of the loading limb at the design stress level is used for the conventional design and numerical analysis of structures founded on hard strata. From the study of the stress deformation curves the following interpretations can be made.

3.1 Assessment of Quality of Rock

The ratio of modulus of elasticity of rock material determined in the laboratory to the modulus of deformation determined in field at different design stress levels gives a clear picture about the number of joints and their quality present in the rock mass (not clear). The assessment of the settlements, which may take place during the construction and after completion of the construction, can be determined from the ratio of modulus of elasticity to the modulus of the deformation.

3.2 Computer Aided Stability Analysis for Dam Foundations

In the conventional approach, the ratio of the sum of the resisting forces to that of disturbing forces is evaluated to obtain the factor of safety. This is an over simplification particularly while analyzing the stability on weak planes within the foundation.

In finite element analysis, the rock mass is divided into different zones as per the geological classifications and deformation modulus values (Fig.10). E_m is very important parameter in FEM analysis for dam foundations with weak zones and seams. To carry out the stability analysis, E_m of all the zones is to determined and taken into consideration by locating it at its proper place as per the site conditions. A dam rests on heterogeneous geological formation consisting of rock masses of widely varying deformability is idealized by a two dimensional finite element model as shown in Fig.10.

3.3 Effect of Deformation Modulus of Rock Mass for Transfer of internal pressure in the Tunnel to the Rock Mass

In a circular tunnel the analysis is carried out assuming perfect contact between concrete lining and surrounding rock mass. The tunnel (Fig.11) is considered to be stable and the lining is installed after the displacements have been stabilized. External loads are therefore not considered and the analysis has been carried out only for internal loads as the analysis is based on thick cylinder theory. The lined tunnel is considered to be surrounded by infinite rock mass. Condition of infinity is satisfied if the extent of surrounding rock mass is more than 5 times the diameter of the tunnel.

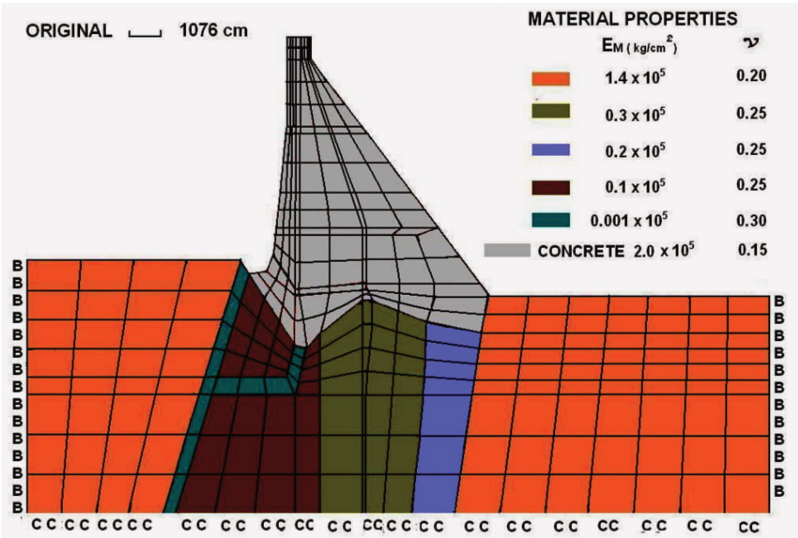

Fig. 10: Finite element idealization model of a foundation with deformation modulus values as determined for different groups

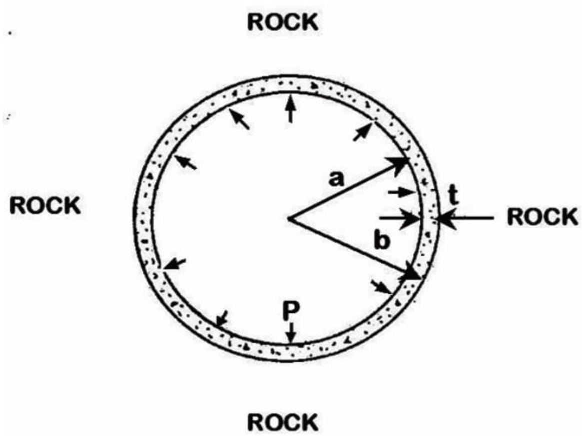

Fig.11: Circular Tunnel

The co-efficient of radial transfer (λ) of radial pressure to rock mass surrounding the concrete lined pressure tunnel is given by Eq. 9 (Herget, 1988):

$$\lambda = \frac{\dfrac{2a^2}{E_c\left(b^2 - a^2\right)}}{\dfrac{M_2 + 1}{M_2 . E_m} + \dfrac{(M_1 - 1)b^2 + (M_1 + 1)a^2}{M_1 . E_c\left(b^2 - a^2\right)}} \tag{9}$$

where

P = internal pressure in the tunnel,
A = radius of the tunnel,
t = thickness of concrete lining,
E_m = static modulus of deformation of rock mass,
ν_m = Poisson's ratio of rock mass,
E_c = modulus of Elasticity of concrete,
ν_c = Poisson's ratio of concrete,
b = (a + t),
M_1 = $1/\nu_c$ and $M_2 = 1/\nu_m$.

Radial stress, σ_r and tangential stress, σ_θ (tensile) in concrete lining at a radial distance `r' from the center of the tunnel are given by Eq. 10 (Herget, 1988):

$$\sigma_r = K_1 - \frac{K_2}{r^2}.P \qquad \text{and} \qquad \sigma_\theta = K_1 + \frac{K_2}{r^2}.P \tag{10}$$

where

$$K_1 = \frac{b^2(\lambda) - a^2}{b^2 - a^2} \qquad \text{and} \qquad K_2 = \frac{(\lambda - 1).b^2 a^2}{b^2 - a^2}$$

The parametric analysis is carried out for understanding the effect of E_m values. For the E_m value of 14.5 GPa the radial stress σ_r transferred to the rock mass is 0.75P (Fig.12) surrounding the concrete lining and maximum tensile stress,σ_θ developing in the inner face of the concrete lining is 1.05P (Fig. 13). The corresponding values of σ_r and σ_θ for E_m value of 24 GPa are 0.82P (Fig. 12) and 0.63P (Fig. 13) respectively. Analysis is carried out for E_m values ranging from 10 GPa to 32 GPa.

3.4 Stability Analysis of Underground Openings

Up to a certain value of E_m (11 GPa) support pressures are influenced by E_m but once that value is achieved then further increase in E_m value does not cause much reduction of support pressure (Dhawan, 2002).

In the case of a horseshoe tunnel the rock mass met within the head race tunnel is predominantly good quality compact Basalt (E_m=2x10^5 kg/cm^2) intermixed with layers of relatively soft volcanic breccia (E_m=0.4x10^5 kg/cm^2). The extent of breccia configuration varies from section to section. Stress analysis studies were carried out to assess the influence of the soft breccia on stress distribution in the concrete lining. In the two cases studied, case I breccia is found between 8 m above base and 5 m below base of lining and remaining portion is basalt as shown by deformation values in Fig. 14. In case II, Fig. 14 breccia exists all around the opening. The determined tangential stresses with finite element analysis at crown, spring, bottom cover and base are 38.68, 31.62, 133.50 and 26.18 kg/cm^2 for case I which varies to 47.20, 49.16, 135.90 and 34.55 kg/cm^2 in case II respectively. It is seen that with reduction of deformation modulus in case II the increase in stresses and also its spread of higher stresses in more portion of the rock mass surrounding the opening (1 MPa = 10 kg/cm^2).

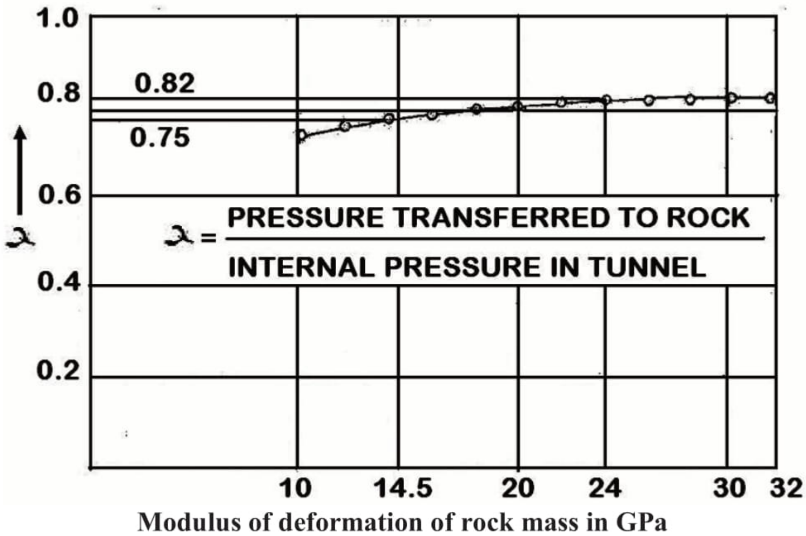

Fig.12: Effect of modulus of deformation on pressure transferred to rock

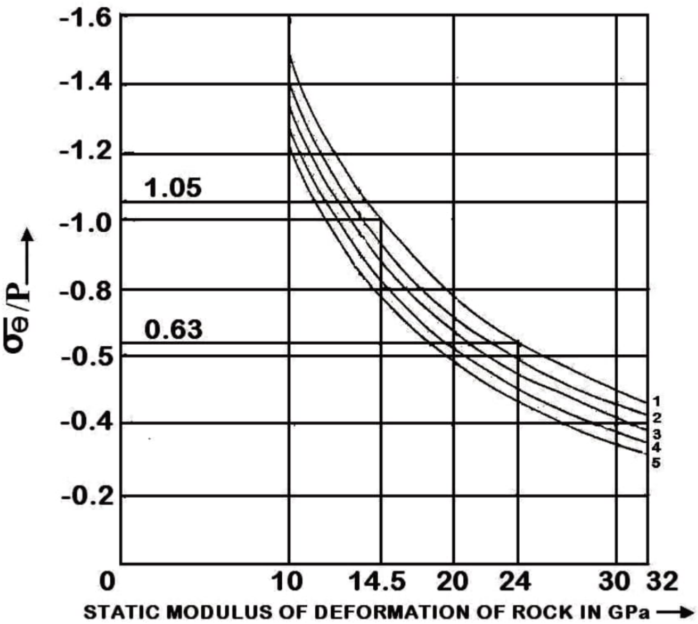

Fig. 13: Effect of deformation modulus on circumferential stress in concrete lining of a circular tunnel

4. CASE STUDIES

CWPRS has carried out modulus of deformation studies for many projects. Five different projects viz. Hirehalla dam spillway, CWPRS report No. 3458 (1997), Supa dam site, Karnataka, CWPRS report No. 1828 (1979), Bhadra dam, Karnataka, CWPRS report No. 3127(1994) and Rajasthan Atomic Power Plant 3 & 4, CWPRS report No. 2752(1990) are considered for the studies.

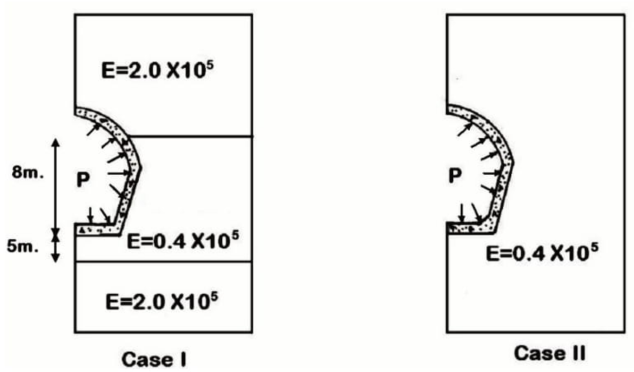

Fig. 14: Different types of rock existing in Head Race Tunnel

4.1 Rock Mechanics Studies for Hirehalla Dam Spillway

4.1.1 Project details

The 15 m high composite Hirehalla dam consisting of 3.60km long earthen embankment with 89.5 m long spillway was constructed near Kinnal village across two tributaries, Hirehalla and Veerapurhalla to Tungabhadra river. From the study of the borehole data, it has been observed that the subsurface has been subjected to deep weathering and very poor to moderate recovery have been reported up to considerable depths. The average depth of weathered strata consisting of soil, murrum and disintegrated rock is about 9.0 m from ground level at EL 536.3 m. A view of various strata from ground level is shown in Fig.15. Below an average EL of 527.3 m, granite gneiss rock mass has been encountered as shown in Fig.16 and is classified as soft, medium and hard.

4.1.2 Test locations

Although the foundation level at spillway zone has been tentatively fixed at a depth of 19.3 m from ground level i.e. at 517.0 m, keeping in view of the small height of the dam and the attendant low order stresses being imposed at the foundation, field investigations were carried out in stages to examine first the possibility of raising the foundation at a higher elevation after ascertaining the deformation modulus of rock mass.

Fig.15: Weathered and disintegrated granite gneiss rock strata at EL 522m

Fig. 16: Moderately good granite gneiss rock strata at EL 518m

Accordingly, first series of insitu testing, two deformation modulus tests of rock mass were carried out in a trench excavated between chainages 1630 m and 1690 m at EL 524 m in the spillway zone. The second series of three deformation modulus tests were conducted in an open pit within the spillway zone at EL 520 m. The third and final stage of tests was conducted at EL 518 m in the spillway zone by conducting two plate load tests. At the above mentioned elevations rigid pad technique of plate jacking test was used. The test procedure consists of applying load by a single 100-ton hydraulic jack on 315cm^2 area through a 20cm diameter and 20cm thick rigid pad resting directly on leveled rock surface.

4.1.3 Test results and discussion

The average modulus of deformability at EL 524m and 520m are found to be 0.24 and 0.25GPa respectively (Fig. 17). The average modulus of deformability at EL 518m increases to 0.50GPa (Fig.18).

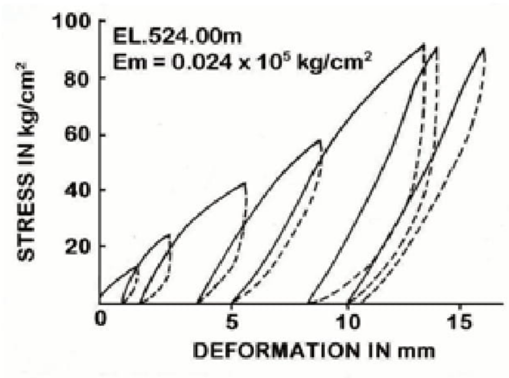

Fig.17: Stress deformation envelopes at EL 524m

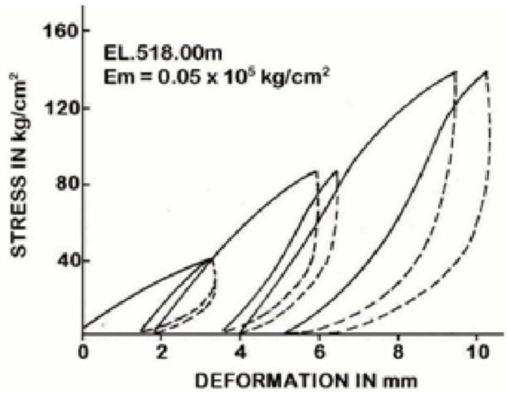

Fig.18: Stress deformation envelopes at EL 518m

Because of deep weathering, the foundation rock mass has been found to be very soft both at EL 524m and 520m with the value of modulus of deformation being 0.25GPa. As per the borehole data the foundation strata at EL 518m is expected to be better that of higher elevation and the same is confirmed by plate load test. The modulus of deformation is found to be 0.50GPa. It was concluded that modulus of deformation of the proposed foundation level is adequate to meet the stresses likely to be imposed by the structure.

4.2 Rock Mechanics Studies at Supa Dam Site, Kalinadi H.E. Project, Karnataka

4.2.1 Project details

The 101m high concrete gravity dam was built across river Kali at Supa forms a major constituent of the massive Kalinadi hydro-electric project. The water of the Supa reservoir is led to Nagjhari power house having an installed capacity of 810MW. The Supa dam powerhouse has two units of 50 MW each. Extensive field investigations for assessing the deformability of the rock mass covering the entire foundation zone were carried out. Drifts were made at different levels to asses the modulus of deformation of foundation rock mass.

The rock mass at the Supa dam site is predominantly banded magnetite quartzite, fresh and hard in the riverbed and jointed and weathered in the flanks to considerable depths. Numerous shear zones containing soft to very soft material traverse the rock mass in the flanks.

4.2.2 Test locations

In order to ascertain the quality of rock at different elevations, foundation blocks 1 to 7 were investigated thoroughly. Deformation modulus investigations were carried out on the excavated grade rock at EL491m and also in drifts at different levels.

4.2.3 Test results and discussion

Extensive testing for determining deformation modulus was carried at excavated grade rock at EL 491m and also in drifts at different levels and the same was used in the FEM stress analysis for block No.5 as shown in Fig. 19. Modulus of deformation is varying from 0.001 to 1.4×10^5 kg/cm^2 for block No.5. Wide variation of modulus of deformation of rock in different zones of foundation was identified with the help of deformability tests. Block No. 5 of Supa dam has deflected towards the upstream due to its own weight. The weak foundation in the upstream portion of the dam and presence of disintegrated dyke on the upstream of the heel of dam seem to have contributed considerably to these phenomena. It is also seen that the reasonably good quality rock formation on the upstream of the dam does not participate in sharing the load because of the presence of soft upstream dyke. Special design and construction methods were used during the construction of dam to overcome the problem of uneven settlement and consequent distress in the foundation as well as in the structure.

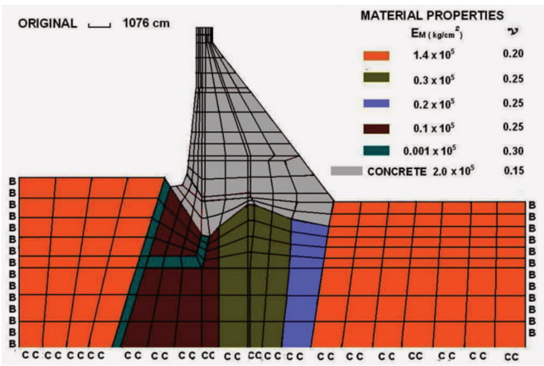

Fig. 19: Finite Element idealization model of Supa dam foundation with deformation modulus values for block No.5

4.3 Rock Studies for the Foundation of Guide wall, Bhadra Dam, Karnataka

4.3.1 Project details

The Bhadra reservoir project comprises of 59 m high Bhadra composite dam across Bhadra river, canal system and dam power house. The benefit from this dam includes irrigation to 105570 ha. of land and 33.2MW of power. A portion of the 33 m high guide wall structure of the right bank canal failed after being in use for nearly 30 years. A new guide wall structure was proposed to be built at the same site.

The foundation rock mass forming the foundation of the guide wall structure consists of talc chlorite schist with schistose planes dipping at about 80°. The rock mass has been subjected to extensive weathering and has become weak and most of discontinuities are filled with disintegrated clayey type of material.

4.3.2 Test locations

In order to ascertain the quality of foundation rock mass, deformation modulus investigations were carried out on the excavated grade rock mass. After removing the distressed parts of the old guide wall structure, the foundation rock mass was exposed and its general quality can be seen in Fig. 20.

Fig.20: A general view of foundation rock mass at Bhadra dam

Two rock ledges i.e. one at CH 30 m (location 1) and other at 35 m (location 2), were selected for conducting the tests. After removing the top rock mass for depths of about 0.5 m, the tests were carried out at elevations of 612.6 m and 610.5 m

4.3.3 Test results and discussions

The results of the deformability tests are presented in Figs. 21 and 22. The rock mass at location 1 was found to be better than that encountered at the location no. 2. The maximum stress applied ranged from 1.50 MPa to 7.90 MPa.

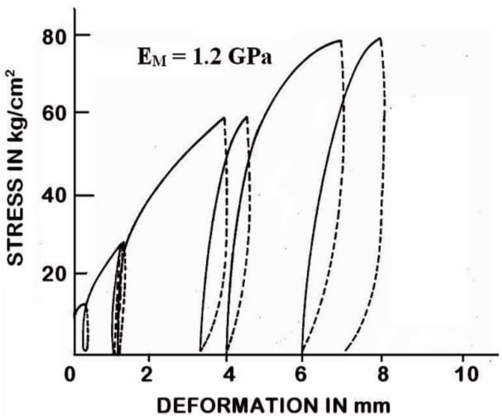

Fig. 21: Deformability of rock mass at Bhadra dam guide wall RL 612.6 m

The modulus of deformation of the rock mass, which is a measure of its stiffness, was found to be very low with its value being 1.2 GPa for location 1 and 0.2GPa for the remaining locations as compared to 15GPa for M15 concrete. In view of the very low stiffness of the rock mass, there could be uneven settlement as a function of elevation differences and consequent unequal loading of the foundation, whenever appreciable level difference is inevitable; the junctions need to be reinforced.

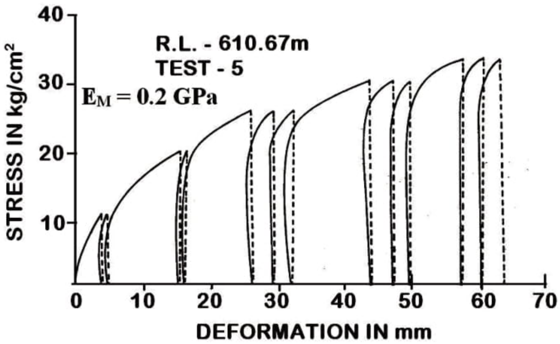

Fig.22: Deformability of rock mass at Bhadra dam guide wall RL.610.50 m.

4.4 Deformability Studies for Rajasthan Atomic Power Plant Units 3 and 4, Rajasthan

4.4.1 Project details

Units 3 and 4 of Rajasthan Atomic Power Plant (RAPP) were constructed near Rawat Bhata in Rajasthan. The capacity of these units is 235MW. Rock mechanics investigations to measure the modulus of deformation of rock mass forming the foundation of the reactor building were carried out. The rock mass at the power plant site is stratified quartzitic sandstone. The thickness of individual layers varies over a wide range. In addition to the near horizontal bedding planes, few prominent vertical joints are also present. Figures 23 and 24 show typical rock formations.

Fig. 23: Horizontal joints at RAPP units 3 and 4

Fig. 24: Vertical joints at RAPP units 3 and 4

4.4.2 Test locations

Four tests were carried out on the excavated walls close to the foundation level in each reactor building housing units 3 and 4. At both the sites, tests were carried out for measuring modulus of deformation in the vertical as well as in horizontal direction.

The flat jack method (IS 13946: Part IV) was used to determine the deformation modulus. The thin slot cut by overlapping holes in the vertical direction is shown in Fig. 25. A flat jack of 30x30 cm in size was fixed tightly into this slot using cement mortar. Reference pins were fixed on either side of the slot at equal distance from the slot axis. The flat jack was connected to a hydraulic pump and pressurized in small increments and corresponding displacement between the reference pins was recorded. A number of loading and unloading cycles were performed at twelve test locations, six each for horizontal and vertical joint sets.

Fig.25: View of slot cut in the vertical direction at RAPP units 3 and 4

4.4.3 Test results and discussions

The quartzitic sandstone rock mass at the reactor site is having a modulus of deformation varying from 8.3GPa to 28.3GPa in the rock mass with horizontal joints and 10.8GPa to 16.6GPa with vertical joints. These values are influenced by the discontinuities and their location with respect to the loading point. It is therefore appropriate to take the effective average value as the representative value for the rock mass at the tested elevation. Considering all the values obtained the effective average values of modulus of deformation of the rock mass with joints in horizontal and vertical direction works out to 14.6GPa and 13.0GPa respectively.

4.6 Test Results from Different Projects

In situ plate load test (PLT) and flat jack tests (FJT) were conducted at many dam sites and underground openings in India for determining modulus of deformation. Some of the results are given in Table 1.

Table 1: Modulus of deformation for different projects

Sr.No	CWPRS Report No. (year)	Name of the Project	Test Ssite	Type of Rock	E_m GPa[#]	Method
1	2429(1987)	Kadamparai H.E.project, Tamilnadu	Pressure shaft Power house tail race tunnel	Charnokite and biotite Granite Gneiss Granite Gneiss	18 31 24	FJT
2	2393 (1986)	Idmalayar H.E.Project, Kerla	Power tunnel surge shaft	Gneiss jointed Gneiss	19 39	FJT
3	2983(1992)	Srisailam dam, A.P	Excavated foundation	Quartzite stratified formation Quartzite and shale formations	20 1.5	PLT
4	1828(1979)	Supa dam, Karnataka	River bed ,, ,, Both abutments ,, Near Toe, heal and shear zones	Fresh magnetite Quartzite Jointed Magnetite quartzite Weathered and jointed Magnetite quartzite Highly jointed weathered manganese bearing strata Highly jointed highly weathered clay bearing Strata	15 7.4 5 3 1 0.1	PLT
5	2809(1990)	Sardar Sarover dam, Gujarat	Foundation Blocks	Massive Trap Amygdolodial Trap Dolomite Sand stone Fractured lime stone Sedimentary Breccia	8.0 4.0 5.0 3.0 0.8 0.6	PLT
6	1990 (1981)	Dudhganga dam, Maharashtra	Foundation Blocks.	1 to 3 m thick quartzite layers weathered quartzite with shale intercalcations. Shale	65 1.9 0.3	PLT
7	NIL(1966)	Tawa Dam, M.P.	Foundation Blocks.	Sand stone	2.4	PLT
8	2682(1989)	Bansagar Dam,M.P.	Foundation Blocks.	Jointed gneiss Chlorite schist	7.0 3.0	PLT
9	2752(1990)	R.A.P.P, Rajasthan 3&4	At foundation Level	Stratified quartzite sand stone with horizontal bedding planes Stratified quartzite sand stone with vertical joints.	14.6 13.0	FJT
10	2874(1991)	Kakkad H.E Project, Kerala	Pressure shaft	Fresh, hard charnokite with bands of granite gneiss, biotite gneiss and boitite schist	42.4	FJT
11	2558 (1988)	Lower Periyar H.E. Project, Kerala.	Power tunnel Surge shaft Pressure shaft	Massive to jointed composite gneiss Gneiss comprising of magmatised charnokite Gneiss with lenses and strings of amphibiotite and granite.	29.4 34.1 33.2	FJT
12	2936 (1992)	Bhira H.E. Project, Maharashtra	Head race Tunnel	Amgdolodial basalt Amgdolodial basalt with Joints	32.0 24.0	FJT
13	3464 (1997)	Koyna H.E. P. stage IV, Maharashtra	Head race Tunnel Surge shaft	Compact basalt Basalt intermixed with Breccia Volcanic Breccia.	44.0 28.0 7.50	FJT
14	3127(1994)	Bhadra dam, Karnataka	Foundation of guide wall	Talc chlorite schist	1.2	PLT
15	3973 (2003)	Ghatghar H.E. Project, Maharashtra	Ventilation Tunnel Transformer hall	Porphyritic basalt Compact basalt Compact basalt	29.0 35.0 30 .0	FJT
16	2447 (1987)	Indira Sagar Project,M. P.	Foundation Blocks.	Quartzite with intercalations of silt stone	0.25	PLT
17	3458(1997)	Hirehalla Dam	Spillway foundation	Weathered Granite Gneiss Disintegrated Granite Gneiss	0.25 0.50	PLT

#1000MPa=1GPa, 10kg/cm^2=1MPa

4.7 Comparison of Test Results for Different Rock Masses

The rock mass tested for deformation modulus is grouped as per engineering geology/rock type classifications. The range and average deformation modulus E_m determined by PLT and FJT are given in Table 2 for classification of different type of rock mass. The E_m values determined by FJT as compared to that obtained by PLT are found to be higher, for massive and excellent rock mass. The flat jack of size 30 cm x 30 cm is covering lesser joints, fissures etc as compared to 60 cm x 60 cm plate size of plate jacking test. In PLT test larger deformations are measured at rock surface which is normally more damaged than the deep surface. So to obtain correct deformations from different loadings in plate loading test, deformations should be measured at different depths in the holes made under the plate of the loading jacks. Making reaction arrangement for vertical loading is costly and time taking task, so determination of deformation modulus by flat jack is preferred and found economical and useful.

Table 2: Comparison of test results of deformation modulus for different rocks

Sl. No	Rock Type	E_m in GPa			
		PLT		FJT	
		Range	Average	Range	Average
1.	Quartzite - massive	65.00	65.00	---	---
2.	Quartzite Stratified	20 to 22.7	21.35	16.70	16.70
3.	Quartzite stratified and jointed	7.40	7.40	13.00	14.60
4.	Quartzite magnatite jointed	3 to 5.00	4.00	---	---
5.	Quartzite with shale intrusions	1.50 to 1.90	1.70	---	---
6.	Quartzite highly jointed and highly weathered	1.00	1.00	---	---
7.	Weathered quartzite	0.25 to 0.50	0.37	---	---
8.	Quartzite with intercalations of silt stone	0.25	0.25	---	---
9.	Limestone	0.8	0.8	---	---
10.	Basalt compact	29.0	29.0	30.0 to 54.0	40.75
11.	Basalt with joints.	---	---	24.0 to 32.0	28.0
12.	Granite gneiss compact.	16.0 to 19.0	17.33	24.0 to 61.0	38.75
13.	Granite gneiss and charnokite	25.0	25.0	30.0 to 38.2	34.10
14.	Gneiss jointed	7.0	7.00	19.0	19.0
15.	Trap massive	8.00	8.00	---	---
16.	Trap jointed	4.00	4.00	---	---
17.	Schist	1.2 to 3.0	2.10	---	---
18.	Breccia volcanic	---	---	6.28 to 7.50	6.90
19.	Breccia sedimentary	0.60	0.60	---	---
20.	Sand stone compact	2.4 to 3.0	3.10	---	---
21.	Shale	0.30	0.30	---	---
22.	Charnokite with granite	---	---	42.90	42.90
23.	Charnokite and biotite	33.0	33.0	---	---
24.	Magnatite	15.0	15.0	---	---
25.	Highly jointed weathered clay bearing	0.10	0.10	---	---

5. CONCLUSIONS

In situ plate load (PLT) and flat jack (FJT) tests were conducted at different dam sites and underground openings in different regions of India having different rock masses. The results given in Table 2 may be adopted for preliminary assessments of deformation modulus. Further confirmation by in situ tests is necessarily depending on the quality of the final foundation grade rock mass. For important structures proper geological classification of foundation grade rock mass and determination of deformation modulus for each classified group is necessary.

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