Journal of Rock Mechanics & Tunnelling Technology (JRMTT)
21 (1) 2015 pp 21-38

Available online at www.isrmtt.com

Shear Strength Evaluation for Dam Foundations on Rock

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ABSTRACT

The rock formations at the foundations of dam sites mostly consist of hard rocks like quartzites interbedded with soft rocks like shales. Strength of rock mass is highly influenced by the discontinuities like joints present in it because of the fact that foundation may fail due to shear displacement of the jointed blocks. Therefore, a detailed investigation of shear strength parameters of rock mass is necessary to assess the requirement of strengthening measures. Any horizontal load on the dam body is to be resisted by the shear strength of the foundation rock mass. Thus for a realistic assessment of the stability of the structure against shear and sliding, estimation of the shear resistance of rock joints and concrete-rock contact surface is essential. Present study deals with the assessment of shear strength of foundation to resist the loading on dam. Data of in situ investigation from sixteen dam sites have been included for the study. On the basis of values, shear strength parameters have been obtained in the field and remedial measures have also been suggested for long-term stability of the dams.

Keywords: Rock joints; Shear strength; Deformation modulus; Cohesion; Angle of internal friction.

1. INTRODUCTION

There is full range of planar weaknesses in rock masses with a statistical distribution of spacings and orientations at all scales. At shallow depths, where stresses are low, failure of the intact rock material is minimal and the behaviour of the rock mass is controlled by sliding on the discontinuities. In order to analyse the stability of this system of individual rock blocks, it is necessary to understand the factors that control the shear strength of the discontinuities separating the blocks. The rock mass may contain various types of cracks/planes of weaknesses or discontinuities with varying spacings. When the discontinuities are very closely spaced, the influence of joints/ discontinuities is determined by rock mass failure criterion instead of shear strength of a single critical joint. Rock mass containing large number of closely spaced joints is assumed to exhibit isotropic behaviour and rock mass failure criterion as proposed by Hoek and Brown (1997) is used to assess the rock mass strength.

One of the primary design requirements in the case of concrete or masonry gravity dams built on rock foundations is to ensure adequate factor of safety against shear and sliding failures at the interface of dams and foundation. This is followed by evaluation of shear and sliding stability along existing weak planes, if any, within the foundation. The parameters needed for primary design are the cohesion and angle of internal friction of the dam foundation interface/weak zones of the foundation. For soft rocks, cohesion and angle of internal friction are to be determined for the whole rock mass. The shear strength and sliding angle of friction depend on roughness of the joints. A silent revolution is now going on in India regarding utilization of difficult geological sites for construction of hydro projects (Dwivedi et al., 2012). The present work is proposed for the refinement of the existing knowledge in this field.

2. SURFACE ROUGHNESS AND ITS INFLUENCE

Jointed surface on a hard rock contain undulations and asperities, which have a significant influence on its shear behaviour. Generally, the roughness increases the shear strength of the jointed surface and this strength contributes towards stability of rock structure.

Patton (1966) demonstrated this influence by means of an experiment in which he carried out shear tests on 'saw-tooth' specimens. Shear displacement in these specimens occurs as a result of the surfaces moving up the inclined faces, causing dilation (an increase in opening) of the joint. Larger shear force was required for joint failure, if the dialation was constrained. The experiment clearly indicated increased shear strength due to presence of strong asperities.

The sum of roughness angle, i and the basic friction angle, ϕ_b makes the total friction angle. The first order of roughness of the surface is responsible for ϕ_b and the small bumps and the ripples (asperities) on the surface are responsible for i value. Patton (1966) called small bumps and the ripples (asperities) as the second order of projections. Patton (1966) suggested following bilinear Eq. 1 for shear strength (τ)

$$\tau = \sigma_n \tan(\phi_b + i) \tag{1}$$

where ϕ_b is the basic friction angle of the joint surface; σ_n is the normal stress and i is the roughness angle of the saw-tooth face. The roughness angle i is stress-dependent and also scale-dependent. The basic friction angle is determined by field/laboratory testing. For a basic friction angle of 30^o, the effective roughness angle may vary from 40^o to 50^o for very low stress levels. In fact, one can assume that almost no fracturing of the (very small) second order projections takes place at low normal stress levels and steep sided projections control the shearing process. As the normal stress increases, the second order projections start shearing off and the first order projections take over as the controlling factor. One can imagine that, as the normal stress increases even further, the first order projections will be sheared off and a situation will eventually be reached where shearing takes place through the intact rock material which makes up the projections and the effective roughness angle i is reduced to zero. Fracturing of interlocking surface projections on rock discontinuities is an important factor which has to be considered while attempting to understand the behaviour of actual joint surface.

An alternative approach for predicting the shear strength of rough joints was proposed by Barton (1973). Based upon careful tests and observations carried out on artificially produced rough surfaces in material used for model studies, Barton (1973) derived the following empirical Eq. 2:

$$\tau = \sigma \tan(\phi_b + JRC \log_{10} \sigma_j / \sigma) \quad (2)$$

where *JRC* is joint roughness coefficient, σ_j is joint-wall compressive strength, σ is effective normal stress and ϕ_b is friction angle of joint surface.

On the basis of results of direct shear test results of 130 samples containing variably weathered rock joints, Barton and Choubey (1977) further revised the Eq. 2 and suggested following Eq. 3:

$$\tau = \sigma \tan(\phi_r + JRC \log_{10} \sigma_j / \sigma) \quad (3)$$

where, (ϕ_r) is the residual friction angle of joint surface.

It is evident that in order to obtain shear strength values for foundation design, evaluation of shear strength of rock mass at foundation level is indispensible. If direct shear tests are not conducted, value of shear strength can also be evaluated using Eq. 3, as the value of JRC can be easily assessed by visual comparison. Barton and Choubey (1977) have explained other methods like push and pull and tilt test methods. The choice of the most appropriate method depends upon the nature of problem being investigated. For the dam foundation, no effort would be spared in attempting to obtain reliable shear strength values for critical discontinuities. The critical discontinuities may be in the form of soft layers, bedding planes and weak seams along which chances of sliding are maximum and the required critical shear force is less than the values predicted by the classical Coulomb's or Barton's (1978) theory. Preliminary stability analysis carried out during the feasibility study is generally restricted to the limitation of funds and time, and therefore in such cases, the shear tests may be carried out by portable shear testing machine. Alternatively, correlation of RMR with shear strength parameters suggested by Bieniawski (1984) can also be used in preliminary stage.

3. EXPERIMENTAL TECHNIQUE OF IN SITU SHEAR TEST

A concrete or a masonry dam (Fig.1) built on rock foundation is checked for shear and sliding stability at the dam foundation interface. For different loading conditions, the required factor of safety is specified as per IS6512:1984. In its simplest form, the friction factor criterion (Eq. 4) for unit length is used for evaluating the factor of safety (FoS):

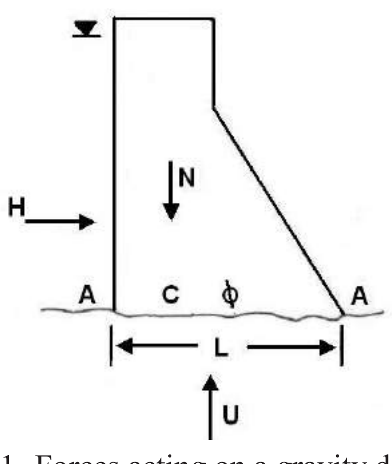

Fig.1- Forces acting on a gravity dam

$$FoS = \frac{\sum\{(N-U)\tan\phi + cL\}}{H} \tag{4}$$

where
N = downward vertical force (kg),
U = uplift force (kg),
H = horizontal force (kg),
ϕ = angle of friction for plane AA (^{o}),
c = cohesion at plane AA (kg/m^2), and
L = base width of the dam (m).

For determining the shear strength of interface or soft layer, following expression proposed by Mohr-Coulomb Eq. 5 has been used:

$$\tau = c + \sigma \tan\phi \tag{5}$$

where τ, c, σ and ϕ are shear strength, cohesion, effective normal stress, and angle of friction respectively. So, cohesion (c) and angle of friction (ϕ) are very important factors which are evaluated from in situ shear tests.

The shear strength of the soft layers and bedding planes control the stability of the dam on stratified foundations. It is necessary to ensure that the dam is safe against shear and sliding failure along its contact with the foundation and also along the bedding planes. According to the code, IS7746:1975, six rock/masonry or concrete blocks are required to be made of size about 70 cm x 60 cm x 30 cm (size may vary based on the availability of space at test locations) on the foundation strata (Fig. 2a). For stratified rock, shear strength parameters are to be determined along the discontinuities. The rock block is carved out from the parent rock mass by careful chiseling up to the plane of weakness and cement slurry is applied to make the surfaces leveled by keeping the surface of interest free for movement. Anchoring and girder arrangements are made to apply normal loading, whereas a concrete reaction wall is built to apply shear load in the open areas (Fig.2b) or reaction is obtained from the unexcavated rock mass.

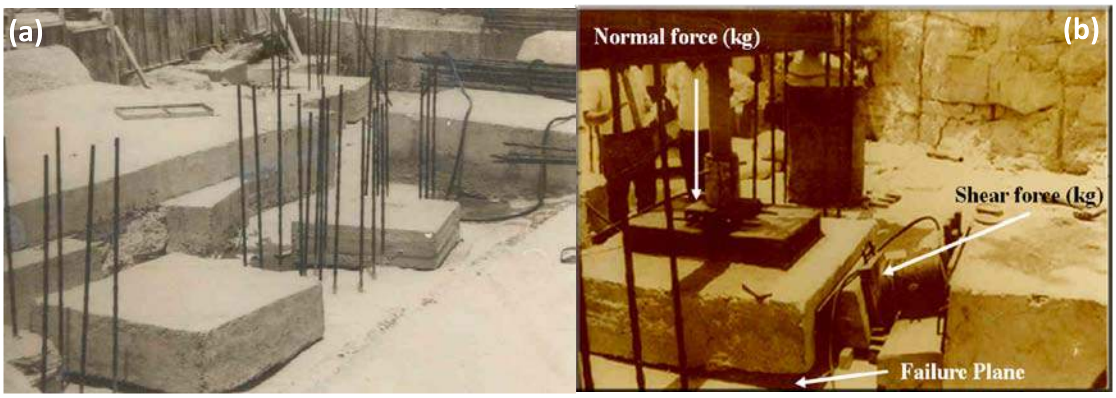

Fig. 2 - (a) Concrete capped rock blocks and (b) application of normal and shear forces on the test block at Rajasthan Automic Power Project (RAPP) site

The testing procedure consists of applying a predetermined normal load on the test block and while maintaining the applied normal load constant, the shear (horizontal) load is applied on the desired plane in increments till the block fails. The hydraulic jack of 200 tonne capacity is used for applying the shear load so positioned as to ensure that the resultant of the normal and shear stresses passes within the middle third of the base of the test block. Single hydraulic jack of 100 or 200 tonne capacity (capacity may vary based on the expected load of the dam body) is used for applying the normal loads. Rollers are introduced below the normal load plate to facilitate smooth movement of the test block during application of shear load. Horizontal displacement corresponding to each increment of shear load is recordrd using two dial gauges. After reaching peak failure stress, each test block is tested under several normal stresses to obtain corresponding residual shear stress. Shear stress, τ is obtained as total shear force per unit area of the sheared surface. Normal stress is obtained as applied normal force per unit area of the sheared surface. Using normal and shear stress values for peak (at failure) and residual (after failure) cases, shear strength parameters are computed to evaluate the cohesion and the angle of internal friction, which are described in the following paragraphs.

3.1 Shear Strength on Planar Discontinuities

When bedding plane is absolutaly planar, with no surface undulations or roughnesses and subjected to normal stress (σ), applied across the discontinous surface, the displacements (u) are caused by the applied shear stress (τ).

Plotting the shear stresses at various shear displacements, at constant normal stress level, results in a type of curve as shown in Fig. 3. At very small displacements, the specimen behaves elastically and the shear stress increases linearly with displacement. As the force resisting movement are overcome, the curve becomes non-linear and then reaches a peak and shear stress reaches its maximum value. Therafter the shear stress required to cause further shear displacement drops rapidly and levels out at a constant value called the residual shear strength.

The peak shear strength values obtained from the tests carried out at different normal stress levels are plotted, as shown in Fig. 4. This curve will approximately be linear, within the accuracy of the experimental results, with a slope equal to the friction angle, ϕ_p and the intercept on the shear stress axis, c_p from which the cohesive strength of the cementing material is determined. This cohesive component of the total shear strength is independent of the normal stress but frictional component increases with increase in normal stress. The peak shear strength is defined by the Eq. 6.

$$\tau_p = c_p + \sigma \tan \phi_p \tag{6}$$

Plotting the residual shear strength againest the normal stress gives a linear relationship defined by the Eq. 7.

$$\tau_r = \sigma \tan \phi_r \tag{7}$$

which shows that all the cohesive strength of the cementing material has been lost. The residual friction angle ϕ_r is usually lower than the peak friction angle ϕ_p. Only one ϕ_p value can be determined by testing one block.

When the discontinuity surface along which the shearing occurs, is inclined at a roughness angle *i* to the shear stress direction, the shear stress is determined by Eq. 8 (Patton, 1966).

$$\tau = c + \sigma \tan(\phi + i) \tag{8}$$

4. TYPICAL GRAVITY DAM FOUNDATIONS AND ASSOCIATED PROBLEMS

After determining the rock mass type and quality, the in situ shear tests can be broadly classified into the following four groups:

(i) Shear strength of hard massive rock foundation interface,
(ii) Shear strength of discontinuity/weak planes in foundation,
(iii) Shear strength of weak rock foundation, and
(iv) Shear strength of heterogeneous rock mass foundation.

4.1 Shear Strength of Hard Massive Rock Foundation Interface

Assessment of the strength of massive rocks (having no weak bedding planes or stratification) at foundation level by qualitative geological analysis and laboratory testing is assumed to be sufficient. The deformability of the foundation does not influence the stress distribution in the dam. The check for the stability of the dam is carried out against shear and sliding failure along the dam - foundation interface.

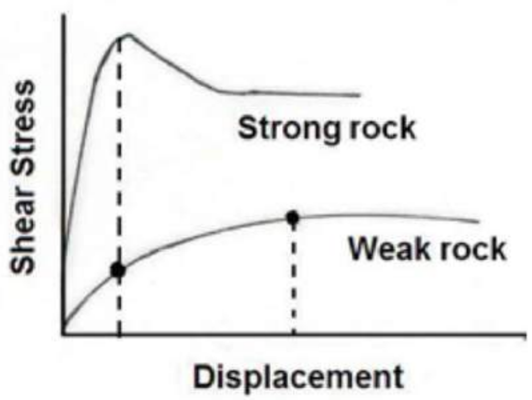

Fig. 3- Shear stress variations

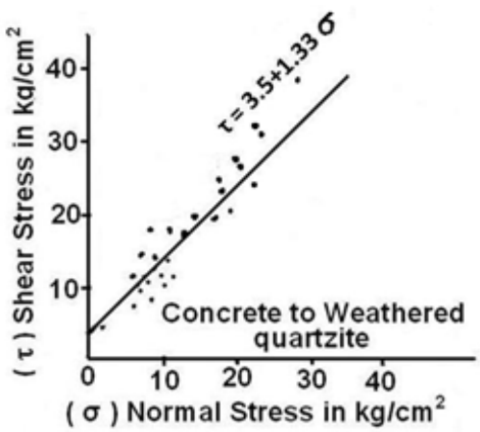

Fig. 4 - Shear strength of dam foundation interface for weathered quartzite

When the stability of the structure is not controlled by the weaknesses within the foundation, weakest link as far as the shear and sliding stability is concerned, is the interface of the structure and foundation. In situ shear test on masonry or concrete blocks cast on foundation strata are carried out using standard direct shear procedure (IS7746: 1975). A minimum of 6 blocks of size varying from 60 cm x 90 cm to 100 cm x 150 cm are tested to evaluate the cohesion and the angle of friction.

Results of tests on concrete blocks cast on fresh hard rock mass having rough surface at Sardar Sarovar Project are plotted in Fig. 5 (CWPRS, 1990). The major failure took place along the contact plane. In case of blocks cast on highly weathered rock mass, the failure was found to be mainly through the rock mass (Fig. 6).

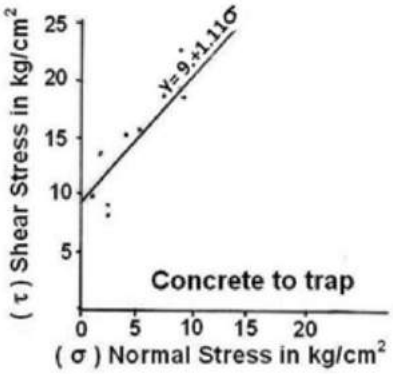

Fig. 5 - Shear strength of dam foundation interface for sound rock

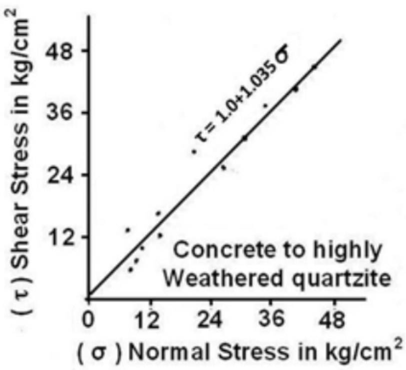

Fig. 6 - Shear strength of dam foundation interface for highly weathered quartzite

4.2 Shear strength of discontinuity/weak planes in foundation

Bedding planes in stratified foundations, shear zones, fault zones, persistent joints, schistose planes are known to have low shear strength. The stability of the structure is influenced by their strength, orientation and locations. It is important to identify all types of discontinuity planes encountered within a given site with particular emphasis on their location, continuity and orientation. Presence of weak layers/seams in the form of persistent joints, bedding planes, clay seams, shear planes, fault planes, red bole breccia layers, shale layers and contact planes of alternate layers of hard and soft strata is by far the most common problem encountered in many dam foundations. The discontinuity planes which are critical to the safety of the structure are identified. Then carving out test blocks with least disturbance and encasing them with concrete or steel mould leaving the plane of interest free. Large number of tests, at least six in number, are to be carried out in each group of rock mass to facilitate selection of viable design parameters. A typical case of Rajasthan quartizitic sandstone (Fig.7), (CWPRS, 1990) is considered herein from Rajasthan atomic power project (RAPP) site. Six blocks of different sizes 100cm x 100cm to 130cm x 130cm were carved out from stratified quartzitic sandstone layers and were tested in situ.

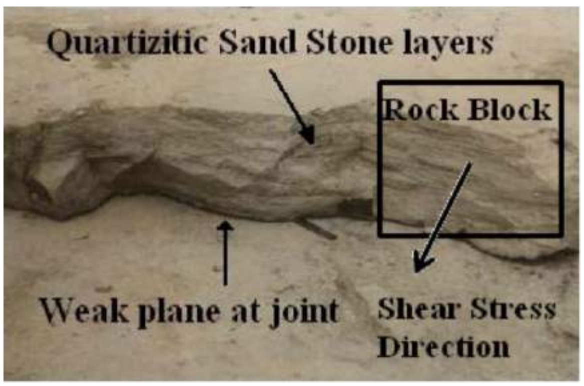

Fig.7 - Stratified quartzitic sandstone layers before carving out blocks at RAPP site

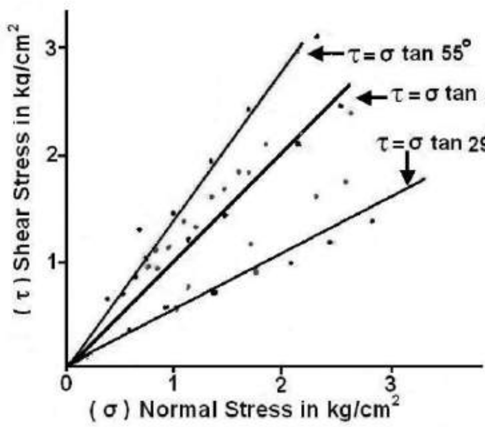

Fig.8 - Shear stress vs normal stress plot for RAPP site

The plot of normal stress and the corresponding shear stress of all the six blocks is shown in Fig.8. The scatter of points in Fig.8 shows that the quartzitic sandstone layers are rough and the degree of roughness varies from layer to layer and from location to location on the same plane. The minimum, maximum and mean values of friction angle work out to be 29°, 55°, and 45° respectively. The minimum friction angle of 29° is a representative value of a clean planar joint. Considering the importance of structure, a friction angle of 29° was chosen for design purposes.

Tawa, Srisailam, Barna, Dudhganga, Kadana, Sardar Sarovar, Hidkal, Bennithora, Amarja and Kodasalli are some of the dam sites where the problems of weak planes within the foundation have been encountered. The shear strength depends on the orientation of the weak planes, the depth at which they are located below the foundation level and the quality of the overlying rock mass. Examples of Amarja and Kodasalli dams have been explained with measures to improve the shear strength of the foundation strata.

4.2.1 Amarja and Kodasalli dams

A horizontally aligned soft clay seam encountered at a depth of 7.5m below the foundation level of 32 m high spillway structure of Amarja dam is shown in Fig. 9 (CWPRS, 1988). Downstream dipping thin shear zone encountered at a depth of only 4m below the foundation level of one of the blocks of Kodasalli dam is shown in Fig. 10 (CWPRS, 1994). These weak seams were analysed using finite element method (FEM).

Factor of safety (FoS) against shear and sliding failure along the weak seam, evaluated by conventional as well as finite element analyses are compared in Figs. 11 and 12 for both Amarja and Kodasalli dams. FoS obtained by finite element method (FEM) are based on the weighted average of several FoS values along the plane of analysis.

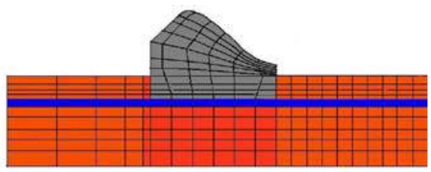

Fig. 9 - FEM model for Amarja dam

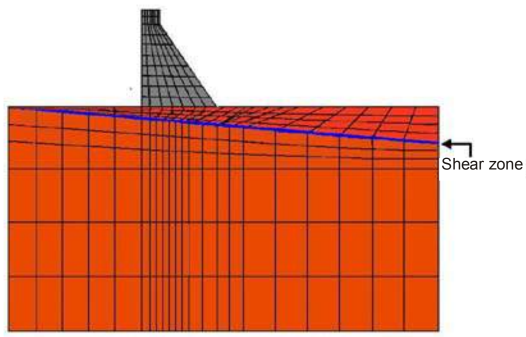

Fig.10 - FEM model for Kodasalli dam

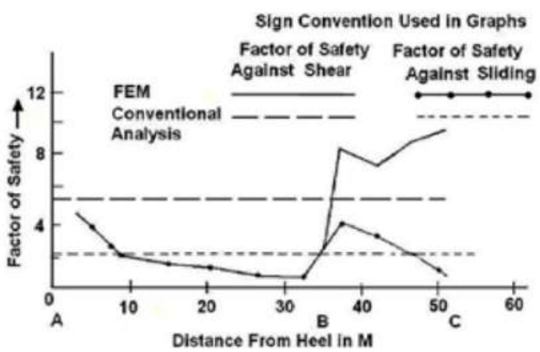

Fig.11- Factor of safety along plane ABC for Amarja dam

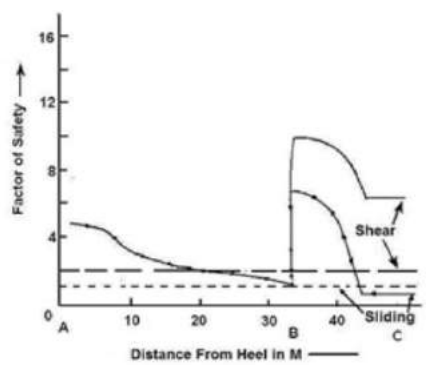

Fig.12 - Factor of safety along plane ABC for Kodasalli dam

As shown above, a better approach is to use the stress-strength relationship at all points of interest and evaluate point to point FoS along the surface. This is an improvement over the overall FoS value criterion, for identification of specific unstable locations for optimization of remedial measures.

In the case of foundation of Dudhganga dam, Maharashtra the distinct bands of shale with thickness of about 0.5 m have been identified in thinly bedded quartzite rock mass (CWPRS, 1981). In situ shear tests were conducted on shale. The linear Mohr-Coulomb envelope was not the best fit to the experimental data. In the case of soft rock masses and filled discontinuity planes containing soft material, the intensity of normal stresses influences the shear strength significantly. The Mohr-Coulomb straight line may not be best fit to the experimental data. It is better to fit an empirical power law (Fig.13), which directly determines shear strength, the value of requied final parameter without calculating the friction angle.

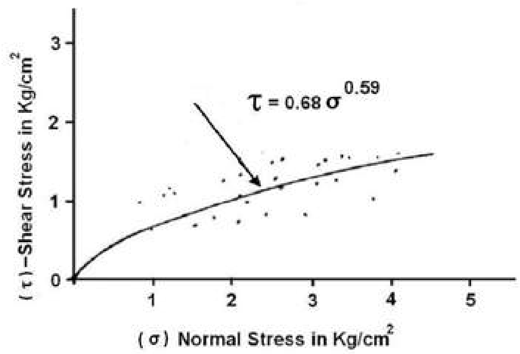

Fig. 13 - Influence of normal stress on shear strength

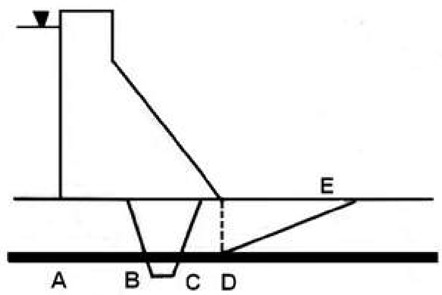

Fig.14 - Shear key across soft layers

Provision of shear key, thrust blocks, shear plugs and pre-tensioned anchors are some of the common remedial measures adopted to realize adequate factor to safety against shear and sliding failure along the weak seams. A standard remedial measure in case of interbedded soft rocks is to provide a shear key (Fig. 14).

Another important aspect, which needs to be considered while calculating the shear friction factor, when a shear key is provided, is the deformation compatibility of the different materials contributing to the shear strength along the plane of analysis. The shear deformation undergone by the stiffer shear key material has to be taken as a limiting deformation. For computing the limiting deformations, the shear strength and passive resistance of shale and down stream rock are used for computations.

However, provision of these remedial measures should be preceded by knowledge of the influence of the weak seam on the stability of the dam. The procedure used for evaluating the factor of safety along dam-foundation interface is not adequate to evaluate FoS along weak plane within the foundation. Biplanar analysis based on limit equilibrium is used to evaluate the FoS against shear and sliding failure along the weak plane. This analysis is based on the simplified stress distribution in the dam foundation system. It is possible to carry out a rigorous analysis using finite element model to obtain realistic stress distribution and evaluate the factor of safety at each and every point along the weak plane and optimize remedial measures to achieve economy without compromising on safety. A conventional analysis of a dam foundation and the biplanar approach used to evaluate the factor of safety against shear (shear friction factor or SFF) along a weak plane is presented in Fig.15.

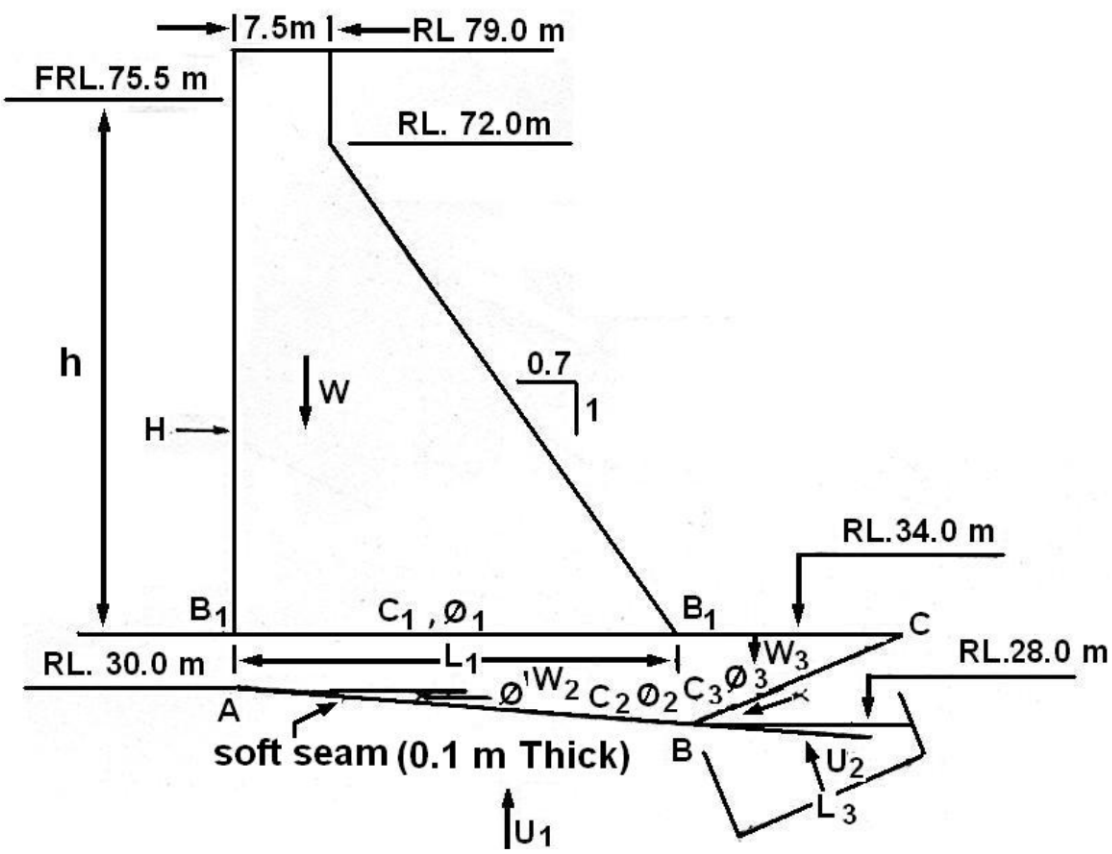

Fig.15 - Biplanar analysis for dam foundation weak planes (conventional analysis)

For stability on dam foundation interface, plane B1-B1 (Fig. 15), Joshi et al. (1989) suggested Eq. 9.

$$SFF = \left[(W - U_1)\tan\phi_1 + C_1 L_1 \right] / H \tag{9}$$

For stability on plane ABC (Fig. 15), Joshi et al. (1989) suggested Eq. 10,

$$SFF = \frac{(W\ W_2 - U_1) tan\, \phi_2 + C_2 L_1 + W_3\, tan(\phi_3 + \alpha) + \dfrac{C_3 L_3}{Cos\alpha (1 - tan\, \phi_3\, tan\, \alpha)}}{H} \tag{10}$$

4.3 Shear Strength of Weak Rock Foundation

In the case of fractured, weathered and soft foundation rock, the shear strength of the rock mass as a whole would be the controlling factor in ensuring stability of structure built on such foundations. In a conventional design analysis wherein the shear and sliding stability of the rock mass is the only property required. Rock blocks of size varying from 60 cm x 30 cm to 150 cm x 60 cm are carved out of the foundation rock. About six rock blocks are freed from surrounding rock mass on the four sides keeping its contact with the plane of interest. Then these blocks are encased by concrete or steel cap, keeping the plane of interest free. The plots of shear test results in weak rock mass at Dudhganga dam site (CWPRS, 1981) and Tons barrage site (CWPRS, 1986) are shown in Figs. 16 and 17.

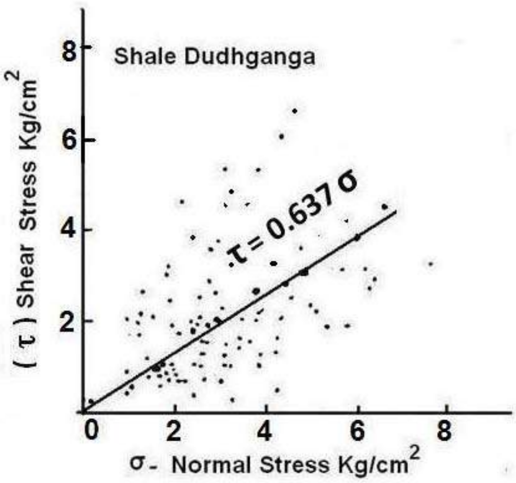

Fig. 16 - Shear test results of shale, Dudhganga site

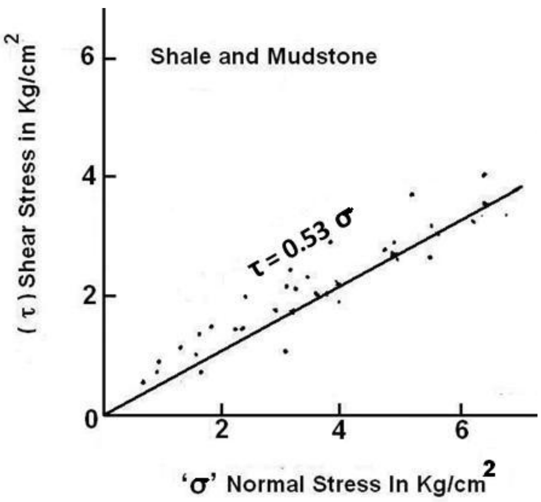

Fig. 17 - Shear test results of shale and mudstone, Tons barrage site

Mudstone, shale, claystone, schist, slate etc. are some of the typical soft rock formations falling in this category (Ramegowda et al., 1986). Strong rocks could also become soft due to disintegration by weathering. Building of relatively stiffer concrete/masonry structures on softer foundations has increased in the recent past. This problem is felt more in the case of spillways and head regulator structures, which are required to be built as part of composite dams.

In the case of these foundations, in addition to ensuring adequate factor of safety against shear and sliding failure along the dam-foundation interface, it is necessary to examine the influence of the soft foundation on the stress distribution in the body of the dam. Knowledge of expected large deformation due to very low moduli of the foundations was not considered to be adequate to check the stability of the structures. In view of the fact that the safety criteria are based on stress conditions at limit equilibrium, the knowledge of stresses developing in the body of the dam as a function of the operating loads, as well as the influence of foundation stiffness are also important parameters which play an important role in deformation. Normal and shear stiffness of joints are basic parameters which are input parameters to any numerical code for analysing failure strain of structure in jointed rock mass. The Hirehalla dam spillway and Bansager canal head regulator, constructed in soft rocks, are analysed by FEM as presented below.

4.3.1 FEM analysis of Hirehalla dam spillway and Bansagar canal head regulator

The spillway of Hirehalla dam is built on a foundation, subjected to deep weathering (CWPRS, 1997) and 40 m high Bansagar canal head regulator is built on a soft foundation consisting of heavily jointed and flaky shale (CWPRS, 1996). Analyses using FEM were carried out to examine the influence of modulus of deformation of the foundation rock (Figs.18 and 19). Figures 20 and 21 show the influence of the foundation stiffness on the stress distribution in the two dams, when the ratio of dam concrete modulus (E_D) to that the foundation rock modulus (E_F) varies from 1 to 60. With the knowledge of the tensile stress values and the zone under tension, it is possible to design necessary reinforcement.

4.4 Shear Strength of Heterogeneous Rock Mass Foundation

Foundations consisting of several rock formations of widely varying stiffnesses in different parts of the zone of influence of the dam cause the problem of uneven settlement and influence the stress distribution, both in the body of the dam and the foundation. The determination of shear strength for different type of rock masses becomes difficult. So stiffness of different rocks existing in the foundation is determined by plate load and flat jack tests (ISRM, 1986) and stress analysis by FEM is carried out. To understand the influence of heterogeneous rock mass foundation, the cases of Supa and Lakhwar dams are discussed.

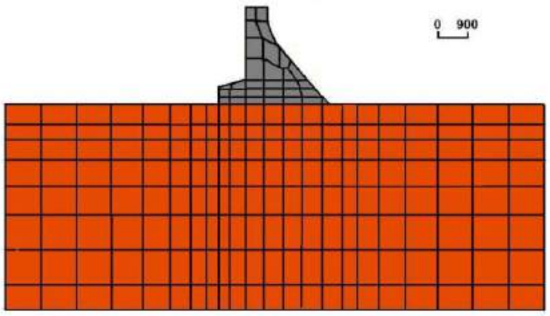

Fig.18 - FEM model for Bansagar head structure

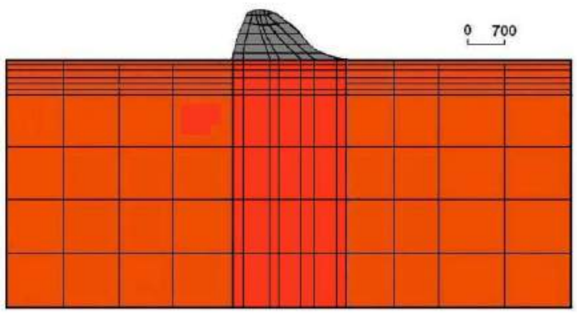

Fig.19 - FEM model for Hirehalla dam spillway

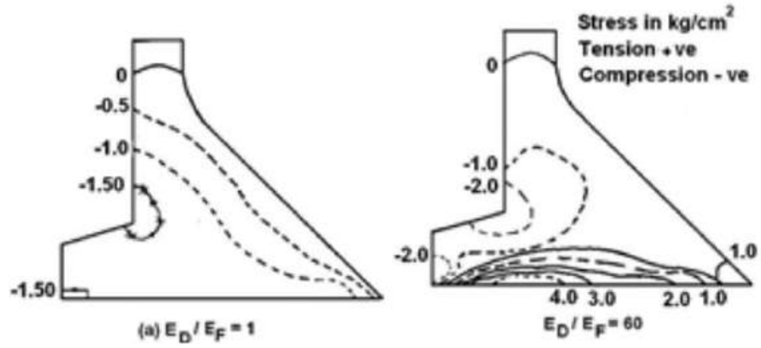

Fig.20 - Major principal stress contours for Bansagar head regulator structure

4.4.1 Supa dam

The foundation of the 101 m high Supa dam in Karnataka is a good example of heterogeneous foundation (CWPRS, 1994). The strata encountered ranges from pulpy clay to very sound magnetite quartzite. The dam rests on a heterogeneous foundation and Finite Element 2D model (Fig. 22) shows the location of different type of rock strata in the foundation. The dam was designed using a combination of conventional and advanced approaches after adopting different remedial measures. As a part of the monitoring programme, the deformation behaviour of some of the instrumented blocks of the dam has been studied using FEM. The deformation pattern of block no. 5 of the dam resting on heterogeneous foundation with rock deformation modulus varying from 0.001 x 10^5 to 1.4 x 10^5 kg/cm^2 is shown in Fig. 23. The top level of the dam block deforms 13.3 cm horizontally towards upstream side of the dam (Fig. 23). The reduction to 3cm horizontal deformation in the top level of the dam block is seen after providing the treatment for improving the deformation modulus of 0.001 x 10^5 kg/cm^2 to 0.2 x 10^5 kg/cm^2 by grouting and anchoring arrangements (Fig. 24).

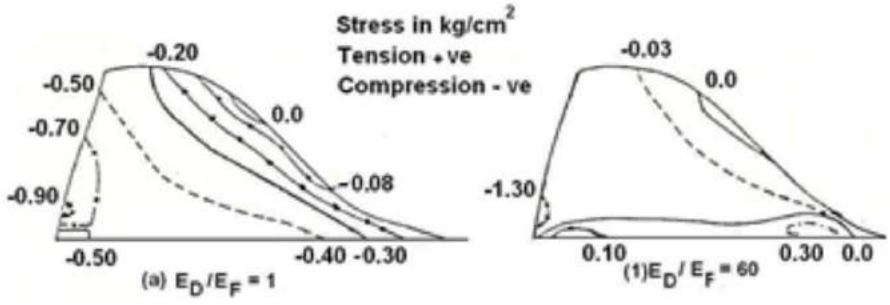

Fig.21- Major principal stress contours for Hirehalla dam spillway

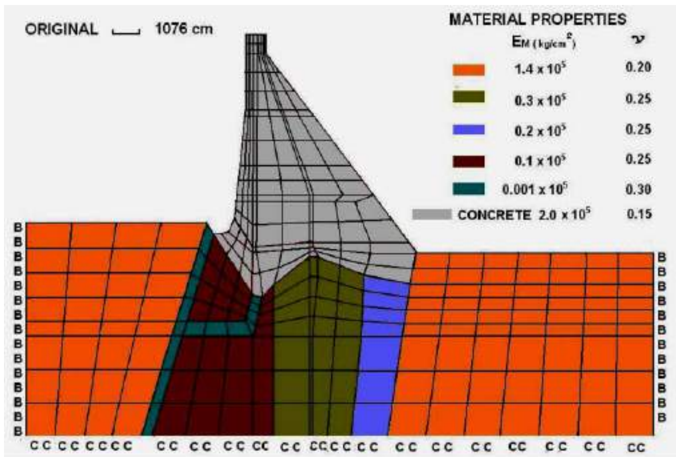

Fig. 22 - Heterogeneous foundation of Supa dam

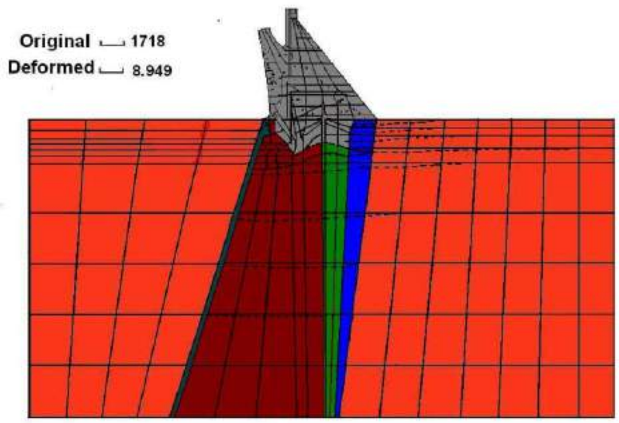

Fig.23 - Deformed shape of Supa dam with heterogeneous foundation

4.4.2 Lakhwar dam

The 200 m high Lakhwar dam in Uttar Pradesh (now in Uttarakhand state) was built on a highly heterogeneous foundation consisting slates, quartzite, quartzitic slates and Jaunsar traps type rock formations with modulus of deformation ranging from 50MPa to 19600MPa (CWPRS, 1993). The configuration of different rock types and the deep gorge forming the dam site necessitated use of a three dimension finite element model (Fig. 25). The soft slate underlying the relatively stiff trap does not adequately participate in sharing the imposed loads and as a consequence the stiff trap gets subjected to very high tensile stresses. Influence of the heterogeneous foundation on the stress distribution in the superstructures as well as in the foundation is shown in Figs. 26 and 27. The location of the foundation of the dam was modified and the tensile stresses and zones under tension were brought down to acceptable limits.

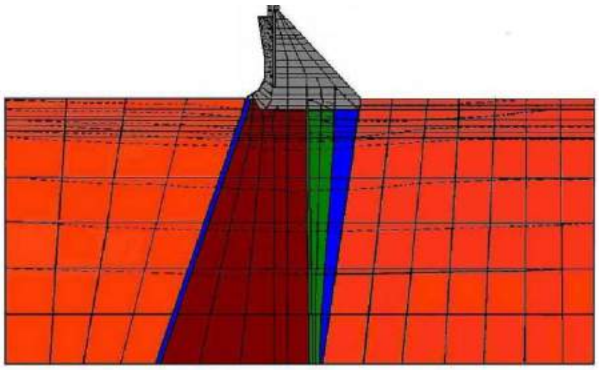

Fig.24 - Deformed shape of Supa dam with uniformly homogenous foundation

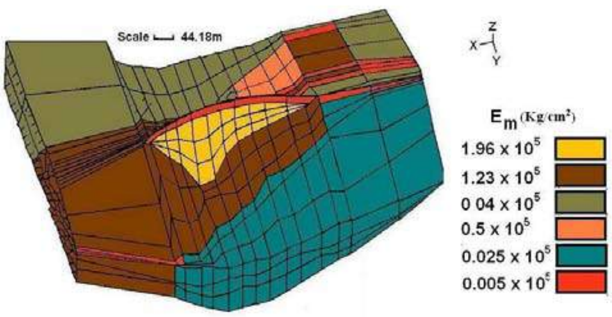

Fig.25 - Three dimensional finite element model of Lakhwar dam

5. DISCUSSION

As per the convention the failure in poor rock formations would be through rock. In the case of hard rocks it would be through concrete or interface of rock and concrete, therefore almost full shear strength of the concrete would be mobilised. In situ studies have shown that while the former assumption is true, the full shear strength of concrete is not mobilized in the case of the later. The average values of friction angle on rock to rock interfaces were higher than that on concrete to rock interfaces. The value of cohesion is found to be low for concrete to rock shear tests and the probable reasons are as following:

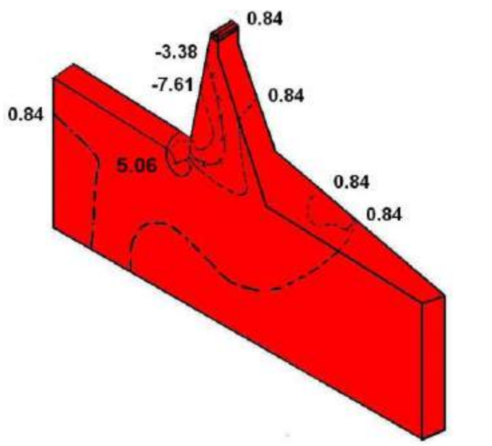

Fig.26 - Major principal stress contours (heterogeneous foundation)

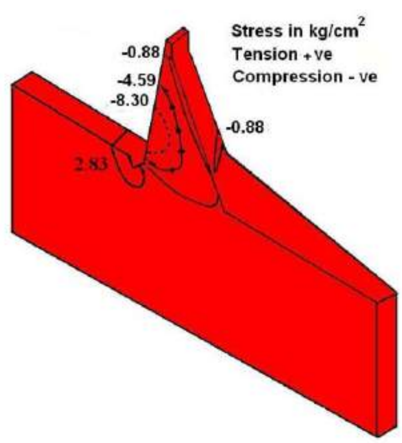

Fig.27 - Major principal stress contours (uniform foundation)

The shear strength of the concrete and rock surface is governed by the amount of keying of concrete into rock. The maximum permissible limit of keying is approximately 50 percent of the contact area. The upper limit of mobilised strength is the average shear strength of keys and unkeyed portion, provided stress deformation characteristics of both systems are same. This is perhaps not the case as the key part can be expected to be stiffer than the unkeyed position. Thus failure may start at the keyed part earlier than the unkeyed one.

Roughness due to natural depressions controls not only the peak shear strength at low normal pressures but the shape of the shear stress versus shear deformation curve and the rate of dilatancy. Natural depressions in the rock surface are such that their sides make small angle of inclination to the direction of shear stress. The failure may occure along the side surface of such depressions with block riding over the inclined surface, instead through the keyed parts, unless the normal stress is very high (Barton, 1978). This would cause low cohesion and high effective angle of friction. In addition to the types of difficult foundations discussed above, there could be individual features needing attention and treatment measures. Some of the major treatments carried out are dental treatment, consolidation/curtain grouting, cast in situ diaphragm walls in the foundation. Some of the shear strength parameters for major projects tested by CWPRS for dam-foundation interface and weak rock masses and discontinuity planes are given in Table 1 and 2 respectively. Shear strength parameters for concrete on rock mass and masonry on rock mass foundation interfaces are given in Table 1 of eleven and three projects respectively. When concrete structure on rock foundation is to be constructed then blocks of concrete on rock mass are to be tested for determining shear strength parameters for concrete rock interface. When masonry structure on rock foundation is to be constructed then blocks of masonry on rock mass are to be tested for determining shear strength parameters for masonry rock interface. The same quality of concrete mix and brick masonry to be used for the structure has to be used for making blocks for testing purposes. For weaker rocks, the shear strength parameters of dam-foundation interface show that the shear failure is taking place through rock or partly contacts/partly rock. For stronger rocks the shear failure is taking place through contacts of rock and concrete/masonry. Table 2 shows results of shear strength parameters of dam and foundation interface obtained from the direct shear tests of foundation rock mass for ten projects. It is seen that shear failure is taking place through rock for weak jointed rock mass. For strong and hard jointed rocks, on the other hand, the shear failure takes place through joints.

Table 1 - Shear strength parameters of dam-foundation interface

CWPRS report number (year)	Project name	Rock type	Failure plane	Cohesion (MPa)	Friction angle (°)
Concrete on Rock Mass					
Nil (1966)	Tawa	Sand Stone	Mainly rock	0.50	44
Nil (1973)	Kadana	Quartzite and Phylite	Contact	0.75	52
1516 (1975)	Salauli	Quartzite, Metagreywacke	Contact	0.53	46
1828 (1979)	Supa	Weathered Banded Megnetite Quartzite	Partly contact Partly rock	0.35	53
		Highly weathered Banded Magnetite Quartzite	Mainly rock	0.10	46

1990 (1981)	Dudhganga	Quartzite	Contact	0.40	50
2350 (1986)	Kodasalli	Gneiss	Contact	0.70	45
2447 (1987)	Bodhghat	Gneiss and Metabasics	Contact	0.90	50
2628 (1989)	Bansagar	Schist	Partly contact Partly rock	0.45	49
2809 (1990)	Sardar Sarovar	Trap	Contact	0.90	48
2983 (1992)	Srisailam	Quartzite	Contact	0.50	43
Masonry on Rock Mass					
Nil (1972)	Bargi	Basalt	Contact	0.20	45
1516 (1975)	Salauli	Quartzite, Metagrey-wacke and phylite	Contact	0.35	47
1990 (1981)	Dudhganga	Quartzite	Contact	0.40	60

Table 2 - Shear strength parameters of weak rock masses and discontinuity planes

CWPRS report number (year)	Project name	Rock type	Failure plane	Cohesion (MPa)	Friction angle (°)
Nil (1966)	Tawa	Sand stone-shale	Contact	0.07	32
Nil (1973)	Kadana	Quartzite and Phylite	Contact	0.0	18
1465 (1975)	Hidkal	Quartzite – shale	Contact	0.0	26
1990 (1981)	Dudhganga	Shale	Mainly rock	0.02	32
2304 (1986)	Subansiri	Disintegrated gneiss	Mainly rock	0.40	44
2628 (1989)	Bansagar	Weathered Granite Jointed porcellanite	Mainly rock	0.04	40
		Fresh, soft shale schistose plane	Partly contact Partly rock	0.04	34
2752 (1990)	RAPP 3&4	Quartzitic sand stone bedding plane	Contact	0.0	29
2983 (1992)	Srisailam	Quartzite – shale	Partly contact Partly rock	0.05	41
3127 (1994)	Bhadra	Sand stone weathered Talc	Partly contact Partly rock	0.03	37
3458 (1997)	Hirehalla	Chlorite Schist	Partly contact Partly rock	0.02	33

6. CONCLUSIONS

The in situ shear test results for the tested dam foundations give a guideline for adopting the results for preliminary investigation stage for new projects in the nearby area with same type of rock strata. The variations in data from different project sites or at the same site make it necessary to conduct the in situ tests separately at each site and rock type. Though the field tests are difficult to conduct and are time consuming and costly, it is always advisable to conduct the necessary in stu tests in pre as well as post construction stages.

In a given site, there could be several zones of rock with different characteristics. For quantitative assessment the foundation zone is to be divided into different zones according to rock mass rating of Bieniawski (1984) and Q system of Barton et al. (1974) classification systems. Identification and characterisation of unfavourably oriented discontinuity planes are of critical importance. Once the rock mass is characterised into reasonably homogeneous zones and weak links are identified, the next step is to conduct in situ tests by adopting one of the four methods described to obtain reasonably reliable properties of the each of these zones for all important structures. Average values for a particular dam site may be obtained by plotting data rather than taking an arithmetic average of cohesion and friction values. Systematic investigation is very crucial by considering all factors which influence the results with reasonable accuracy. It gives the designer required design parameters for the design of the important structures.

Acknowledgements

The author is grateful to Shri S. Govindan, Director, Shri R. S. Ramteke, Additional Director and Shri P. C. Pethe, Scientist 'E' for encouragement and according permission to publish this work. The assistance of Shri J. M. Deodhar, Laboratory Assistant-II is acknowledged for his help during the laboratory and field investigations.

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