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DAM FOUNDATION AND ITS TREATMENT

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Abstract- India is endowed with rich hydro-power potential but till date, only about 23% of the country's vast hydro-power potential has been harnessed. The major components of a hydro-power project are: dam, intake structure, head race tunnel, power house, tail race tunnel, etc. Among these, dam applies significant loads to the underlying geologic media and disturbs the pre-existing natural conditions. The interaction of the dam with the foundation, especially one with difficult or troublesome geology, needs greater attention and analysis. Numerical models, particularly finite element models, are routinely used in the analysis of dam-foundation systems under normal operating conditions. However, in case of foundations having joints and shear seams, discrete element models that incorporate deformable blocks are preferable.Not many systematic studies have been carried out to understand the stress-deformation response of dam-foundation system founded on rock mass and the weak rock foundations. The paper presents the review of the systems being used for analysing the dam foundation interaction and the different foundation treatments used for the weak dam foundations.

I. INTRODUCTION

India is fortunate to be endowed with all primary energy sources like solar, wind, coal, petroleum, lignite, hydro-power, nuclear power, etc. However, among these, hydro-power is the only renewable source of energy and has been recognized to be economical and preferred source of electricity due to its various inherent benefits. India is endowed with rich hydro-power potential; it ranks fifth in the world in terms of usable potential distributed across six major river systems.

Features of the hydro-electric projects, being site specific, depend on the geology, topography and hydrology at the site. In spite of detailed investigations carried out at the time of preparation of Detailed Project Report, there is still possibility of hydro-electric projects facing some uncertainties during implementation of the project. Geological surprises may result in taking mitigation measures, which could increase the project cost. Construction time of a hydro-electric project is greatly influenced by the geology of the area and its accessibility. Even when extensive investigations using state-of-the-art techniques are undertaken, an element of uncertainty remains in the sub-surface geology. The geological surprises during actual construction cannot be completely ruled out which result in cost escalations and time over-runs.

The major components of a hydro-power project are: dam, intake structure, head race tunnel, power house, tail race tunnel, etc. Among these, dam constitutes to be a large surface structure and applies significant loads to the underlying geologic media and disturbs the pre-existing natural conditions. Despite the apparent favourable stability conditions for the structures founded on the strong rock, there are unfortunately instances of the foundation failures. Failures may include excessive settlements due to the presence of undetected weak seams or deterioration of rock with time. Faults or seams not only change the physical properties of the rock in and near the discontinuous zone, but they substantially affect the distribution of stresses and the overall stability of the dam-foundation system. Because of low modulus values, these geological complexities may create zones of stress concentration and also become potential sliding paths. Numerical models, particularly finite element models, are routinely used in the analysis of the global mechanical and hydraulic behaviour of dam-foundation systems under normal operating conditions. However, in case of foundations having such complex and potentially weak zones as shear seams, discrete element models that incorporate deformable blocks are preferable, since structural behaviour may be strongly influenced by the rock mass deformability (Lemos, 1999).

Construction of dams forms an invariable part in the development of hydropower potential. Gravity dams form the second largest percentage of the dams built in the world as well as in India. The main characteristic of gravity dams is the development of high stresses in the foundation due to the large loads imposed by the dam body and the reservoir. This condition forces the designer to evaluate the condition of the foundation so that the

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dam is founded on competent, sound rock of sufficiently high strength and appropriate stiffness over most of the foundation area. In fact, the foundation has a threefold function, namely (i) to provide stability, (ii) to provide sufficient stiffness for limiting deformations within acceptable behaviour patterns under the weight of the dam and the forces acting on it, and (iii) to provide control of seepage, both in-flow, through adequate water tightness, and principally in uplift pressure and erosive stress (ICOLD 2005). Though the superstructures of the projects are constructed under strict quality control and as per the design and construction specifications, the foundation geology and geomechanics are always unknown to some degree, and are subjected to heterogeneities and complex pattern. Foundation investigations have revealed that complex geological conditions encountered in dam foundations have always resulted in a weaker rock foundation having elasticity values half to one third of the dam body. In India, many dams are being constructed in the Himalayan region where the rocks are not massive and have mostly criss-cross joints and layered strata with different elastic properties (Varshney, 1974). Thus, the rock mass encountered in the dam foundation may have a large variation in the values of elastic modulus in comparison with the modulus of the dam concrete.

The interaction of the dam with the foundation, especially one with difficult or troublesome geology, needs greater attention and analysis. When joints filled with clayey materials predominate or a discontinuity of major thickness or a shear seam is encountered, the usual practice is to resort to spot excavations backfilled with concrete. The design philosophy is that as soon as such an element is constructed, it acts as a reinforcing key that allows the transfer of compressive stresses and is able to withstand shear stresses.

Not many systematic studies have been carried out to understand the stress-deformation response of dam-foundation system founded on rock mass with different foundation modulus values. The literature is also lacking detailed studies on the influence of various inclinations of the set of joints present in the dam foundation on the stress-deformation response of the dam-foundation system. Systematic studies to define the effective depth of concrete plug to be provided for the treatment of dam foundation with shear seam are also not available except for the practical studies carried out at Shasta and Bhakra dam projects.
The present paper presents the method of dam-foundation interaction analysis and the availability of various treatment techniques.

II. DAM-FOUNDATION INTERACTION

For centuries, the dam engineer has been designing dams on the basis of past success and more importantly, past failures. Today with the advancement in technology, engineers are gaining confidence in the design and construction of dams and it is possible to make use of almost any site. However, every project's behaviour is basically dependent on the supporting ground. Difficult or problematic geologic conditions can be characterized by seams, joints, adversely dipping rock beds, highly fractured and sheared ground, heterogeneous and highly permeable alluvial deposits, presence of fault zones, unusual in-situ stresses and karsts soluble evaporites such as gypsum (ICOLD, 2005).

Faults and other major discontinuities which pass through the dam foundation are often serious deficiencies. Such features result in greatly reduced shear strength, i.e. approximately equal to the residual value. Seepage through a fault zone and its adjacent shear zones can later lead to piping and/or uplift if the fault has not been treated properly. Weak strata and seams that tend to form foliations represent potential failure zones, even before any relative displacement has taken place. Faults or seams not only change the physical properties of the rock in and near the discontinuous zones, but they substantially affect the distribution of stresses in the dam-foundation and the overall stability of the system. These geological complexities may create stress concentration zones in the dam-foundation structure because of their low value of Young's Modulus and may be the potential sliding paths in case of sub-horizontal seams and faults. A critical zone in dam foundation is the contact between the dam body and the natural ground. The transition from the material of the dam body to that of the foundation represents a major discord and therefore also a disharmonious zone that, if not properly dealt with, can compromise the safety of the entire project. Dam-foundation interaction as a result of abrupt differences in the stiffness of the foundation materials along the dam axis and/or sharp irregularities in the foundation base, can lead to internal stress distributions which can result in fracturing and the phenomenon of arching (ICOLD, 2005).

II.1. Conventional method for dam foundation analysis

For consideration of stability of a concrete dam, the following assumptions are made (IS: 6512, 1984):

- The dam is composed of individual transverse vertical elements each of which carries its load to the foundation without transfer of load from or to adjacent elements. However for convenience, the stability analysis is commonly carried out for the whole block.

- The vertical stress varies linearly from upstream face to the downstream face on any horizontal section.

The design requirements for stability of concrete gravity dams according to IS:6512 (1984) are:

- The dam shall be safe against sliding on any plane or combination of planes within the dam, at the foundation or within the foundation;
- The dam shall be safe against overturning at any plane within the dam, at the base, or at any plane below the base; and
- The safe unit stresses in the concrete or masonry of the dam or in the foundation material shall not be exceeded.

Depending upon the scope and details of the various project components, site conditions and construction programme, one or more of the following loading conditions may be applicable and may need suitable modifications. The seven types of load combinations are as follows (IS: 6512, 1984):

1. Load combination A (construction condition): Dam completed but no water in reservoir or tail water;
2. Load combination B (normal operating conditions): Full reservoir elevation, normal dry weather tail water, normal uplift, ice and silt (if applicable);
3. Load combination C: (Flood discharge condition) - Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift, and silt (if applicable);
4. Load combination D: Combination of A with earthquake;
5. Load combination E: Combination B, with earthquake but no ice;
6. Load combination F: Combination C, but with extreme uplift, assuming the drainage holes to be inoperative;
7. Load combination G: Combination E but with extreme uplift (drains inoperative).

The characteristics of a dam foundation are analysed with respect to their effects on (ICOLD, 2005):

- Stability of the foundation, including abutments and adjacent slopes;
- Deformation of the foundation, such as differential settlement and distortions causing cracks and concentrated leaks; and
- Uplift, hydraulic gradient, internal erosion and piping.

For dams resting on rock foundations, planes of weakness and discontinuities (joints, fractures, shear zones, weak beds or seams) may cause instability and movements under the dam, in the abutments or adjacent slopes. The strength along a plane of weakness is governed by the rock type, the joint roughness characteristics, the extent and the type of infilling, the effective normal stress across the plane and the magnitude of deformation relative to that at peak strength.
The modelling of uplift pressures within a dam, on the foundation on which it is constructed, and on the interface between the dam and foundation is a critical aspect in the analysis of concrete gravity dams.

Evaluating the safety of concrete gravity dams against sliding requires an understanding that rock foundation and the structure above it are an interactive system whose behaviour is controlled by the mechanical and hydraulic properties of concrete material and rock foundations.

II.2. Computational methods for dam foundation analysis

A number of computational methods of analysis have been developed over the past five decades. They have become popular due to rapid advancements in computer technology. Before the advent of computers, the structures were designed largely using analytical methods or rules of thumb. The increased consciousness amongst the public regarding the safety and economy has led the engineers to seek more rational solutions to the problems. Two approaches to model the dam-foundation system can be identified, both recognizing geological structures as being discontinuous due to joints, faults and bedding planes. A continuum approach treats the rock mass as an equivalent continuum, while a discontinuum approach views the rock mass as an assemblage of independent blocks. If material along the failure surface exhibits brittle behaviour (i.e. strain softening from a peak strength value to a residual value), finite element analysis may be required. For dams on rock foundations, a critical requirement in the stability analysis is the localization of planes of weakness or discontinuities that combine to give a kinematically possible sliding block mechanism (ICOLD, 2005).

Continuum theories, which use elasticity, plasticity, creep and anisotropy, are considered valid when the scale of joint spacing is small as compared to the size of the rock mass under study. The discontinuous character of the rock mass is taken into account by considering the properties of an "equivalent" continuum (e.g. reduced moduli of elasticity, marked anisotropy, etc.). Explicit modelling of discontinuities in the finite element analysis is done by incorporating special interface elements to simulate the behaviour of joints, shear zones and faults (ICOLD, 1993).

The Finite Difference Method (FDM), Finite Element Method (FEM) and Boundary Element Method (BEM) are based upon the continuum approach. The FEM is uniquely capable of handling complex geometries and non-homogeneities, and material and geometric nonlinearities (Desai and Abel, 1972; Bathe, 1982; Zienkiewicz and Taylor, 1989). The method involves discretisation of the entire region of interest. In the boundary element method (BEM), the interior or exterior boundaries only are discretised (Banerjee and Butterfield, 1981; Crouch and Starfield, 1983). The method is most appropriate for modeling linear-elastic systems. The boundary element method provides an economical means of two-and three-dimensional analysis of rock masses and is particularly suitable for use when conditions at the boundary are of most concern.Discontinuum models feature numerical procedures involving the equations of blocks rather than continuum. Cundall (1971) was among the first to implement the discrete element method (DEM), also called as distinct element method, to represent rock mass as an assembly of discrete blocks where joints/discontinuities are viewed as interfaces between distinct bodies.

II.3. Distinct element analysis

A jointed rock mass is better modelled by a discontinuum approach using a distinct element method for discontinuum modelling. Here the discontinuous medium is represented as an assemblage of discrete blocks. By modelling the actual structure of rock mass made up of individual blocks, it simulates the real conditions in a satisfactory manner (Lin and Fairhurst, 1988). Cundall originated the distinct element codes (UDEC in 2-D and more recently 3DEC in 3-D), which analyse both small strain conditions and large displacements (Hart et al., 1988). The key block method is of the same family, which basically considers the kinematics of the problem and studies the limit state conditions (Goodman and Shi, 1985; Goodman et al., 1982). Similarly the method proposed by Londe et al. (1969, 1970) mainly for surface rock masses, models the basic structure of the site as mapped by the geologists.

Universal Distinct Element Code (UDEC) is a two-dimensional numerical analysis program based on the distinct element method for discontinuum modelling. UDEC simulates the response of discontinuous media (such as a jointed rock mass) subjected to either static or dynamic loading. The discontinuous medium is represented as an assemblage of discrete blocks. Different representations of joint material behaviour can be modelled. Blocks in UDEC can be either rigid or deformable. UDEC is able to simulate the flow of fluid through the discontinuities and voids in the model. A fully-coupled hydro-mechanical analysis can be performed in which fracture conductivity is dependent on mechanical deformation of the joint aperture; conversely, joint water pressures affect the mechanical behaviour (UDEC, 2004).

III. DAM FOUNDATION TREATMENT

Dam foundations typically require significantly more extensive investigation and design programs than do buildings, and most bridges. The consequences of the failure of a dam are usually very severe and can result in loss of life and property damage. Moreover, most dams are a vital part of the infrastructure of a community. The general requirements of the design of rock foundations for gravity dams are stability against sliding and overturning, acceptable levels of differential deformation, and control of seepage. The ability of the rock in the foundation to resist sliding failure depends on the orientation and continuity of faults, joints, and bedding planes in the foundation, the shear strength of the discontinuities, and the uplift pressures generated by the head of water in the reservoir.

The wide varieties of geologic conditions that can result in sliding failure of dams are shown in Fig. 1 (Wahlstrom, 1974). There are six different cases viz. i) brittle jointed sandstone containing beds of clay shale dipping upstream and daylighting beyond the toe of the dam, ii) horizontally bedded limestone with clay shale seams that daylight downstream of the dam , iii) fractured crystalline rock containing a fault with low strength clay infilling that dips upstream, iv) conjugate joint sets with orientations that will result in easy shear dislocation of the rock mass, v) sedimentary rocks dipping downstream intersected by a fault that daylights beyond the toe of the dam, and vi) folded sequence of sedimentary rocks containing clay shale beds are presented in the figure.

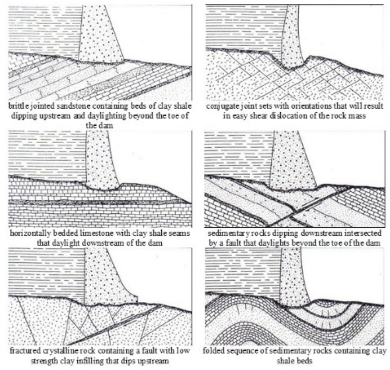


Fig. 1 Geological conditions in dam foundations (after Wahlstrom, 1974)

The one common condition in all these six cases is the presence of weak planes that daylight at the ground surface downstream of the dam. However, the presence of low strength, near-horizontal discontinuities that do not daylight downstream of the dam may result in excessive displacement of the foundation.

III.1. Stabilization techniques

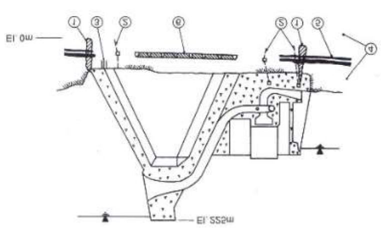
Stabilization techniques are adopted to overcome these problems of geological conditions. However, in general, stability conditions are unfavourable if the discontinuities are continuous and planar, contain a low strength or brittle infilling, and have positions and orientations that form a wedge of rock that can slide from the foundation. Some stabilization techniques used for foundation remedial works of dams to prevent sliding in the foundation are (Duncan, 1999): Concrete shear keys, Bored concrete piles, Concrete ballast, Excavation and concreting and Tensioned anchors. The examples of dams where these remedial works, in addition to drainage and grouting, have been carried out to prevent sliding in the foundation are given in the following sections.

III.1.1. Concrete shear keys

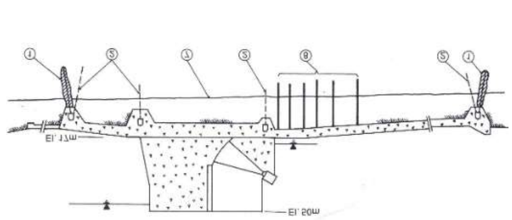
The 180 m high Itaipu dam in Brazil is a hollow concrete gravity dam founded on a series of basalt flows. Within the foundation there occurs a series of sub-horizontal flow contacts containing contact breccia. In order to prevent sliding failure on these contacts, concrete shear keys were constructed to increase both the shear strength and shear stiffness of the contacts. They were formed by excavating eight tunnels at about 16 m centres, both parallel and normal to the dam axis, and filling them with concrete. The tunnels were 2.5 m wide and 3.5 to 7 m high so as to cut through the weak layer and into sound rock above and below. A cross-section is shown in Fig. 2 (a). The total area of the shear key is 125 m by 150 m. A system of drainage tunnels surrounds the square grid of shear keys to ensure that there is no build up of water pressure (Abrahao et. al., 1983).

III.1.2. Bored concrete piles

The spillway structure of the Gezouba project in China is a 35 m high concrete gravity structure, shown in Fig. 2 (b), founded on a horizontally bedded sequence of sandstones, siltstones and claystones. These rock types are all of low strength and are highly deformable. The shear strength of the foundation was improved by installing a pattern of 20 m deep bored concrete piles downstream of the dam (Xu et. al., 1983).



(a) Concrete shear key in Itaipu Dam (Abrahao et. al., 1983)



(b) Concrete bored piles atGezouba project, China (Xu et. al., 1983)

Fig. 2 Examples of methods of preventing sliding failure of gravity dam foundations

III.1.3. Concrete ballast

The Morris Shepard Dam in Texas is a 57 m high, flat slab buttress dam founded on low permeability shale. In 1986, movement monitoring results and observations of cracks in the upstream and downstream spillway foundations showed that the structure has slid a distance of about 115 mm since construction in 1941. Remedial work consisted of installing 145 pressure relief wells because there has been no drainage in the original construction, and placement of 60,000 m^3 of concrete in the hollow core of the spillway. The combination of reduced uplift pressures and increased weight of the dam has the effect of increasing the net vertical force and the shear strength of the slip planes in the foundation (ENR, 1988).

III.1.4. Excavation and concreting

The Liu-Jia-Xia dam on the Yellow River in China is a 147 m high concrete gravity dam founded on extensively faulted and folded micaceous and hornblende schist. During construction of the foundation selective excavation and concreting was carried out in a number of fault zones to improve both the bearing capacity and shear strength of the rock mass. On the right abutment, poor rock was excavated to a depth of 25 m, and in the main fault zone a 3 m by 4 m shaft, 15 m deep was excavated and then backfilled with concrete followed be extensive grouting (Fu et. al., 1983).

III.1.5. Tensioned anchors

The Inguri Dam in Soviet Union is a 271.5 m high arch dam founded on limestone and dolomite with the beds dipping downstream at an angle of 50° to 70°. The foundation rock also contains six sets of joints and is extensively jointed. Stabilization of the foundation consisted of excavating a network of tunnels in the fault zones and backfilling these with concrete. A concrete slab was then poured on the rock surface at the toe of the dam and tensioned anchors installed to provide as additional restraining force (Mgalobelov and Lomov, 1979).

III.2. Dental treatment

The foundations of a dam must be able to withstand the loads imposed upon it by the structure without unacceptable deformation, both immediately after filling the reservoir and in the long term. With time, deterioration by saturation and percolation of water can occur, whilst soft rocks and clays usually exhibit lower residual strengths under sustained loading than under rapid testing. If it is economically feasible, all material under the base of a proposed dam which could cause excessive settlement and leakage should be removed. If this cannot be done, the dam design should be modified to take account of such material. Sometimes it may be necessary to remove material to considerable depths in isolated areas of the foundation. This is known as dental work. The general overall removal of material is termed stripping, whereas the removal of loose masses of rocks on the abutments is termed scaling.

III.2.1. USBR formulae

Frequently, relatively homogeneous rock foundations with only nominal faulting or shearing do not require the sophisticated analytical procedures described above. United States Bureau of Reclamation (USBR) during the construction of Shasta Dam undertook extensive studies for strengthening the weak rock seams in the dam foundation. Two-dimensional analyses undertaken utilizing Airy's stress function showed that beyond a certain thickness of concrete plug, the rate of decrease in deflection was exceedingly small. Based on theoretical studies undertaken, the following formulae for determining the depth of concrete plug were evolved (USBR, 2001):

$$d = 0.0066bH + 1.5 \qquad \text{for } H > 46\,\text{m}$$
$$d = 0.3b + 1.5 \qquad \text{for } H < 46\text{m}$$

where

d = Depth of plug (m)
b = Width of seam (m)
H = Height of dam above foundation level (m)
(In clay gouge seams, d should not be less than 0.1 H.)

These rules provide an estimate of how much depth of shear seam material should be excavated, but final decision must be made in the field during actual excavation (USBR, 2001).

IV. CASE HISTORIES ON DENTAL TREATMENT IN INDIA

IV.1. Bhakra dam project (after ISEG, 1982)

The 226 m high and 518 m long Bhakra dam is a concrete gravity dam constructed across Sutlej river to generate 1204 MW power having a gross storage of 9621 Mm^3. The dam is located on thick sandstone bands interbedded with bands of siltstone and claystone (ISEG, 1982). The aerial view and section of Bhakra dam are presented in Figs. 3 (a) and 3 (b) respectively.The major foundation problems encountered were i) steeply dipping shear zones and claystone bands posing problem of shear and settlement, ii) gentle downstream dipping cross-shear zones in the abutments which posed the problem of shear, settlement and seepage in the abutments, and iii) highly fractured and jointed nature of the competent members of the foundation which posed the problem of uplift pressures in the foundation.

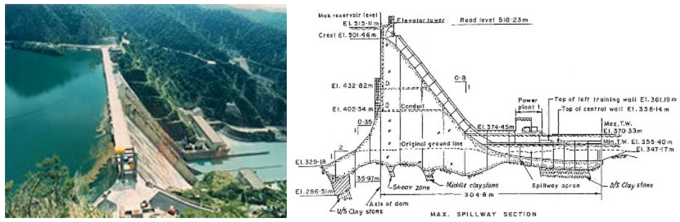


Fig. 3Bhakra Dam (CWC, 2000)

In the foundation, the load was distributed to the adjoining competent sandstone members by replacing the intervening weak zones of clay stone and sheared sandstone with concrete plug down to adequate depth. The ‘heel’ clay stone was removed to a depth of 42.6 m below the river bed by mining method and backfilled with concrete. Further a concrete strut of adequate thickness spanning the clay band was provided for transferring the load to the member exposed upstream of 'heel' clay stone, The 'heel' clay stone exposed on the valley slopes in the reservoir was provided with concrete cover to confine the member.

The axis shear zone was removed down to a depth of 1.5 to 2 times the width and backfilled with concrete. Plug tunnels of 2 m x 2 m in size and up to 15.2 m to 30.5 m depth following the cross-shear zone were provided into the hill to confine the crushed material and act as shear keys and increase the path of percolation. Such drifts were given at the axis; toe of the dam and at intermediate spacing of about 22 m apart. In addition the shear zones in between the consecutive plug tunnels were washed and grouted up to a depth of 22 m and surface trace of these were further excavated and provided with dental fill of concrete.

IV.2. MyntduLeshka hydro-electric project (after CSMRS, 2005)

The project is a run-off-river scheme, located at the Jaintia Hills District of the State of Meghalaya. The project envisages construction of the 63 m high and 322 m long Concrete Gravity Dam across the Myntdu River, just below the tri-junction of its two other tributaries, the Lamu and the Umshariang. The major geological features encountered during the excavation at the project were i) medium to coarse grained granitic rock traversed by shears and joints, and ii) presence of number of shear zones in the foundation of Blocks 7 to 9 and Block 17 and the width of the shear zones varied from 5 to 12 m (CSMRS, 2005). The typical shear seams in the foundation of the project are presented in Fig. 4.

Fig. 4 Shear seam at MyntduLeshka hydro-electric project, Meghalaya

The treatment carried out were i) for shear zone of 5 m width, reinforced beam was provided spanning over the weak zone. The width of the beam was determined using Eq. (2.3), ii) for shear zone of width greater than 5 m, FEM studies were undertaken to determine the depth/dimension of the plug. The depth of plug was kept equal to the width of the shear zone.

V. CONCLUSIONS

Development of hydro-power potential of a country invariably requires the construction of gravity dams which form the largest percentage of dams built world over. The literature review carried out on detailed project reports of various hydroelectric projects also reveals that the rock foundations are seldom homogeneous and presence of geological features such as seams, joints, adversely dipping rock beds, highly fractured and sheared ground etc. are more common. Analysing the complex behaviour of these weaknesses in the rock foundations is not possible using the conventional dam analysis procedures. Understanding the behaviour of dam-foundation system requires a systematic study. Advanced numerical modelling techniques enable engineers to model the various foundation conditions leading to a better understanding of the stress-deformation response of the dam-foundation system.

Dam-foundation interaction analyses have been carried out by various researchers using mostly the finite element analysis. Some studies are reported on the analyses considering the foundation as a jointed rock mass and explicitly modelling the rock joints using UDEC. However, all these studies are based on the analyses for a specific project having site specific material properties. Joint spacing of more than 10 m and even equal to the width of the dam base is considered in these studies whereas the actual joint spacing found in the geological investigations of various hydro-electric projects is small. Though extensive studies are carried out on jointed rock mass, the studies on the effect of joint set inclination on the stress-deformation response of a dam-foundation system is not reported so far. The interaction of the dam with the foundation specially one with difficult or troublesome geology needs greater attention and analysis.

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