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ABSTRACT

Dam-foundation interaction plays an important role in the design and analysis of concrete gravity dams as has been reviewed from the failures of dams in the literature. The objective of the study is to understand the behavior of a concrete gravity dam taking into account the effect of dam and foundation discontinuities using three-dimensional Distinct Element Code. Different models have been formulated considering the complexities involved in the modeling of the dam structure, dam-foundation system and the results have been compared. Modeling the dam with the discontinuity in the form of crack which runs through the concrete section has a complex phenomenon taking place at the interface where sliding and rotation deformation contribute to the overall stability. Modeling dam with discontinuities in the dam monoliths proved to be more realistic in the analysis of dam than considering a monolith dam with no dam discontinuities. Significant relative displacements between the monoliths and the permanent relative displacement at the end of seismic input have been observed at the discontinuities in this case. Modeling the foundation rock discontinuities represent the field conditions in the analysis more realistically. It is clearly indicated from the study that the interaction between dam monoliths and discontinuities in the foundation rock mass provide more realistic dam response.

INTRODUCTION

Dam, reservoir and the foundation on which it rests is a complex structure comprising of a rock mass which inevitably undergo changes with time. Some of these changes are slow and subtle and don't reveal their existence unless precisely and constantly monitored. Concrete dam structures are not monolithic structures rather have discontinuities inherence to construction phases, such as vertical construction joints. Thus, with the presence of these planes of weakness, even the limited tensile strength of concrete may not be attained over the major portion of the dam (Dowling, 1987). Study has been carried out taking into account the effect of joints opening and closure on arch dam models, which showed that the joints opening occur which redistributes stresses in the dam after the Northridge earthquake of 17 January 1994 (Fenves & Mojtahedi, 1995). Research in the area of dam monolith joints has also been carried out by many researchers such as Fenves e*t al.*, (1989), Ayari, (1988), Derucher *et al.*, (1987), Droz (1987), Kuo (1982) and others.

On the other side, foundation interaction not only alters the dynamic characteristics of the system due to elongation of time periods but it also alters the stress distribution due to static and seismic loads. Thus, the inclusion of its effect in the study of seismic response of any structure, particularly in dams requires a thorough understanding for implementation. The interaction effect of the foundation rock, the dam and the reservoir has a significant effect on the response of the dam as has been observed from the study of past dam failures like Shih-Kang Dam, St. Francis Dam, Malpasset dam and others (revealing the lack of understanding in dam-foundation-rock interaction). Malpasset dam was a 60m high double arch concrete dam in southern France located in a gorge where the Reyran river flows from north to south. Downstream fault at the toe of the dam was not known during the design and the construction stage which was a water tight fault. As can be seen from the Fig 1 which depicts the three dimensional failure mechanism, it is clear that the presence of the parallel discontinuities with variable zones of permeability were the major reason for the failure (Londe, 1987). As a result high uplift pressure developed which led to the wedge type failure. Failure of Malpasset dam highlights that the inadequate evaluation of foundation geological structure has caused the failure. Such gaps in the analysis of the dams can hinder the growth of dam industry.

Finite element modeling of dams taking into account thedam flexibility has become common. Elaborate models have been developed to include the interaction of the structure with the foundation, represented as an elastic or visco-elastic continuum, and the effect of hydrodynamic forces. By and large most of the methods developed for the analysis of structure-foundation interaction idealized the unbounded foundation as a uniform homogenous one.

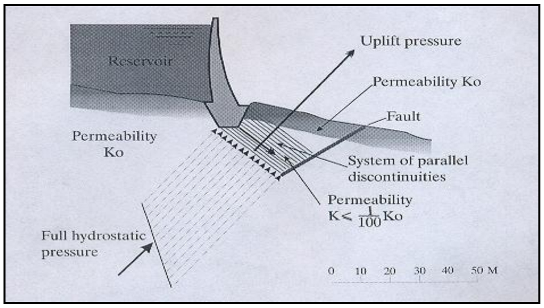

Fig 1: Three dimensional failure mechanism of Malpasset dam (Londe, 1987)

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As every dam has unique site features which are difficult to generalize for all dam locations. Strong emphasis should be placed on relevant site investigation to acquire a thorough knowledge of rock structure, type of rock, type of weakness planes, their orientation and other features relevant to analysis and design. The objective of analysis should be to extend the designers experience, judgment and intuition in a rational manner towards the successful creation of an entity. Thus, the problem of dam, foundation and abutment modeling remains one of the most critical aspects of dam analysis and design.

The analysis of such discontinuous system is compounded by the sliding and separation that may occur along the interfaces between adjacent blocks or interface in the form of crack in the concrete block. Consequently, an analysis that assumes perfect bonding at interface would predict an over or under estimation of the response of the structure. Special linear or non-linear analysis technique like DEM that takes into accounts the effect of these discontinuities on the response of the system is employed to study the behavior of the problem.

DISTINCT ELEMENT METHOD (DEM)

The system in DEM (Itasca UDEC and 3DEC Manuals, 2003) is divided into polygonal blocks by joints, where blocks are discretized into triangular constant-strain elements by the automatic mesh generator, making the block as a fully deformable one. Normal and shear springs and dashpots are set between contacts blocks for simulating the behavior of joints. Main attributes of DEM, which makes it different from other software's, are its capability of the individual blocks to undergo large displacements and rotations. Secondly, interaction forces between blocks arise from changes in their relative geometrical configuration and lastly the explicit time marching scheme used for solving equilibrium equations.
Explicit time marching scheme is applied to solve the above equilibrium equations, which makes it suitable for both dynamic and non-linear analysis with the drawback that the time step is very small for each cycle.

MODELING OF DAM-FOUNDATION SYSTEM

Analysis of any structure depends on the various aspects among which modeling plays a very important role. Before carrying out the analysis of any structure, first and foremost point to be taken into account is to model the structure as close to the actual behavior. With the objective to understand the effect of discontinuous nature of dam portion case studies have been done. In the first instance the behavior of crack in the dam portion has been analyzed to understand the effect of toppling of the top block can lead to disaster in the downstream of the dam by causing flood etc. In the second study the influence of discontinuities in the form of interface between concrete monoliths has been studied.

2-D Case Study of Dam

For the case study the cracked Koyna dam which is a 103 m high gravity dam having an unconventional cross-section, which made it more vulnerable to the Koyna earthquake of 11 December 1967. After this earthquake shaking, it suffered severe structural damage. An approximately horizontal crack at EL 66.5m occurred on the upstream and downstream side of the non-overflow monolith. Leakage from some of the crack was also found, which implied that the dam has been fully cracked. As the cracks are expected to run through the non-overflow sections, splitting the dam into two totally different sections, which implies that the stability of the top block needs to be studied very carefully. Research on the crack propagation has highlighted that the crack propagation apart from exhibiting the horizontal profile, follows the upstream and downstream slope cracking pattern. The crack profile from upstream to downstream face is difficult to ascertain. This profile influences considerably the stability of the cracked dam. Modeling of the cracked dam has been done using the discontinuum based formulation known as Universal Distinct Element Code (UDEC). Parameters of the Koyna dam considered were: Elastic Modulus E=3.1e10 Pa; mass density ρ = 2640kg/m^3; and poisons ratio ν = 0.2. The cohesion coefficient of the crack was set to zero, friction coefficient was set at 1.0. An elastic model following coulomb's friction law with zero tension in the normal direction was employed for the contacts and linear elastic model was adopted for the deformable blocks. Though there were no test values for the stiffness of the springs K_n and K_s, which represent the elastic properties of the crack, they were assumed to have an isotropic behavior, as equal to 2e9 Pa/m, approximately one-fifteenth of the value of the elastic modulus of the concrete blocks (Pekau, 2004). The nodes in the bottom face of the lower block were fixed in at level of 0.0m, and the maximum reservoir level was kept equal to 96.5m as per the report (1968).

Loads applied include self-weight of the dam, hydrostatic loads, hydrodynamic loads, uplift and earthquake forces. For the analysis, the section was analyzed assuming the plane stress behavior. The recorded Koyna ground accelerations on December 11, 1967 were used as earthquake input whose peak value is 0.60g in the longitudinal direction, 0.49g in the stream direction and 0.34g in the vertical direction w.r.t. the dam respectively. Time step $\Delta t = 1.7*10^{-4}$ was assumed in the computation.

Damping is a very important parameter for the study of the top profile of the fractured dam especially during earthquakes. The dashpots for the spring connecting the separate blocks will dissipate energy when there is impact between them. Thus, damping of the system was divided into three zones as suggested by Pekau (2004) and selected according to the behavior of the corresponding parts as depicted in Fig 2

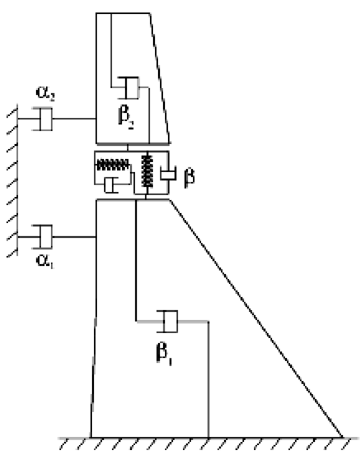

Fig 2: Damping in the Koyna Dam

Analysis The response of dam depends on many factors like crack pattern, properties of the material, environmental and geological conditions, damping coefficients, etc. Before the analysis of any structure is carried out, first and foremost point is to simulate the actual conditions that are prevailing at the site. The system was thus analyzed under static conditions until the equilibrium condition was reached. To check the validity of the model in UDEC, the effect of the different reservoir levels was studied which complied.

At zero water level, max displacement of the top block was 0.119m to the left side i.e. in the negative x-direction at the end of the earthquake (11s). When the water level was raised up to the crack level i.e. at 66.5m, displacement was 0.047m, it has decreased, as the x-direction force including hydrostatic loads, hydrodynamic loads has decreased which resists its movement in negative direction. At water level upto 81.5m, displacement was decreased to a lesser extent to 0.007m at the end of earthquake because of uplift pressure, which was trying to move it in positive x-direction. At water level upto 96.5 m i.e. maximum reservoirs level, it had maximum displacement of the order 1.578m. Thus, it is concluded that with increase in the reservoir loads, in addition to the oscillatory motion caused by earthquake, water loads including hydrodynamic and uplift causes the fracture top profile to move and rotate. It was observed that there was more of sliding than the rotation with transverse and vertical motion of earthquake applied simultaneously.

Effect of Increase in Shear Stiffness at the Contact Joint properties such as normal and shear stiffness, joint friction angle, cohesion, dilation angle etc. are conventionally derived from laboratory or field-testing. Taking shear stiffness among them as the important factor affecting at concrete to concrete interface, the literature gives a wide range from 10 to 100 MPa/m, for joints with soft clay in-filling to over 100GPa/m for tight joints in granite and basalt. Large data has been published by Kulhawy (1975) and Rosso (1076) and Bandis *et al.* (1983). In case of crack in dams, with penetration of water other particles can also enter which may reduce or increase the shear resistance, and on the other hand cracked surface can be a smooth or rough. Keeping in view all the possibilities it is very difficult to have an idea of what is the best value at the crack at different situation. Therefore, to study this effect, variation of shear stiffness value was carried out under transverse and vertical ground motion, keeping the water level to the maximum level. With increase in K_s value from 2e9 to 1e10 the sliding at the end of earthquake decreased from 1.7m to 1.095m whereas if the value of K_s is decreased from 1e9 to 1e7 Pa, the sliding increases from 1.7m to 15.308m, where sliding is accompanied by rotation of the top profile at 6sec, as shown in Fig 3. It is observed that with the decrease in value of K_s, instability of the top block takes place.

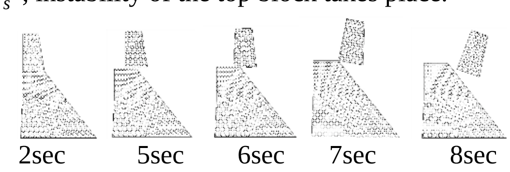

Fig 3: Collapse Mechanism with low K_s

Crack Profile Crack propagation in a concrete mass is a very complex phenomenon, which cannot be determined easily. As has been discussed before, that angle of crack propagation is a very difficult question; therefore, there is a need to study the critical crack profile at different angles from upstream to downstream direction. To study this, five crack profiles (i) horizontal crack- Case A (ii) upstream (Case B & C) and downstream (Case D & E) crack profiles at crack angle 15^o and 7.5^o was taken for the analysis discretizing the dam portion as the triangular constant-strain elements for the stress-strain analysis in each block.

As was seen, the top block behaves differently in all the five cases. In the case where crack is in the downstream directions, sliding is accompanied by rotation and ultimately falling of the top block, as motion is added up by the hydrostatic, hydrodynamic load and uplift at the crack level, where as in case where crack is in upstream direction, there is sliding, no toppling of the top block has been observed to the left, because motion is stabilized by hydrostatic and hydrodynamic loads, which were producing unfavorable condition in the previous cases.

Figure 4 depicts the motion of the top block in terms of sliding, and Fig. 5 depicts the motion in terms of rotation of the top block, where it can be seen that in case of horizontal crack and upstream crack, there is opening and closing of the crack, where as with downstream crack, with inclination 15^o, it has started rotating very early as compared to crack with angle 7.5^o, explaining the unstable crack profile where there is no readjustment as the motion is enhanced as shown in Fig 4. Large values of permanent displacement of 1.4 m are noticed in Cases A and B, while the corresponding values are small, not more than 0.4m at the end of Koyna earthquake input in Cases C, D and E. Rotation oscillation is very severe in A and B cases with downstream crack.

Maximum rotation in Cases C, D and E has reached a value of 0.002145 rad, 0.000260 rad and 0.000267 rad respectively where as in Case B rotation is 0.036700 rad at the end of record. In Case D, there is severe rotation of the top block which is shown in Fig. 5, where sliding has been accompanied by rotation which rolls down the downstream face of the dam.

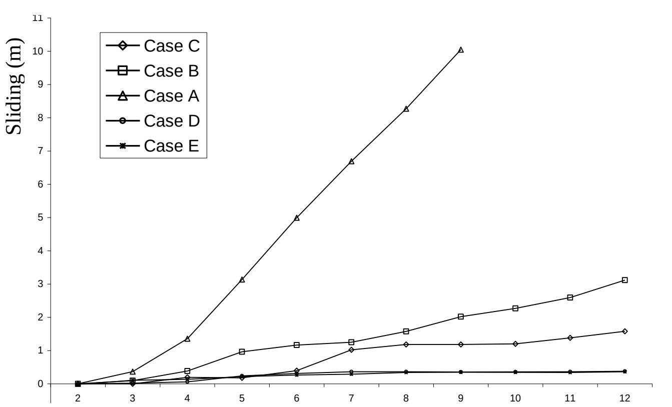

Fig. 4 Sliding Response under Koyna EQ

From all the cases discussed above, it is concluded that the crack profile with an inclination in the downstream direction is more vulnerable than those in the upstream direction. In the Koyna earthquake, crack was observed at the neck and therefore in the present study same the effect of crack profile was also studied. Both sliding and rotation deformation contribute to the overall stability. But the cracked dam is stable in case of crack in the horizontal direction to a large extent.

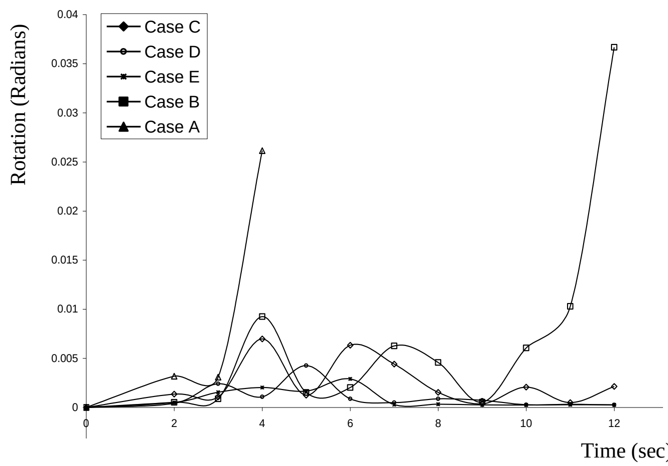

Fig. 5 Rotation under Koyna EQ

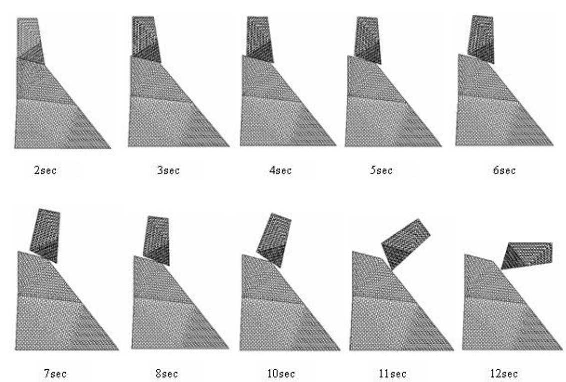

Fig. 6 Progress of Dam Failure in Case D (downstream crack with 15^o inclination)

3 D Case Study of Dam

With the objective of understanding the effect of dam discontinuities a case study of upcoming dam in the lesser Himalayas using the three-dimensional Distinct Element Code (3DEC). The dam site consists of Phyllites of the Chandpur Formation of the Upper Proterozoic age (Pt3) and belongs to Jaunsar Group of the Krol Super Group which has been differentiated as Phyllites quartzite massive (PQM) and Phyllites quartzite thinly bedded (PQT). The modeling of the dam monoliths with monolith discontinuities has been done by creating an interface at the junction of two adjacent monoliths. During the modeling of the foundation lithological bands, L and D- shear zones have been considered for the study. The D and L- shear zones along with lithological bands form wedges on the left abutment which are of major concern from structural stability of the dam structure. After understanding the complexities involved in the modeling of the dam concrete portion and foundation rock mass, different models on 3DEC software have been considered.

The study starts from the simple model where dam monoliths consisting of no dam joints i.e. monolithic construction in the dam portion has been considered and foundation with no discontinuities. Then model comprising of dam with discontinuities in the form of joints separating the monoliths and foundation with no discontinuities as shown in Fig 7 and Fig 8.

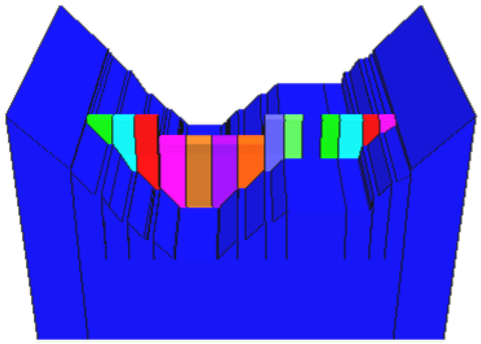

Fig 7: Model with the interaction between the dam monoliths

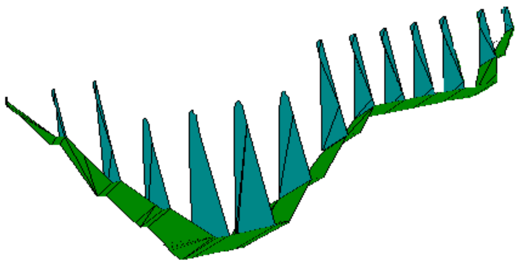

Fig 8: Model with joint pattern

Material Properties and Loads Joints in dam and foundation comprise of different material strength properties which have been assigned the values as obtained from field studies. Non-linear constitutive model (Mohr- Coulomb slip model) has been applied to joints in the dam portion as well as in foundation rock joints. Analysis of dam has been carried out for static and dynamic load conditions. In the present study, insitu stresses, gravity load, hydrostatic loads and uplift pressure are considered for static analysis. Under seismic load, hydrodynamic loads are worked out as per the Westergaard's theory (1933). The Uttarkashi earthquake record of 1991 has been used as dynamic input (in all the three directions) as the dam site is situated in similar geological setup. The viscous boundaries proposed by Lysmer and Kuhlemeyer (1969) consist of independent dashpots attached to the boundary in the normal and shear directions have been applied in the rock foundation in all the directions. Earthquake motion has been applied at the applied at the base of the foundation as stress wave using the formula

$$\sigma_n = 2\left(\rho C_p\right) v_n \quad (1)$$

$$\sigma_s = 2\left(\rho C_s\right) v_s \quad (2)$$

where σ_n = applied normal stress; σ_s = applied shear stress; ρ = mass density; C_p = speed of p-wave propagation through medium; C_s = speed of s-wave propagation through medium; v_n = input normal particle velocity; and v_s = input shear particle velocity. The formulae assume plane-wave conditions. The factor of two in eqs. (1) and (2) accounts for the fact that the applied stress must be doubled to overcome the effect of the viscous boundary.

Before the seismic analysis, stability equilibrium under static loading i.e. under gravity load, hydrostatic load and uplift pressure has been obtained. Displacements at the crest of the monoliths have been monitored during the seismic load application in cases with no foundation discontinuities. Comparison of the total crest displacement has been made in Table 1. Maximum crest displacement in model with no dam discontinuity is 13.6 mm (Fig. 9) against 127 mm in model with dam discontinuities (Fig. 9). This is because of additional movement at the interface of the monoliths in comparison with the other model as there is less flexibility in the dam portion. Therefore, the first model does not represent the actual behavior, leading to misinterpretation of results i.e. the behavior of dam to seismic input. On the other hand, in the first model, it can be seen that permanent displacement at the end of the seismic input reaches a zero value and in case of model with dam monolith interfaces, there is permanent displacement at the end which is taken as permanent relative displacement between the two monoliths of the order 4.83 mm. However, these values will very much depend on the input earthquake motion, frequency of vibration of the dam and the interface properties at the interface It is observed that analyzing a dam as a monolithic construction may not lead to actual behavior of the dam. Henceforth, in the further study dam discontinuities have been considered.

Table 1: Comparison of transverse displacement between two adjacent monoliths

Model	Maximum Crest Displacement (mm) (Transverse Direction)	Permanent Displacement(mm)
With no dam and foundation discontinuities	13.60	0.00
With dam discontinuities and no foundation discontinuities	127.00	4.83

From the above discussion on the effect of dam discontinuities on the response during the seismic load clearly indicates that the dam analysis with no discontinuity in the dam or monolithic structure does not provide a realistic response.

Reason for this is that when dam monoliths interaction is considered, they periodically open and close with the excitation of the external seismic force.

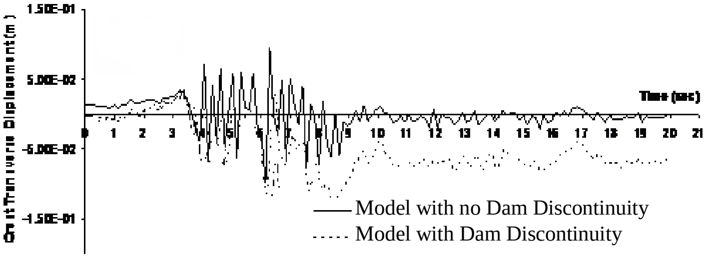

Fig 9: Transverse crest displacement observed during seismic input

Effect of Foundation Discontinuities

It is observed that the dam discontinuities play an important role and therefore these have been considered in further study. Foundation on which dam rests, has major influence on the behavior of the dam. As has been learnt from past dam failures about the role of foundation mass, it is important to study the response of dam resting on foundation rock mass comprising of discontinuities. Foundation geological conditions clearly demarcate the discontinuities in the foundation region under study. For understanding the effect of foundation discontinuities, model with dam discontinuities and foundation discontinuities has been taken into account as shown in Fig10 & Fig 11.

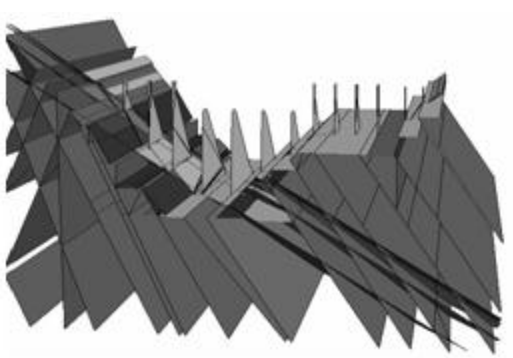

Fig 10: Jointing pattern with dam and foundation discontinuities

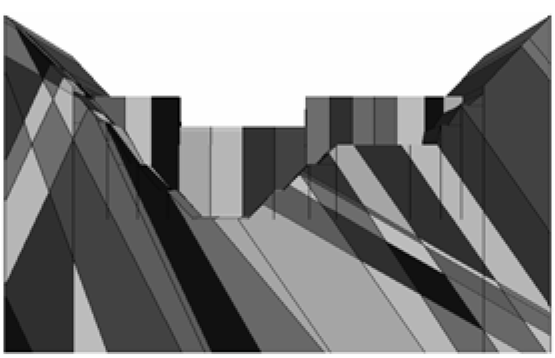

Fig 11: Model with dam and foundation discontinuities

The boundary and loading conditions remaining the same, seismic analysis has been carried out. First, it is important to look at the overall behavior of displacement pattern, velocity pattern, and stresses. Relative transverse displacement at the interface of monolith between overflow and non-overflow section shows an important implication, that with the inclusion of discontinuities, permanent displacement at the end of the seismic input increases (Fig 12). The increase is from 4.83 mm in case of model with no foundation discontinuity, which increases to 23.10 mm with the inclusion of foundation discontinuities. Second important implication is the increase in relative transverse displacement from 57.20 mm to 76.70 mm. It is observed that after including foundation discontinuities relative transverse displacement and permanent displacement between dam monoliths increases by 25% and 79% respectively. The difference in the amount of displacement, in case of models with varying complexities of foundation discontinuities are nearly the same as given in Table 2.

Table 2: Comparison between different models

	Model with no foundation discontinuities	Model with Foundation discountinuities
Max. Rel Trans Disp (mm)	57.20	76.70
Permanent Trans Disp (mm)	4.83	23.10
Max. Rel Vert Disp (mm)	5.02	10.80
Permanent Vert Disp (mm)	0.31	4.11

But clearly noticing at the cross section on the left abutment, large numbers of discontinuities with four major ones intersecting with lithological features form wedges at various points in model. As a result model with large number of discontinuities forming small blocks, results in wedge failure in the left abutment which was not observed in previous model.

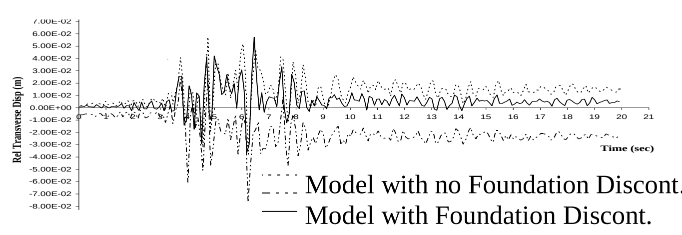

Fig 12: Comparison of models with foundation discontinuities (Transverse Displacement.)

This failure has induced large amount of shear displacement at the concrete–concrete interface and at dam abutment interface. This failure if occurs in reality will pose a huge problem. Such type of failure has been noticed in the field on the left abutment very close to that observed in Vajoint Slide

Catastrophe. This highlights the need of more detailed study on slope stability aspects in the vicinity of the dam needing proper treatment. Therefore, it can be concluded that model with dam and foundation discontinuities represents the actual nature of the dam-foundation when the lithological and major shear zones have been incorporated.

After looking into the behavior of the foundation discontinuities especially at the abutment, it is important to study, how the abutment can be stabilized so that such failure does not take place. Shotcreting is normally adopted in the field to stabilize the slope. It is very difficult to access the strength which will be achieved.

CONCLUSION

Influence of discontinuities which can either be in the form of crack, joints which can be concrete-concrete, concrete-rock, and rock-rock interface has been undertaken. From the 2D-DEM modeling of the post crack behavior of the cracked Koyna dam it has been found that interface properties (Shear Stiffness K_s, and damping at crack surface) are sensitive to the behavior of cracked dam. A parametric study of the crack profile at the neck of the Koyna dam i.e. KRL 2060 (as observed during Koyna earthquake) showed that the crack profile sloping u/s to d/s is critical from stability considerations of the top block above the crack. Crack profile sloping upstream has not been found to be critical in case of reservoir level above the crack.

The 3D study concludes that the effect of dam discontinuities on the response during the seismic load clearly indicates that the dam analysis with no discontinuity in the dam or monolithic structure does not provide a realistic response as the dam monoliths periodically open and close with the excitation of the external seismic force.

Analysis of dam with rock interaction i.e. inclusion of rock discontinuities or condition prevailing in the field should be considered for the analysis which represent actual events taking place in the field, as has been indicated by wedge failure on the left abutment in the present study. It is observed that after including foundation discontinuities relative transverse displacement and permanent displacement between dam monoliths increase by 25% and 80% respectively. It is observed that most of the permanent displacement in the dam occurs at the end of earthquake motion due to the plastic movement at the dam and foundation discontinuities. It is clearly indicated from the study that the interaction between dam monoliths and discontinuities in the foundation rock mass should be a necessary part of the gravity dam analysis.

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