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Experimental investigation on nonlinear dynamic response of concrete gravity dam-reservoir system

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ABSTRACT

The nonlinear response of concrete gravity dam-reservoir system has been investigated by conducting experiments on small scale model of Koyna dam on a horizontal shake table using sinusoidal chirp motions. Two dimensional models of non-overflow section of Koyna dam have been prepared in the laboratory with a scale ratio of 1:150. To satisfy the laws of similitude, an appropriate ratio of cement, sand, bentonite and water has been mixed to find the target properties of model dam. Dam models have been casted over a wooden base plate using a wooden mould. After setting and drying process the wooden mould has been removed and each dam model has been assembled with a reservoir model after placing them over a horizontal shake table. The interaction between dam and reservoir model has been made in such a manner, so that it can transmit the hydro dynamic force of reservoir water to the dam model without allowing any seepage of water across it. Experiments have been performed on the dam-reservoir system with empty and full reservoir water by applying horizontal sinusoidal chirp excitations to the shake table to observe the basic behaviour, crack formation, crack opening, sliding along crack planes and stability after crack formation of the dam model. The numerical analysis of the system has been carried out using ABAQUS 6.10 considering damage plasticity model. The effect of foundation has been neglected assuming a rigid dam foundation as the dam is placed on a rigid plate. The eigen frequencies of the dam model have been calculated and compared with experimental values to calibrate the analysis model. The crack propagation due to tensile damages is computed and the results are compared with the experimental results. The outcome of the response results shows the correctness of the developed experimental model of dam-reservoir system.

1. Introduction

A gravity dam is a solid structure, made of concrete or masonry, constructed across a river to create a reservoir on its upstream. The section of the gravity dam is approximately triangular in shape, with its apex at its top and maximum width at bottom. The section is so proportioned to resist the various forces acting on it by its own weight [1]. Concrete gravity dams serve electricity generation, water supply, flood control, irrigation, recreation, and other purposes. They are an integral component of the society's infrastructure system. Concerns about their safety in a seismic environment have been growing over the past few decades, partly, because earthquakes may impair their proper functioning and trigger catastrophic failure causing property damage and loss of life [2]. Though gravity dams can survive moderate earthquake motion they present difficult problem if they are built in seismically active areas as little is known about the response of the dam to severe levels of excitation. The damage caused mainly by the cracking of the concrete with subsequent opening and closing of cracks and sliding along crack planes, additional damage can be caused by high compressive stresses resulting from impacts during crack closure and from small contact zones during maximum crack opening [3]. It should be mentioned that the occurrence of cracks does not imply complete failure which is proved by the survival of the 103 m high Koyna dam during an earthquake (1967) with a magnitude of 6.5 Richter scale and with the peak ground acceleration in the stream direction of 0.49 g [1,31]. The duration of the strong shaking lasted about 4 s and the water level stood 11.278 m below the crest. After the earthquake, a major crack was noted at a level of 36.576 m below the crest, which coincides with the level of slope change on the downstream face [4–6]. The other concrete dams known to have suffered cracking as the results of earthquake is Hsinfengking Dam (China,1962), a 104.851 m high buttress dam and the Shih-Gang Dam which was also severely damaged by fault movements and ground shaking during the 1999 Chi-Chi earthquake in Taiwan [7].

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The issues of seismic safety of dams have been looked at with increased attention in various parts of the world in recent years. It has become a major factor in the planning and designing of new dams, proposed to be built in seismic regions and for safety evaluation of existing dams in these regions. To prevent the failure of a dam, it is important to assess its behaviour at any age during its lifetime [8,9], so that remedial measures can be undertaken to strengthen or perhaps decommission the dam at the right time. For the design of an earthquake resistant dam and the evaluation of the safety of an existing dam, it is important to use a rational and reliable dynamic analysis procedure. The analysis procedure should be capable of evaluating the deformations and stresses in a dam subjected to a given ground motion. Various numerical techniques have been used by number of authors to investigate the evaluation of damage in dam. However, smear crack model and discrete crack model have been used by many researchers to study the damage propagation in concrete dam such as Lotfi and Espandar [29]. Ghrib and Tinawi [10] investigated the damages in Koyna dam with an initial damage using pseudo-dynamic earthquake forces. The continuum damage model and co-axial rotating crack model (CRCM), which includes the strain softening behaviour of concrete, has been carried out by Calayir and Karaton [11] to investigate the seismic fracture response of gravity dam. Guangluna et al. [12] proposed nonlinear crack band theory for the seismic fracture behaviour of concrete gravity dams. Khaji and Ahmadi [13] used distinct element–boundary element approach for seismic analysis of cracked gravity dam-reservoir systems. Mirzabozorg and Ghaemian [14] examined the crack profile in the dam using smeared crack approach. Ftima and Leger [15] investigated the stability of cracked concrete dams under seismic excitation by means of rigid block model. A continuum damage model based on continuum damage mechanics was developed by Silva and Castro [16] to represent the nonlinear behaviour of the quasi-brittle material. Mansouri et al. [17] introduced the Bazant's non-linear fracture mechanics criteria for the seismic analysis of the concrete gravity dam. The crack propagation of concrete dams with initial cracks was studied by Zhang et al. [30] during strong earthquakes using extended finite element method.

A number of attempts have been made to mathematically model the nonlinear response of gravity dam during severe earthquake in order to determine whether a gravity dam can remain stable and retain the impounded water [18,19]. But the large amount of variation in the computed behaviour of the dam implies that nonlinear earthquake analysis of concrete gravity dam is not straightforward and is highly uncertain. This uncertainties and the absence of proper field data motivate laboratory experiments on small scale models [20,21]. Many experiments [22–24] have been carried out on small scale models of dam to investigate the dynamic response of concrete gravity dam during seismic excitation. However, the experiments considering the hydrodynamic effect of reservoir water are very rare.

The present study concentrates on evaluation dynamic response of concrete gravity dam-reservoir system by experimental and numerical analysis. Dynamic analysis of a 1/150 scale model of Koyna dam along with reservoir has been carried out in a shake table with high frequency capability. Sinusoidal loadings have been applied to the shake table and the responses of the dam reservoir system have been observed. The small scale modelling of concrete gravity dam incorporates additional demands on properties of model materials. Therefore, a similitude analysis has been carried out and material properties of the model dam have been derived using the similitude equations. A mix of cement, sand, bentonite and water has been used in the mix design of dam model. Several samples have been prepared and tested by trial and error method to find out the exact mix proportion required to arrive at the target material properties used to prepare the dam model. Dam models are casted using the selected mix and pouring it in a wooden mould. At first the casting, curing and drying process are carried out with the horizontally aligned mould. However, in this process, shrinkage cracks develops at the bottom and upper side of the model. To overcome this, the wooden mould is vertically placed and fixed on a wooden base plate using wooden channel nuts and bolts. Materials are poured through top opening and compacted accordingly to ensure no voids remains inside the model. After the setting and drying period, the dam models are de-moulded and each of the dam models along with the wooden base plate is placed on the shaking table where dam models are assembled with a reservoir model which is prepared mainly using perspex sheet. The side of the reservoir which is facing towards dam model is kept open to transfer the hydrodynamic effect of reservoir water to dam model. But the prepared dam model cannot sustain water flow as it starts getting dissolved when come in contact with water. Therefore, to protect the dam model from reservoir water a very thin polyethylene membrane is used at the open side of the reservoir-dam interface. Thus, it prevents reservoir water from seeping through it but transmits the hydrodynamic force to the dam model. Experiments are carried out using horizontal sine chirp motions to determine the natural frequencies of the system. The crack initiation, propagation and response of the structure during crack formation have been observed. A numerical modelling of dam with hydrodynamic effect of reservoir water has been done using ABAQUS 6.10 [25]. The hydrodynamic effect of reservoir has been modelled using Westergaard added mass technique [26]. The stresses at various points and path of propagation of tensile damages are computed and the numerical results are compared with the experimental results.

2. Theoretical formulation

2.1. Modelling of damage plasticity

The uniaxial tensile and compressive response of the material is characterised by damaged plasticity. The stress–strain response under uniaxial tension follows a linear elastic relationship until the value of the failure stress. Beyond the failure stress the formation of micro-cracks is represented macroscopically with a softening stress–strain response, which induces strain localization. Under uniaxial compression the response is linear until the value of initial yield. In the plastic regime the response is typically characterised by stress hardening followed by strain softening beyond the ultimate stress [25]. The uniaxial stress–strain curves can be converted into stress vs. plastic-strain curves by the following equations:

$$\sigma_t = \sigma_t(\tilde{\varepsilon}_t^{pl}, \dot{\tilde{\varepsilon}}_t^{pl}, \theta, f_i) \tag{1}$$

$$\sigma_c = \sigma_c(\tilde{\varepsilon}_c^{pl}, \dot{\tilde{\varepsilon}}_c^{pl}, \theta, f_i) \tag{2}$$

where $\tilde{\varepsilon}_t^{pl}$, $\tilde{\varepsilon}_c^{pl}$ represent the equivalent plastic strains and, $\dot{\tilde{\varepsilon}}_t^{pl}$, $\dot{\tilde{\varepsilon}}_c^{pl}$ are the plastic strain rates. θ is the temperature and f_i refers to other predefined field variables. Subscripts, t and c correspond to tension and compression, respectively.

The degradation of the elastic stiffness of the material when the specimen is unloaded from any point on the strain softening branch of the stress–strain curves, is characterised by two damage variables, d_t and d_c, which are assumed to be functions of the plastic strains, temperature, and field variables and represented as:

$$d_t = d_t(\tilde{\varepsilon}_t^{pl}, \theta, f_i) \quad 0 < d_t < 1 \tag{3}$$

$$d_c = d_c(\tilde{\varepsilon}_c^{pl}, \theta, f_i) \quad 0 < d_t < 1 \tag{4}$$

Therefore, the stress–strain relations under uniaxial tension and compression loading are defined as:

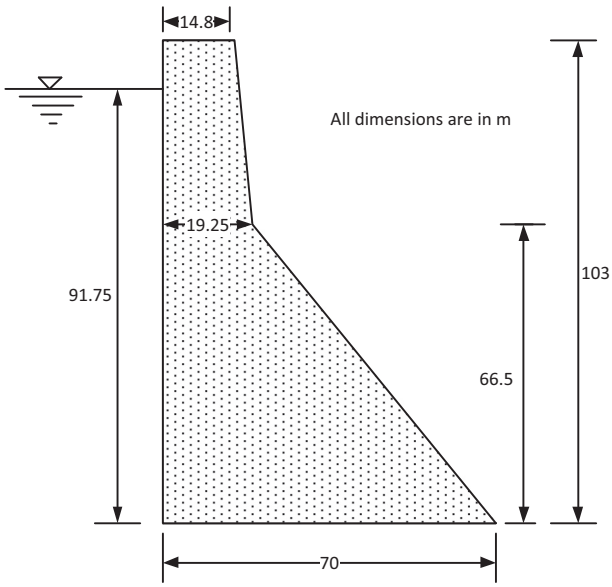


Fig. 1. Cross section of non-overflow monolith of Koyna dam.

$$\sigma_t = (1 - d_t)E_0(\varepsilon_t - \tilde{\varepsilon}_t^{pl}) \tag{5}$$

$$\sigma_c = (1 - d_c)E_0(\varepsilon_c - \tilde{\varepsilon}_c^{pl}) \tag{6}$$

where E_0 is the initial undamaged elastic stiffness of the material.

2.2. Modelling of hydrodynamic effect of reservoir water

The dynamic response of dam is greatly influenced by the hydrodynamic effect of the adjacent reservoir water. The hydrodynamic forces due to adjacent reservoir is modelled using the Westergaard added mass technique [26]. It is assumed that the hydrodynamic pressures that the water exerts on the dam during an earthquake are the same as if a certain body of water moves back and forth with the dam while the remainder of the reservoir is left inactive. The added mass per unit area of the upstream wall is given in approximate form by the expression

$$\frac{7}{8}\rho_\omega\sqrt{h_\omega(h_\omega - y)}, \quad \text{with} \quad y \leqslant h_\omega \tag{7}$$

where h_ω is the height of water in m and $\rho_\omega = 1000\ \text{kg/m}^3$, is the density of water.

3. Model selection

In the present study, Koyna dam of Maharashtra, India is considered for the seismic analysis. The dam is a concrete gravity type with maximum height of 103 m above the deepest foundation level. The water level in the reservoir at the time of earthquake is 91.75 m. The Koyna dam consists of non-overflow section and overflow section. During the 1967 Koyna earthquake, only non-overflow section geo damaged and overflow section did not show any detrimental effects.

Generally dam is analysed as two-dimensional structure. At small vibration amplitudes a concrete gravity dam will behave as a solid even though the construction joints between the monoliths may slip. However, during large amplitude motion, the behaviour of a dam depends on the extent to which the inertia forces can be transmitted across the joints. For dams with straight joints, the inertia forces that develop during large-amplitude motion are much greater than the shear forces that the joints transmit. Consequently, the joints would slip and the monoliths vibrate independently, as evidenced by the spelled concrete and water leakage at the joints of Koyna dam during the Koyna earthquake. Hence only non-overflow section has been taken for the present investigation. A cross section of non-overflow monolith is shown in Fig. 1 along its dimensions.

4. Material development

Materials for small-scaled models of concrete gravity dams should be chosen very carefully to get the nonlinear dynamic response of the model. In order to get the behaviour of a small-scaled model to accurately represent the corresponding behaviour of its prototype, the model must follow certain laws of similitude. These laws, which are determined by dimensional analysis of the problem under investigation, are relationships among the dimensionless ratios formed by corresponding parameters of the prototype and model structures. They establish requirements for the materials used to construct the model and the loading used to excite it.

Physical model can be classified as either linear or nonlinear. A linear model simulates the behaviour of its prototype structure in

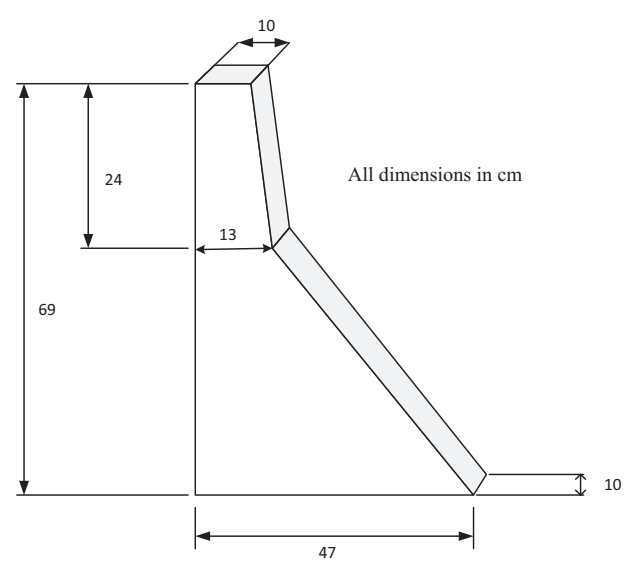


Fig. 2. Geometry and dimension of model dam (scale: 1:150).

Table 1
Summary of the mechanical properties for the prototype and the model.

Properties	Prototype value	Scale factor	Target values for model	Actual values used in laboratory
E	31,027 MPa	150	206.85 MPa	203.68 MPa
ρ	2643 kg/m^3	1	2643 kg/m^3	2578 kg/m^3
σ_u^c	27.11 MPa	150	0.180733 MPa	0.19023 MPa
σ_u^t	2.9 MPa	150	0.019333 MPa	0.01869 MPa
L	103 m	150	0.68667 m	0.687 m

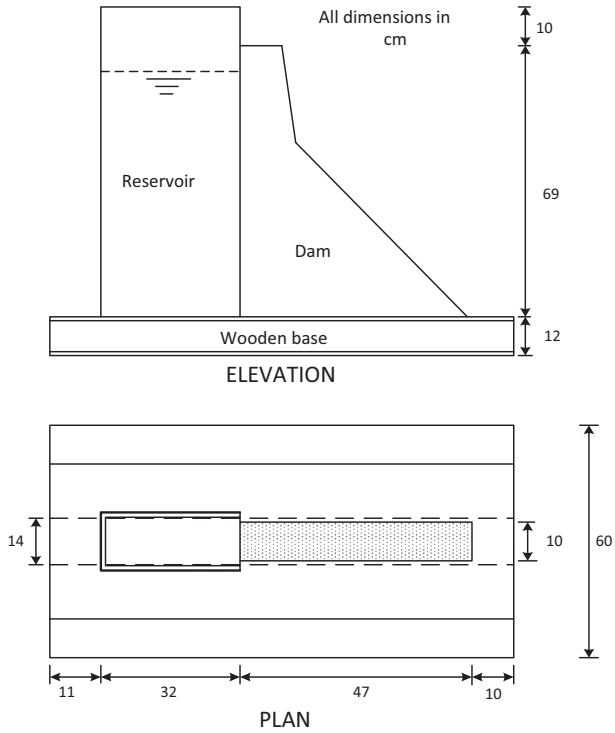


Fig. 3. Plan and elevation of the whole system.

the linearly elastic response range only. The compressive and tensile strengths and associated failure strains do not have to be scaled. The nonlinear model should simulate the response of its prototype through failure. As a result, the stress strain relations of the model material must be properly scaled from those of the prototype material. In order to model the failure of a concrete gravity dam subjected to earthquake excitation, neglecting the foundation interaction but including the reservoir fluid, the following system parameters are considered [3]: length L, time T, the ground acceleration A, the stress–strain relations of the dam material E_d, the tensile strength σ_u^t and compressive strength σ_u^c, the tensile and compressive failure strains ε_u^t and ε_u^c, the bulk modulus of reservoir fluid E_f, the mass densities of the dam and the reservoir fluid ρ_d and ρ_f, the vapour pressure of the reservoir fluid P_f the gravitational acceleration g, and the atmospheric pressure P. The similitude relation involving the above parameters can be written as [3]:

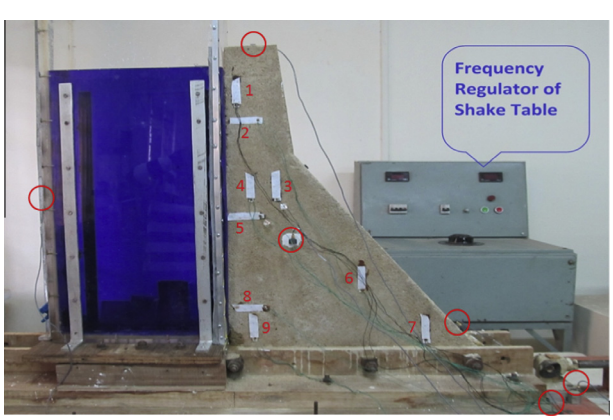


Fig. 4. Experimental setup with accelerometers and strain gauges.

$$\frac{S_r T_r^2}{\rho_r L_r^2} = 1 \tag{8}$$

$$\frac{A_r T_r^2}{L_r} = 1 \tag{9}$$

$$A_r = G_r \tag{10}$$

$$\varepsilon = 1 \tag{11}$$

where

$$S_r = \frac{P_p}{P_m} = \frac{E_{dp}}{E_{dm}} = \frac{E_{fp}}{E_{fm}} = \frac{\sigma_{up}^t}{\sigma_{um}^t} = \frac{\sigma_{up}^c}{\sigma_{um}^c} = \frac{P_{fp}}{P_{fm}}$$

$$\rho_r = \frac{\rho_{dp}}{\rho_{dm}} = \frac{\rho_{fp}}{\rho_{fm}}$$

$$L_r = \frac{L_p}{L_m}; \quad A_r = \frac{A_p}{A_m}; \quad T_r = \frac{T_p}{T_m}; \quad G_r = \frac{g_p}{g_m} \quad \text{and} \quad \varepsilon_r = \frac{\varepsilon_{up}^t}{\varepsilon_{um}^t} = \frac{\varepsilon_{up}^c}{\varepsilon_{um}^c}$$

Here, p denotes the prototype and m denotes the model. Response parameters scale are as follows: L_r for dam displacement, A_r for dam acceleration, and S_r for dam stress and fluid pressure. In the experiment carried out here, $G_r = 1$ and Eq. (10) becomes

$$A_r = 1(12) \tag{12}$$

And Eq. (9) becomes

$$T_r = \sqrt{L_r} \tag{13}$$

From Eq. (8) and (13),

$$S_r = \rho_r L_r \tag{14}$$

The use of water as the reservoir fluid also establishes the density scale $\rho_r = 1$, which requires that the material used to construct the model dam have a density equivalent to that of concrete. The selection of length scale L_r is typically based on the size of the prototype structure and the capabilities of the testing facility. The length scale for the present experiments is 150 which when substituted into Eq. (14) with $\rho_r = 1$, results in a value of 150 for S_r. From the relations of S_r we can see the model dam material should have a modulus of elasticity, compressive and tensile strength which are a factor of 150 less than the prototype concrete. Eqs. (11) and (14) state that the compressive and tensile failure strains of the prototype and model material must be equal. Table 1 lists typical values for properties of mass concrete and target values for the model material. Mechanical properties of the prototype such as Young's modulus, mass density, length are drawn from those reported in the study by Chopra and Chakrabarti [27] and values of compressive strength σ_u^c and tensile strength σ_u^t of the prototype are drawn from the study by Lee and Fenves [28].

To get the desired properties of the model dam, a mix of cement, sand, bentonite and water has been used. Several trial mixes have been done in laboratory to satisfy the similitude requirements. Tests have been carried out on the cylindrical

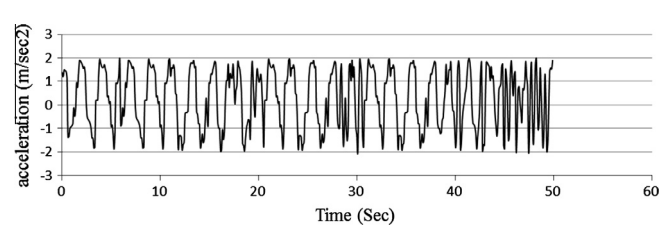


Fig. 5. Reading of accelerometer placed at the base of the shake table.

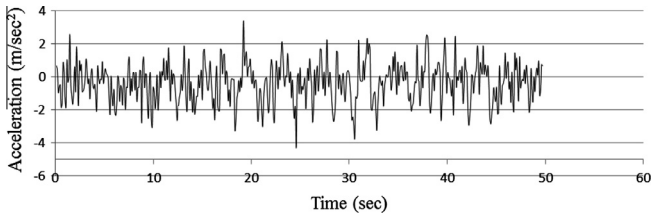


Fig. 6. Reading of accelerometer placed at top of the dam (empty reservoir).

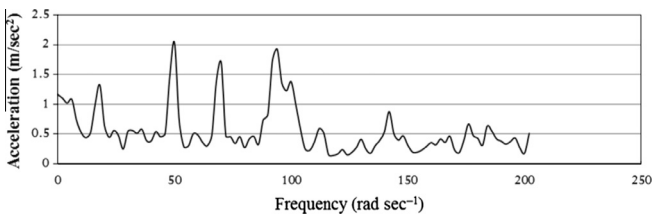


Fig. 7. Frequency response function of empty dam.

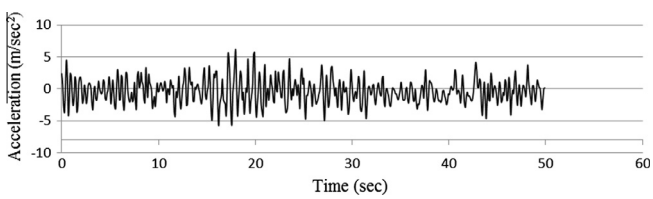


Fig. 8. Reading of accelerometer placed at the top of dam with full reservoir water.

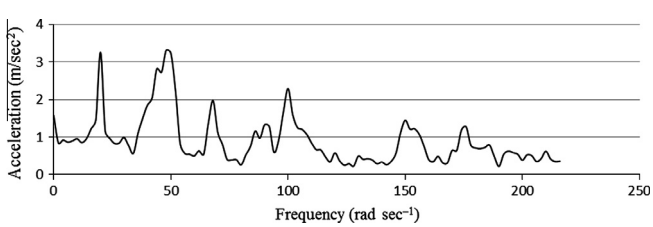


Fig. 9. Frequency response function of dam with full reservoir water.

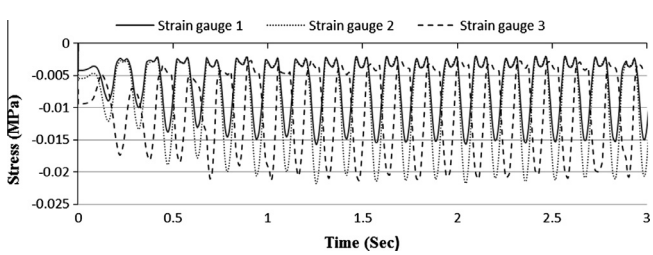


Fig. 10. Variation of stress with time of first three strain gauges for full reservoir condition.

specimen to get the failure strength and stress–strain curve of the mixes. After performing several tests, finally a mix proportion of cement: sand: bentonite: water = 1:42:0.4:7 has been selected for model dam material. The actual values of material properties and length of dam model which have been used in the laboratory for experiments are tabulated in Table 1. The Rayleigh's proportional damping has been used for the dynamic analysis. The damping properties of the model are calculated directly from the mass and stiffness which are derived from the similitude analysis.

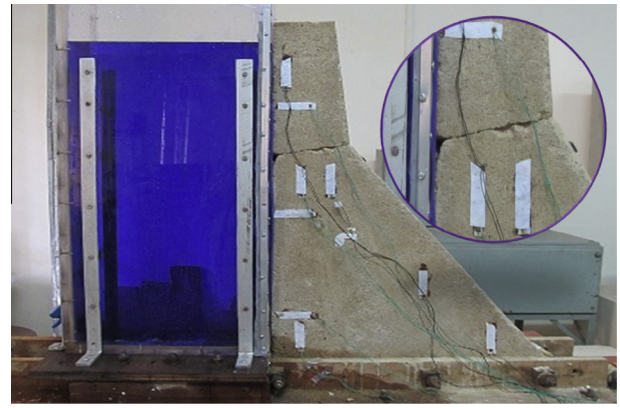


Fig. 11. Crack pattern in dam model.

5. Experimental procedure

The taller non-overflow monolith of Koyna dam has been chosen here for investigation because it would be the most highly stressed during an earthquake and because the region of the neck has a high potential for cracking. The cross section shown in Fig. 2 shows the dimensions of the monolith used in experiment. The models were placed on a shake table which would apply the excitation motions to their base. A scale of 1/150 was chosen to ensure that the table could accommodate the size and weight of the model.

Three dam models were prepared for experiments of the dam reservoir system to observe the crack pattern in the dam. Each of the dam models was constructed by mixing cement, sand, bentonite and water with a proportion of 1:42:0.4:7 and pouring it into the wooden mould which was made on a wooden base plate. Due to the large water-cement ratio, the setting and drying process required a large amount of time. The whole system was kept in a normal room temperature and was covered with wet jute sacks. The dam was de-moulded after 20 days and was placed on the shake table with the wooden base plate.

A model reservoir of size 0.78 m × 0.32 m × 0.14 m was made using 0.01 m thick perspex sheet on a wooden base plate. Materials used for casting the reservoir were perspex sheet, chloroform adhesive, araldite adhesive, steel sheets, Parker-Kalon screw which is basically self-tapping sheet metal screws, steel angles (0.012 m), a water tap and wooden base plate. The top of the reservoir kept open. The bottom and three sides of the reservoir ware modelled using perspex sheets which were added with chloroform and araldite adhesive and strengthened by steel angles and steel sheets using self tapping sheet metal screws. No perspex sheet was used on the side of the reservoir which was facing towards the dam model. A very thin polyethylene sheet was used on that face to stop the leakage of water at the interface.

The dam and the reservoir model both were assembled on a large wooden base plate. To ensure the base of the dam and reservoir did not slide over the base plate they are fixed properly using nuts and bolts. Plan and elevation of the whole system is shown in Fig. 3. The reservoir side in which a thin polyethylene paper was connected, was attached with the dam upstream face. The thin polyethylene does not allow the reservoir water to come across it but transmits the hydrodynamic effect of reservoir to the dam.

Six accelerometers (Type 4507) and nine strain gauges were attached at different positions of the dam model for collecting the necessary data during experiments. For collecting the data from accelerometer a FFT analyzer (Bruel and Kjaer) and a Pulse 6.1 key (Type 3560c) were used and a Data acquisition system

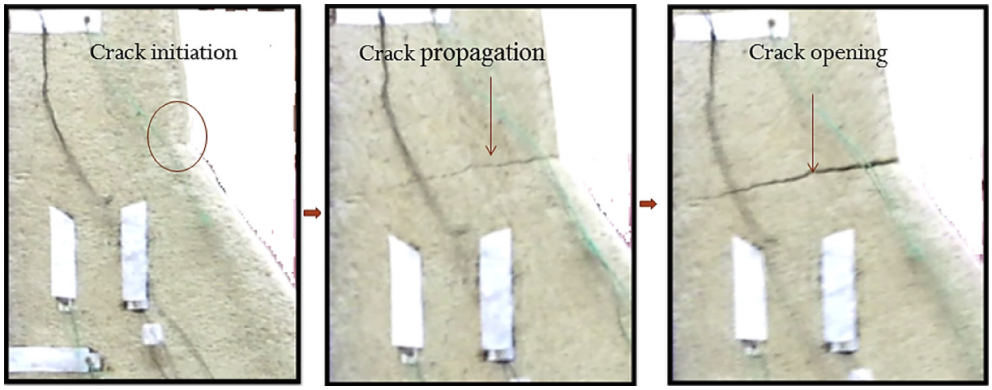


Fig. 12. Crack propagation in dam model.

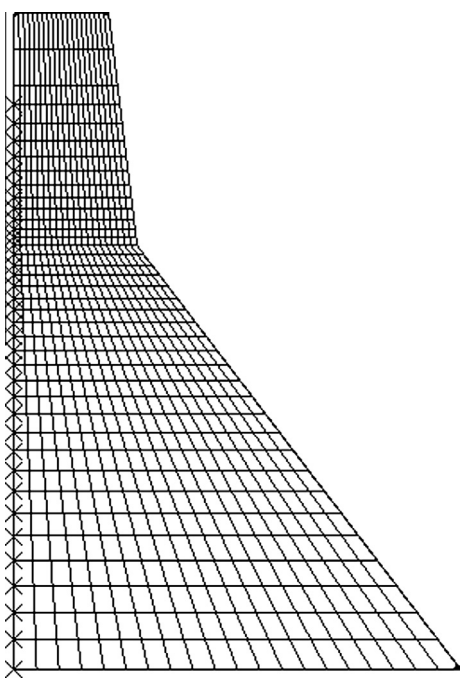


Fig. 13. Finite element discretization of model dam.

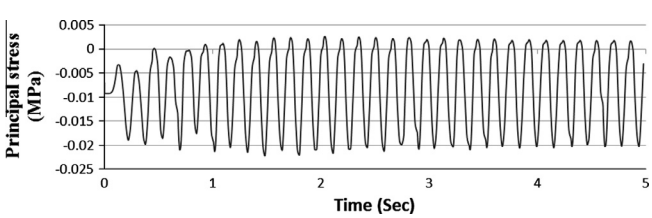


Fig. 14. Variation of stress at the point where the third strain gauge was fixed.

was used to get the strain measurement from strain gauges. The whole experimental system with the positions of accelerometers and strain gauges are shown in Fig. 4, where the circles show the positions of accelerometers and the white strips are the strain gauges.

Three experiments are conducted on the dam-reservoir system after placing them on the high frequency shake table powered by 2800 rpm and 7 HP DC motor which can produce horizontal excitation only. The shake table has a peak frequency of 50 Hz. The shake table has the capacity to a maximum stroke length of 0.15 m. Horizontal sinusoidal loadings with linearly increasing frequency over time i.e. linear sine chirp motions were applied to the shake table. Experiments were conducted on empty, half and full reservoir condition in low frequency level to get the basic response of the system and after that the system was vibrated in high frequency level to observe the failure pattern and crack propagation in the dam.

6. Experimental results

For each experiment, first the whole system was tested both in empty reservoir as well as full reservoir condition with low input frequency and strain gauge and accelerometer reading were taken to get the basic behaviour of the system. It may be important to note that the shake table has a peak frequency of 50 Hz for which the stroke length becomes less than 1 mm. For the present test, we have observed the stroke length to vary up to 3 mm.

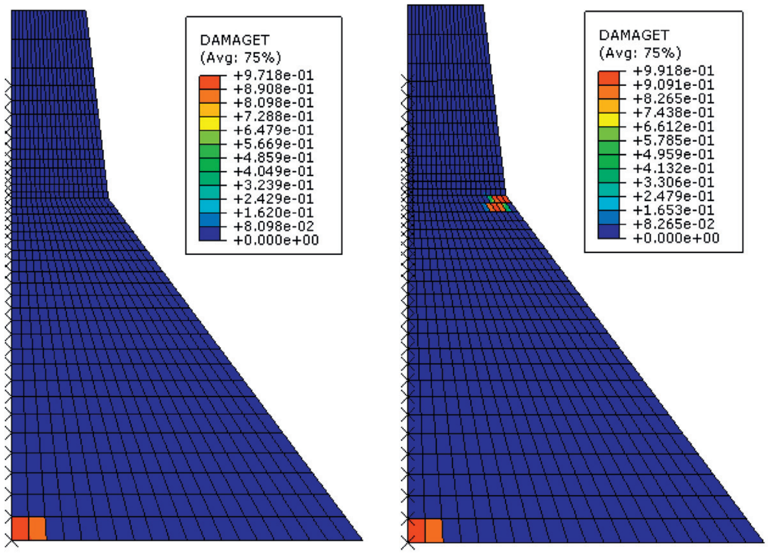
From the reading of accelerometer placed at the top of the dam a time vs. acceleration graph has been plotted. Though we provided constant amplitude of vibration to the shake table, because of resonance effect, the amplitude of resultant acceleration was not constant. At some positions the amplitudes of acceleration were amplified. These amplifications occurred when applied frequencies matched with natural frequencies of the system. To find out the natural frequencies of system a frequency response function (FRF) has been plotted by transforming the time domain data to the frequency domain using a FFT analyser. From the FRF, we can easily find out the frequencies corresponding to the amplification in accelerations and these are the natural frequencies of the system.

The time vs. acceleration graph from the reading of accelerometer placed at the base of the shake table has been plotted and shown in Fig. 5 Similarly, the time vs. acceleration graph from the reading of accelerometer placed at the top of the dam in empty reservoir condition has been shown in Fig. 6 and the frequency response function from these data has been plotted in Fig. 7.

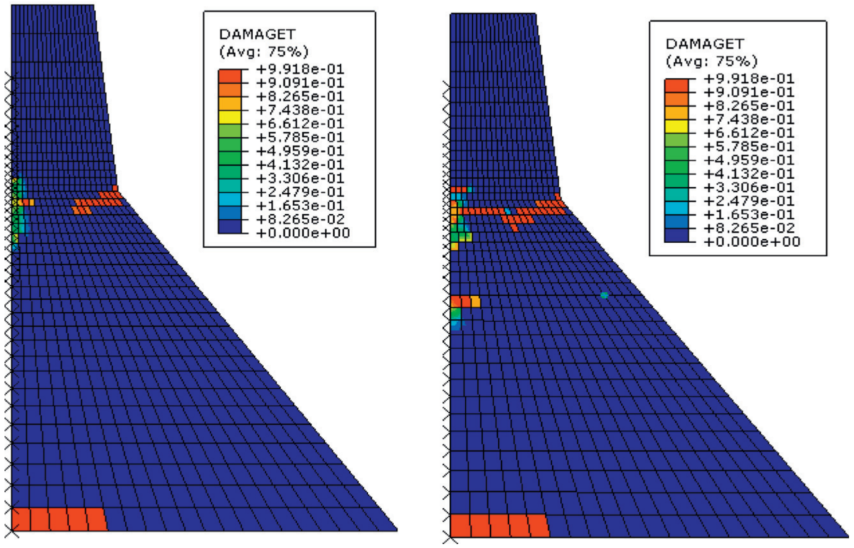
To find the effect of reservoir water on the behaviour of dam and natural frequency, the reservoir was filled with water up to 50 mm below the crest of the dam model as shown in Fig. 4, and the whole system was tested on the shake table. The accelerometer reading is plotted in Fig. 8 in time domain and frequency response function is plotted in Fig. 9 and the corresponding natural frequencies are calculated.

Strain gauge readings are taken from the data acquisition system and corresponding strains are calculated using the following formula:

$$Strain = \frac{\frac{\Delta R}{R}}{G.F.} \tag{15}$$



(a) Time = 2.92 sec (b) Time = 4.26 sec



(c) Time = 5.22 sec (d) Time = 10 sec

Fig. 15. Propagation of tensile damage in the dam.

Table 2
Comparison of natural frequencies of model dam.

Mode no.	Natural frequency (rad s^{-1})				
	Experimental results				Numerical results
	Model 1	Model 2	Model 3	Average	
Mode-1	19.34	19.13	17.63	18.7	19.27
Mode-2	48.5	48.8	54.2	50.5	51.5
Mode-3	68.7	68.4	72	69.7	70.56
Mode-3	95.3	100.4	87.8	94.5	99.73

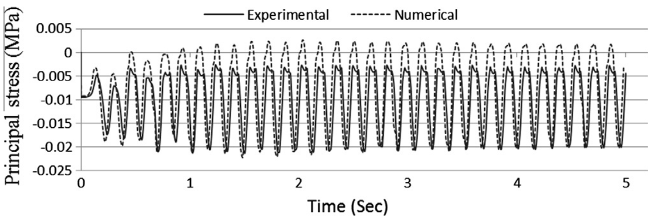


Fig. 16. Comparison of experimental and numerical stress variation.

where

ΔR = change in strain gauge resistance.
R = unstrained resistance of strain gauge.
G.F. = Gauge factor.

Here ΔR is found from data acquisition system where R is 120 and *G.F.* is 2 for strain gauges used in the present case. The stresses at positions where the strain gauges are fixed have been calculated using the strain readings. The variation of stresses with time corresponding to the first three strain gauges which were placed at three different places over the dam model, used in the second experiment conducted with full reservoir water has been plotted up to three second time span in Fig. 10 and this results is almost similar with other two experimental results.

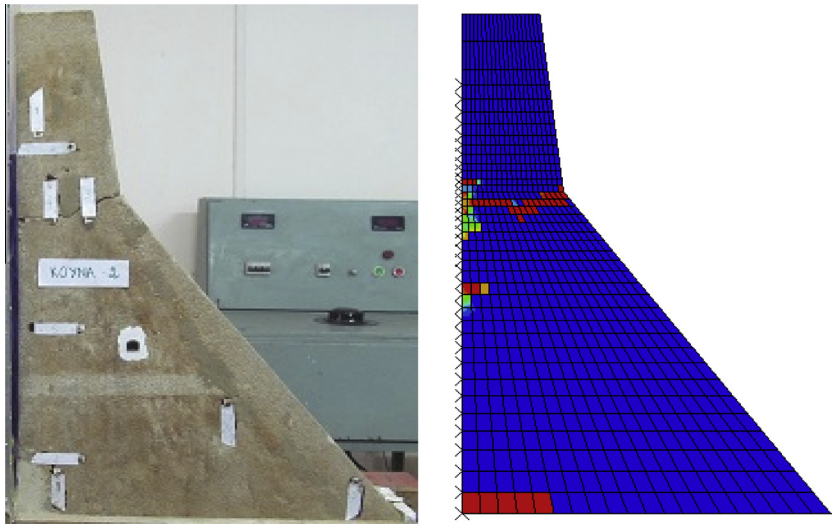


Fig. 17. Comparison of crack pattern obtained from experiments and numerical analysis.

After that the whole system (with reservoir water level 0.05 m below the top of the dam) is vibrated with high frequency loading to find the failure pattern of dam. It is observed that the neck position of the dam is the most vulnerable for failure in vibration. Crack initiates at the downstream face of the dam where the slope changes and it propagates toward the upstream face with an inclination of around 45° towards the base of the dam. A minor damage has also been observed at the heel region of the dam. The crack pattern of the dam has been shown in Fig. 11 and the sequence of crack propagation is shown in Fig. 12.

7. Numerical analysis

To compare the experimental results with numerical analysis, a 2D finite element model of Koyna dam model which is used in the experiment has been analysed using ABAQUS 6.10 [25]. The dam is modelled by CPS4R elements. The dam–foundation interaction is neglected by assuming the foundation as rigid. Hydrodynamic effect of reservoir has been considered using added mass technique. The elastic modulus, density, compressive strength and tensile strength are used as considered for model dam material. The damage plasticity properties are taken from ABAQUS 6.10 documentation and are scaled accordingly. Added mass approach is implemented using a simple 2-noded user element.

The finite element discretization and the user elements are shown in Fig. 13. For direct comparison between numerical and experimental results the actual shaking table motions recorded during the tests are used as the input motions in the analyses. First a finite element based code has been written for the dam model to get the natural frequencies of the model. These results are used in ABAQUS model to get the material damping to be used during the dynamic simulation of the earthquake.

The variation of principal stress with time of the element which represents the position of third strain gauge fixed on the dam model has been plotted in Fig. 14 and the propagation of tensile damage in the dam is shown in Fig. 15.

8. Comparison between experiment and numerical results

The first four natural frequencies are calculated for both empty reservoir and full condition for all the three experiments and the average of the results are compared with the numerical results of the dam model as shown in Table 2. The natural frequencies obtained from numerical results are showing good agreement with the experimental results.

The variation of stress corresponding to the third strain gauge obtained from second experiment which shows similar results with other experiments has been compared with the stress obtained from numerical analysis and are shown in Fig. 16. The crack patterns in dam model during all the three experiments are almost same. The crack pattern in the dam model during second experiment has been compared with the tensile damage occurred in ABAQUS model in Fig. 17.

9. Conclusions

The present investigation focuses on the development of an experimental model to observe its behaviour under dynamic excitation. The properties of the prototype dam have been scale down successfully after several trial mixes following the law of similitude to obtain the target properties of the model dam. The dam models have been casted successfully without any shrinkage crack at the base and sides of the dam model which shows the feasibility of the materials and casting procedure proposed here. The dam reservoir interaction has been modelled appropriately which can transfer the hydrodynamic forces to the dam. In the present investigation the dam foundation interaction has been neglected by assuming a rigid dam foundation. Further investigation may be carried out by incorporating the foundation flexibility to get more realistic results. Three experiments have been carried out both for empty dam and dam with adjacent reservoir water. The natural frequencies obtained from experiments and numerical analyses are showing good agreement which ensure the correctness of the developed experimental setup. The crack patterns developed in the dam model in all three experiments are similar in nature. From the experimental results it is clear that the neck position of the dam, from where the downstream slope changes is the most vulnerable area. Crack initiates at the downstream where the slope changes and propagates toward the upstream with slight inclination towards the base. Similar crack patterns are observed in case of numerical analysis as well which ensures the correctness of the developed experimental setup.

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