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Flexible Foundation Effect on Seismic Analysis of Roller Compacted Concrete (RCC) Dams Using Finite Element Method

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Abstract

Recently, Roller Compacted Concrete (RCC) dams have become one of the most applicable types of dams across the globe. However, the basic challenge in analysis of RCC dams is evaluation of the actual response under earthquake excitations with considering flexible foundation and impounded water. For this purpose, a finite element model of RCC Dam-Reservoir-Foundation is accurately developed and dynamic time history analysis is utilized to assess the seismic responses in terms of acceleration, displacements, stresses, cracking patterns and crack propagation by implementation of concrete damaged plasticity model. A verification model is carried out to show the work accuracy. Based on these explanations, the obtained results showed that, however, the hydrodynamic pressure due to the reservoir water had great influence on seismic responses of the RCC dam with rigid foundation especially in terms of displacement response but overall responses of the dam are greatly fluctuated while flexible foundation is taken into consideration.

Keywords: *RCC dam, nonlinear time history, dynamic response, dam-reservoir interaction, flexible foundation, finite element model, earthquake*

1. Introduction

As a matter of fact, it may not be promised to predict the exact time of a robust earthquake (Ghaedi and Ibrahim, 2017). This can cause wide fatalities due to collapse of structures and infrastructures including dams. Currently, a number of studies have been done by many researchers to investigate the real behavior of different types of dams such as gravity dams, arch dams and rock-filled dams under dynamic loading. Since 2002, another type of dam has been constructed employing the Roller Compacted Concrete (RCC) technology and referred to as RCC dams. Therefore, on the one hand, there is a need to study the actual response of this type of dam under various loading conditions such as hydrostatic/hydrodynamic pressure of the reservoir water and earthquake loading. On the other hand, the factual responses of the RCC dams under such loadings considering different interactions have to be taken into account including dam-reservoir and dam-reservoir-foundation interaction. Thus, in consideration of the real responses of RCC dams under the aforesaid circumstances, the subsequent steps were taken.

Hatami (1997), Lotfi (2003), Fazeli and Ghaemian (2005) and Bayraktar *et al.* (2010) considered different interactions in analysis of dams. For instance, Bayraktar *et al*. (2010) investigated the dam-reservoir-foundation interaction considering length of reservoir for near and far field on seismic performance of concrete gravity dams. In that study, different reservoir lengths, H, from 1H to 4H, were taken into account. The Lagrangian approach and Drucker-Prager yield criteria were used to investigate the dam-reservoir-foundation interaction. The obtained results illustrated that the dam was affected by reservoir length changes. However, it was obtained that the most adequate reservoir length was 3H. Huda *et al.* (2010) studied the influence of a thin layer interface element between a dam and rock foundation considering the effect of sediment on seismic response of the dam when the dam was subjected to a horizontal earthquake component. Based on that study, at the thin layer interface element zone the participation of stresses was lesser compared to the dam body and foundation stresses. Kartal (2012) used the constriction joints by means of contact element to investigate the Cine dam-reservoir-foundation interaction. In order to model the hydrostatic and hydrodynamic pressure effect, the Lagrangian method was employed using the fluid finite elements. Based on the analysis, increasing the horizontal displacement was apparent when the hydrodynamic pressure was taken into account.

In addition, researchers proposed different models to investigate the crack propagation of dams considering water pressure effect

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such as Chahrour and Ohtsu (1994), Guanglun *et al*. (2000) and Ayari (1990). In this relation, Calayir and Karaton (2005b) carried out the seismic fracture analysis of the Koyna dam considering the reservoir effect. To that purpose, a coaxial rotating crack model (CRCM) was implemented for dam concrete material. Then, to solve the dynamic equilibrium equations, an enhanced type of the HHT-α time integration algorithm was used to investigate the cracking effect on the dam seismic response. Ftima and Léger (2006) presented the calculation feasibility of In-Structure Response Spectra (ISRS) at the base level of a dam block to determine a proper compatible spectrum accelerograms. To this end, a nonlinear ISRS calculated on crack beam models was recommended to take compatible accelerograms. Long *et al.* (2009) investigated a 160-m high concrete gravity dam with and without the existence of reinforcement. Therefore, the numerical analyses were done for different nonlinearities like tensile plastic offset strain of concrete, concrete cracking, and the effect of bond-slip and stiffness recovery. the results demonstrated that the existence of reinforcement had a beneficial effect on improving the capacity of the seismic resistance of the concrete gravity dam. Mansouri *et al.* (2011) implemented a 2D Finite Element (FE) method to analyze the seismic fracture of the Koyna concrete gravity dam. Hence, the Banzant's model (nonlinear fracture mechanism criteria for crack) and the smeared crack model were utilized to measure the crack growth and to develop crack profiles, respectively. As a result, the dam with and without reservoir was modelled in order to examine the growth profiles of the cracks. Zhang and Wang (2013) investigated the duration effect of severe motions on damaging of the Koyna gravity dam. To study the crack distribution of the dam, the Concrete Damaged Plasticity (CDP) model was used for the concrete material. The obtained results indicated that the duration of a strong motion was undoubtedly associated to the accumulated damage.

Moreover, investigators carried out different aspects of the analyses to evaluate the behavior of the dams under seismic loadings. Raphael (1984) proposed equations to compute the tensile strength under static loading. Patel *et al.* (1991). developed BEM (Boundary Element Method) and FEM (Finite Element Method) to inspect flexible structures under earthquake excitations and uplift pressure by combination of the BEM, applied to the semi-infinite soil medium model, and the FEM, applied to the super-structure and foundation. The obtained results showed the reduction of base shear in sliding. Also, different coefficients of friction were directed to consider the responses under unreliable friction conditions. Zhang *et al.* (2001) used a rigid body-spring element approach to assess the static and dynamic consistency of dam foundation or slopes. The adaptability of the approach was applied to both static and dynamic analysis for random polyhedral shape blocks with various re-entrant surface properties. The safety factor of dynamic analysis was changing with time. The approach showed a capability of searching to detect the most possible sliding mass. Lotfollahi and Hesari (2008) analyzed the Karoon 1 double curvature arch dam located in Iran under two conditions; with rock support and without support by means of the ABAQUS, FE software. The time history of the dam crest displacements and stresses during the earthquake were investigated. Ghaedi *et al.* (2016) investigated the effect of shapes and sizes of openings in the Kinta RCC dam considering the hydrodynamic reservoir pressure. The results showed that, under hydrodynamic pressure, the RCC dam attracted stresses around openings and cracks occurred around them. Monteiro and Barros (2008) considered the seismic analysis of an RCC dam in Portugal with 52 m height subjected to a bidirectional accelerations using the Maximum Expectable Earthquake (MEE) and Base Design Earthquake (BDE). The results were presented based on the stresses of specific elements and the nodal displacement to evaluate the safety parameters of the RCC dam. Pathan (2012) carried out a dynamic analysis of a non-overflow RCC dam with 99.60 m height using the FE method. The used method was compared to the equivalent static method. Eventually, the results were compared in order to determine the stress ranges.

According to intensive review of the literature, it is proved that interactions between dam, reservoir and foundation are very vital in order to deliberation of RCC dams' responses subjected to severe earthquakes. Although, hydrostatic and hydrodynamic pressure effects have studied well but still there is a feel to do more investigations on the effect of flexible and massive foundation (Mridha and Maity, 2014; Wang *et al*., 2015a) on RCC dams' seismic responses. Based on this viewpoint, in present paper an attempt is made to evaluate the effects of reservoir water including hydrostatic and hydrodynamic pressure. In addition to this, a flexible foundation is considered to investigate the concurrent effects of water pressure and foundation flexibility on seismic responses of RCC dams. To aid the aim, the Kinta RCC dam located in Malaysia is selected and subjected to a bidirectional earthquake accelerations. Also, to inspect the seismic damage of the RCC dam, the Concrete Damage Plasticity (CDP) model is implemented through the commercial FE software, ABAQUS.

2. Kinta RCC Dam

In the present study, the Kinta roller compacted concrete dam was considered as a case study with 81.8 m height, located in the Ipoh district in Malaysia, in order to evaluate the seismic responses of the dam under earthquake accelerations. Fig. 1 shows different landscapes of Kinta RCC dam (Rosdi, 2008).

The geometry of deepest section of the Kinta RCC dam with fully reservoir and foundation is shown in Fig. 2. The dam body section consists of three different parts, i.e. main body, up- and down-stream faces constructed with Conventional Vibrated Concrete (CVC up- and down-stream) and CVC foundation.

3. Finite Element Modelling and Equations

3.1 Boundary Condition of the Reservoir Water

The equation of boundary condition of the reservoir is normally simplified by the statement of the mathematical expression according to the physical conditions and a number of solutions

Fig. 1. Two Different Views of Kinta RCC Dam

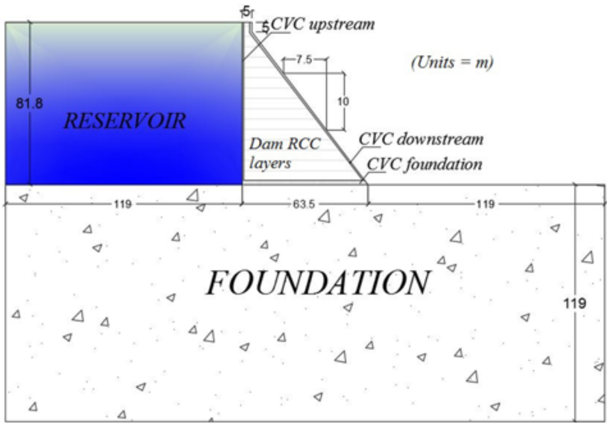

Fig. 2. Kinta Dam-reservoir-foundation System

exist for differential equations (Ghaemian, 2000). In this study, to model the dam-reservoir interaction four geometric boundary conditions are considered in the modelling (see Fig. 3) as below:

a) dam-reservoir
b) foundation-reservoir
c) free surface
d) far-end boundary condition of the reservoir

3.2 Finite Element Equations of the Dam

The interaction of the reservoir and foundation with dam body has an effect on the dynamic response of the dam and this effect has to take into account. Thus, the FE discretization of the differential equation defines the displacement of the dam structure as below:

$$M_s\ddot{u} + C_s\dot{u} + K_s u = F_g + F_p \tag{1}$$

where the M_s is mass and C_s and K_s are damping and stiffness respectively; in which $\ddot{u}, \dot{u}$ and u are the relative acceleration, velocity and displacement of the dam for nodal points of the FEM with respect to time, t. In addition, F_g and F_p are force and extra force vectors which are defined as:

$$F_g = -M_S I\ddot{u}_g(t) \tag{2}$$

and

$$F_p = QP \tag{3}$$

where, I is the unit influence vector, $\ddot{u}_g$ is the earthquake acceleration and F_p is the hydrodynamic force acting on the dam in the upstream face and it is a function of unknown parameters of the nodal pressures vector of the reservoir water, P, through the transformation matrix, Q, which is determined as:

$$Q = \int_{r_1} N_U^T n N_p \, dr \tag{4}$$

where N_p and N_u are shape functions utilized for the pressure fields and nodal displacement of the dam, respectively. r_1 is reservoir interface and n is the normal upward unit. This explanation is due to the discretization of the boundary conditions. Also, the energy dissipation inside the dam is characterized by Rayleigh damping matrix and is calculated by the following equation:

$$C_s = \alpha M_s + \beta K_s \tag{5}$$

Which α and β are constant of damping (as 5%) of the highest and lowest modes in relation to the dynamic response. The natural frequencies ω_1 and ω_2 with values of 9.571 and 51.238 rad/sec according to the first and last modes of vibrations is calculated. This gives the values of α = 0.806 and β = 0.00164 (Chopra, 2001).

3.3 Couple Model of the Dam-Reservoir

Under earthquake motion, the dam and reservoir interact together. Therefore, the hydrodynamic pressure effect due to the reservoir water and its interaction with the dam has to be recognized (Ghaemian, 2000; Chopra, 2001). Hence, to consider the reservoir hydrodynamic pressure, the force vector, F_g, due to acceleration ($\ddot{u}$) is applied to the upstream face of the dam. So, the force vector can be also defined as:

$$F_g = -\int_{r_1} N_p^T \rho \ddot{u}.n dr \tag{6}$$

Substituting the acceleration vector to the nodal vector, namely $\ddot{u} = N_u \ddot{U}$ gives the transposed matrix of the Q^T in Eq. (4) and multiplying the transposed matrix by, reservoir water density, ρ, gives the equation below:

$$K_F P = -\rho Q^T \ddot{U} \tag{7}$$

where $K_F = [G^{-1}]^T[A][G^{-1}]$, A is the linear element matrix, G is the basic solution of Laplace's equation or Green function and $K_F P = F_q$. Then, the pressure vector can be completed as:

$$P = -\rho K_F^{-1} Q^T \ddot{U} \tag{8}$$

By substituting Eq. (8) into Eq. (3), the dynamic equation for displacement of the structure can be obtained as below:

$$[M_s + \rho Q K_F^{-1} Q^T]\ddot{U} + C_s \dot{U} + K_s U = F \tag{9}$$

In which $F = F_g$ (force vector). This is the well-known equation in FE modelling which is known as added mass approach for fluid structure interaction. Accordingly, the effects of the reservoir water on the dynamic analysis of dams were considered by many researchers through a number of equations, however, the above equations is a piece of thousand equations of dam-reservoir interaction. In this study, these effects have been considered to evaluate the actual behavior of RCC dams under earthquake excitations.

3.4 Reservoir Water Domain Modelling and Relative Boundaries

To investigate the seismic analysis of dams considering hydrodynamic pressure due to the reservoir water, there is a vital need to identify the boundary condition of the reservoir. The main boundary condition may be widely considered to the reservoir water, which interacts with the dam and foundation. The equation of small movement in the linear range of an inviscid fluid (reservoir water) is represented as:

$$\frac{1}{B^r}\ddot{p} - \frac{1}{\rho^r}\nabla^2 p = 0 \tag{10}$$

where, p is the hydrodynamic pressure, B^r is the bulk modulus for reservoir, ρ^r is the reservoir water density, ∇^2 is the Laplace's operator. Therefore, the boundary conditions of reservoir can be defined as

(i) At zero pressure for the free side of the reservoir (Γ^{rs}). (P = 0)

(ii) The other side is the infinite side of the reservoir (Γ^{rt}). The equation below can be used for the infinite face of the reservoir water (Sandler, 1998):

$$\frac{1}{\rho^r}\frac{\partial p}{\partial n^r} = -\frac{\cos\theta}{\sqrt{B^r \rho^r}}\dot{p} \tag{11}$$

where, θ is the inclined angle of the plane waves and n^r is the normal upward unit of the fluid.

(iii) At the common surface of the dam-reservoir (Γ^{rd}). The equation of compatibility between solid and reservoir domain is determined as (Rizos and Karabalis, 2000):

$$\frac{1}{\rho^r}\frac{\partial p}{\partial n^r} = -n^r \ddot{u}_1^s \tag{12}$$

where the $\ddot{u}_1^2$ is dam acceleration.

(iv) At the common surface of the foundation-reservoir (Γ^{rf}). The equation of compatibility between foundation-reservoir is characterized (Fenves and Chopra, 1984) as:

$$\frac{1}{\rho^r}\frac{\partial p}{\partial n^r} = -n^r \ddot{u}_2^s \frac{1}{\sqrt{B^2 \rho^r}}\frac{(1-\alpha^r)}{(1+\alpha^r)}\dot{\rho} \tag{13}$$

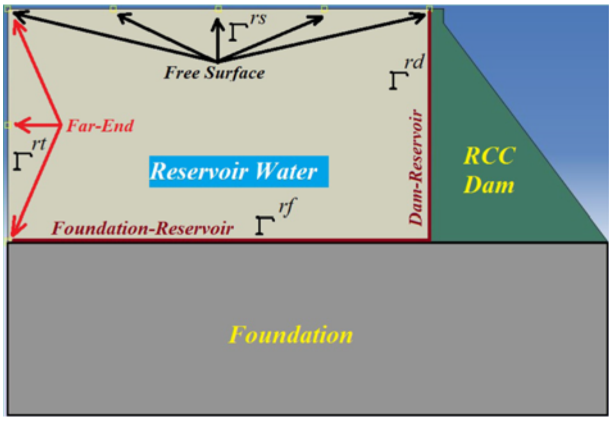

Fig. 3. The Reservoir Water Boundary Conditions in the Numerical Analysis

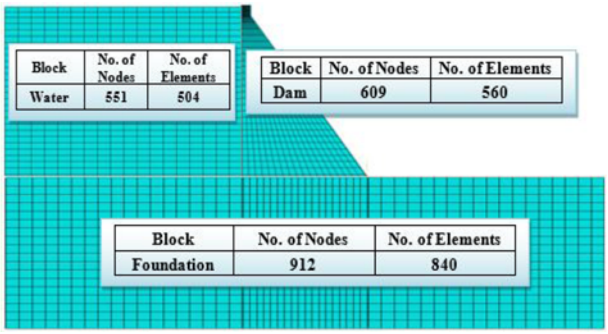

Fig. 4. Discretization of the Dam-reservoir-foundation Model

in which, α^r is the wave coefficient of the reservoir-sediment and $\ddot{u}_2^s$ is the foundation acceleration.

Figure 3 shows the boundary conditions of the reservoir water in different phases in present study.

3.5 Finite Element Model of the Dam, Reservoir and Foundation

Based on the geometry of Kinta RCC Dam, the FE model of the system is accurately developed by means of ABAQUS software considering three cases:

Case 1: Only Dam body (rigid foundation)

Case 2: Dam-Reservoir interaction (rigid foundation)

Case 3: Dam-Reservoir-Flexible Foundation interaction

To discretize the FE model of the system, four-node bilinear plane stress quadrilateral finite elements, reduced integration, hourglass control (CPS4R) is implemented to represent the dam body as well as to represent the foundation. Besides, four-node linear two-dimensional acoustic quadrilateral finite elements is conducted to represent the reservoir water. Fig. 4 indicates the number of nodes and elements of the RCC dam, reservoir and foundation.

4. Material Properties and Loadings

4.1 Material Properties

The non-linear material properties of the Kinta RCC dam are

Table 1. Material Properties used for Different Sections

Material properties	RCC	Foundation	CVC-Foundation	CVC-Facing
Poisson ratio v	0.2	0.2	0.2	0.2
Young modulus (N/m^2) *E+11	0.23	0.30	0.23	0.32
Mass density (ρ)	2386	2650	2325	2352
Compressive strength (MPa)	20	18	20	40
Tensile strength (MPa)	2	1.8	2	4
Dynamic Tensile strength, DTS (MPa)	2.5	2.25	2.5	5
Allowable tensile strength (MPa)	3.2	2.8	3.2	6.25

given in Table 1. All these properties are taken from reliable source (GHD, 2002). It is obvious that consideration of the foundation materials in the nonlinear behaviour generate more stresses and displacements in dynamic response of the dam in comparison with the condition while the foundation materials are elastic and linear. Therefore, for a massive structure such as a dam subjected to the ground accelerations, it is recommended to perform non-linear dam-foundation interaction analysis for evaluation of its precise behaviour. (Burman *et al.*, 2010). Thus herein, the tensile strength of the concrete materials is defined as 10% of their compressive strength (US Army Corps of Engineers, 2000). The foundation rock based on the reference data (GHD, 2002) has been modified by concrete material during the dam construction, thus, this modification is taken into consideration to compute its tensile strength. Furthermore, for reservoir water the density, ρ = 1000 Kg/m^3 and the bulk modulus, K_w = 2107 MPa was implemented. It has to mention that, the effect of uplift pressure has not been considered in Case 2 due to rigidity of the foundation as many researchers have taken the same manner (Calayir and Karaton, 2005a; Akkose *et al*., 2008; Gao *et al*., 2011; Khazaee and Lotfi, 2014; Demirel, 2015). Consequently, because of neglection of the uplift pressure in Case 2, no uplift force is taken into account for Case 3 as well. This is only because of checking the effect of modelled foundation itself (without considering concurrent effects of the uplift force and the foundation flexibility) to show how its flexibility affect the dam response under seismic ground motions.

4.2 Concrete Damaged Plasticity (CDP)

In the incremental model of plasticity, the strain tensor (ε) is separated into two portions counting the elastic strain (ε^e) and plastic strain (ε^p) in which the below equation can be stated for the linear elasticity:

$$\varepsilon = \varepsilon^e + \varepsilon^p \tag{14}$$

By considering the above variables within a given time interval, the stress tensor can be calculated as:

$$\sigma = (1-d)\bar{\sigma} = (1-d)E_0(\varepsilon - \varepsilon^p) \tag{15}$$

in which $d = d(k)$ and is the scalar stiffness degradation variable which is ranged from 0 to 1 and E_0 is the undamaged elastic stiffness for concrete material. The failure mechanism of the material associated with the damage, thus, reduction of the elastic stiffness that supposed a function of the internal variable $\{k\}$ including of compressive and tensile variables (k_c, k_t) . The tensile and compressive damage functions are nonlinear parameters. They are computed by means of uniaxial response in compression with practical data. Hereupon, the effective stress can be defined as:

$$\bar{\sigma} = (\sigma/1-d) = E_0(\varepsilon - \varepsilon^p) \tag{16}$$

4.3 Seismic Loading

The transverse and vertical accelerations of the Koyna earthquake acceleration (December 1967, India) are applied to the Kinta RCC dam as indicated in Fig. 5.

4.4 Hydrostatic and Hydrodynamic Loadings

The Kinta RCC dam was modelled considering hydrostatic pressure for Case 1. It is obvious that the proportion of hydrostatic pressure decreases from the base level to the crest zone along the dam height at upstream face. The value of the hydrostatic pressure at base level of the dam was computed to be 0.802458 MPa. The hydrodynamic pressure of the reservoir was deliberated by

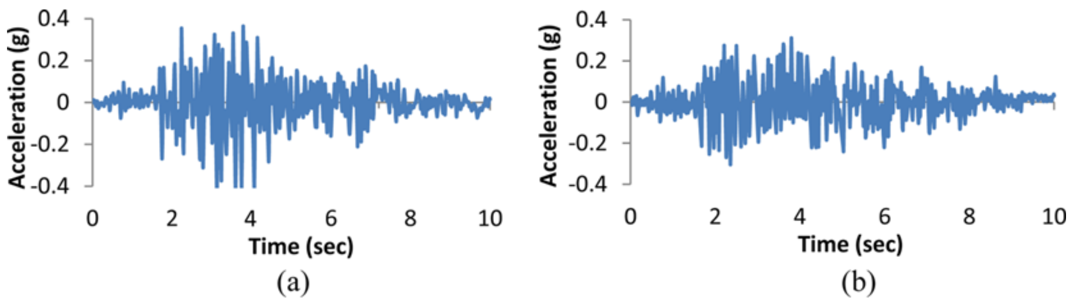

Fig. 5. Recorded Acceleration of 1967, Koyna Earthquake: (a) Transverse Acceleration, (b) Vertical Acceleration

modelling of the reservoir water to interact with the dam and foundation. The non-linear analysis of the RCC dam was accomplished for both hydrostatic and hydrodynamic pressure of the reservoir water to be compared to each other.

5. Validation

The main and the most complicated scenario in the analysis of dams, is dam-reservoir coupling model such that the interaction between water and dams' body in upstream side has been the key concern of several investigators in different analysis aspects. In this direction, it is clear that if the problem of dam-reservoir water interaction be paved, subsequently, in the same manner the foundation-water interaction problem will be solved. As a result, present study pays a high attention to model the dam-reservoir water interaction in a satisfactory level by verifying the method with a similar scenario done by others. In other words, a sensitive care is made to precisely implement the dam-reservoir couple model in which the acoustic elements (water elements) meet the solid elements (dam elements) using ABAQUS software. Based on these statements, the Koyna dam, as the most investigated case in relation to dam engineering, has been chosen to validate the accuracy of the software modeling. Consequently, the present verification considers a dynamic analysis of the Koyna dam subjected to the Koyna earthquake, December 1967, to verify the displacement response of the Koyna dam as well as the crack propagation in the dam body using CDP model. This example shows a usefulness of the CDP model for the evaluation of the structural stability and damage of the dam subjected to ununiformed loading. The Koyna dam is selected for validity of present study because it has been widely studied by a many investigators, as mentioned above, such as (Chakrabarti and Chopra, 1973; Bhattacharjee and Leger, 1993; Ghrib *et al.*, 1995; Omidi *et al.*, 2013; Huang, 2011; Mansouri *et al.*, 2011; Burman *et al*., 2012; Wang *et al.*, 2015b; Hariri-Ardebili and Seyed-Kolbadi, 2015; Hariri-Ardebili *et al.*, 2016). Herein, to validate the methodology used in present study the Koyna dam is modelled and its responses under seismic motion in terms of displacement response and crack propagation are compared to related study done by above-mentioned researchers. The geometrical section of the Koyna dam is shown in Fig. 6.

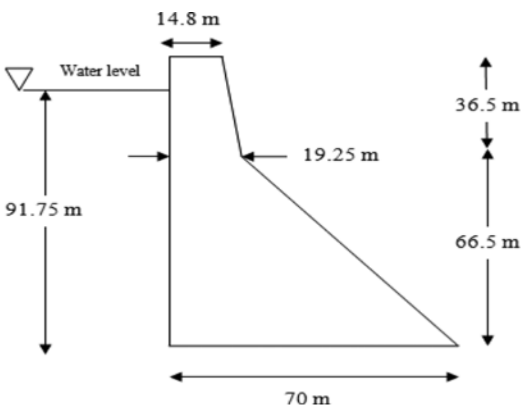

Fig. 6. Geometrical Section of the Koyna Dam

Table 2. Comparison of the Maximum Horizontal Displacement of the Dam Crest

Study	Crest Displacement (cm) in Upstream Direction	Crest Displacement (cm) in Downstream Direction
Present	4.22	3.33
Zhang *et al.* (2013)	4.07	3.41
Different percentage (%)	3.6	2.3

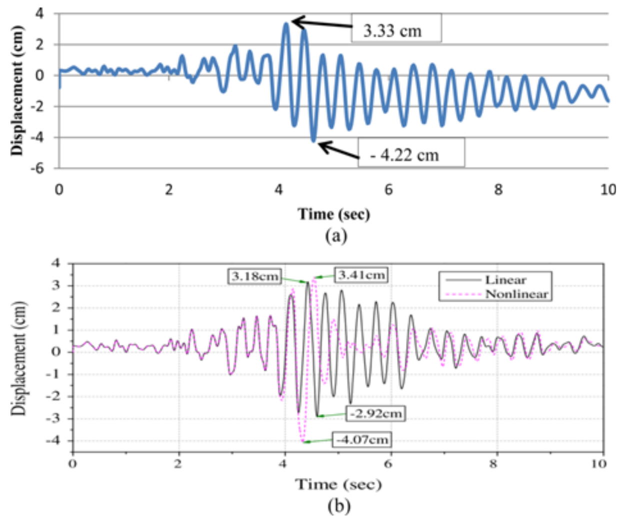

Fig. 7. Comparison of Horizontal Displacement for Dam Crest in Present and Zhang *et al.* (2013) Study

5.1 Displacement Response

The nonlinear displacement analysis of the Koyna dam under the Koyna ground motions is carried out and results of the maximum horizontal displacement of the dam crest is verified to Zhang *et al.* (2013) study as demonstrated in Table 2. It can be seen from the table that, the obtained displacement from present study has a fine agreement with Zhang *et al.* (2013) study in which the differences are less than 4% and 3% in upstream and downstream directions, respectively. Fig. 7 depicts the displacement time history analysis of the both present and Zhang's studies.

5.2 Tensile Damage and Cracking

The displacement of the dam is the main reason of the dam cracking. The tensile damage (cracking) of the Koyna dam due to the above obtained displacement considering dam-reservoir interaction is investigated and the result is also verified to the other studies as illustrated in Fig. 8. This comparison confirms that the methodology used in present study is capable well to predict cracking in body of the Koyna dam as the result shows a nice agreement between the current work and other given works. Furthermore, Fig. 9 also demonstrates the experimental scaled tests of the Koyna dam which affirms the precision of the all

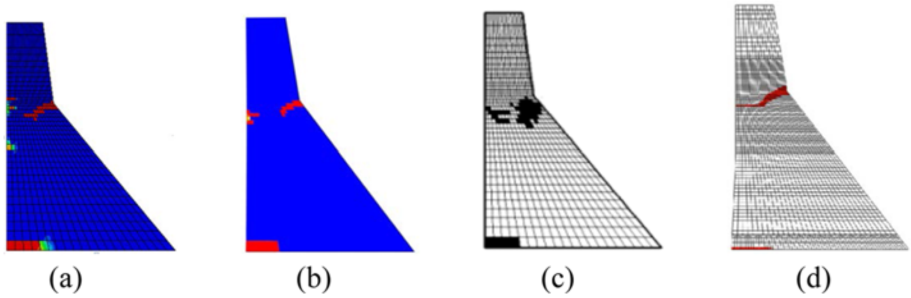

Fig. 8. Crack Propagation of the Koyna Dam in: (a) Present Study, (b) Huang (2011), (c) Mansouri *et al.* (2011), (d) Zhang and Wang (2013) Studies

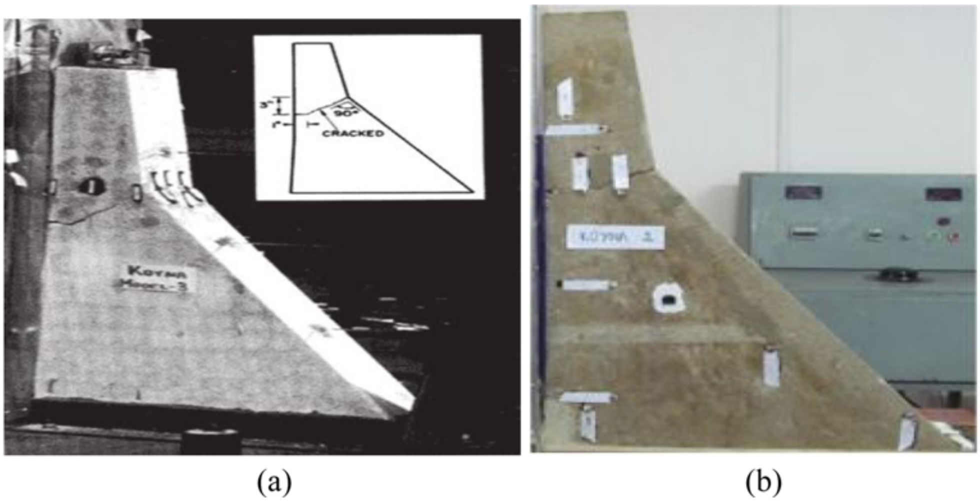

Fig. 9. The Experimental Failure Response of Koyna Dam: (a) National Research Council (1990), (b) Mridha and Maity (2014) Study

studies, including present analysis, in term of forecasting the crack tendency of the dam under the same earthquake excitations, i.e., Koyna excitations.

6. Results and Discussions

In the validation section, the methodology used has been confirmed. Regardless of shape and size of the dams, it must be guaranteed that the dam-reservoir interaction or solid-acoustic element interaction is accurately performed. Based on this assertion, the interaction of the Kinta dam and the foundation (both as the solid elements) is correspondingly implemented with the reservoir water (as acoustic elements) and then the dam is subjected to the same loadings as the Koyna dam has been verified.

Figure 10 demonstrates the time history analysis of the relative horizontal acceleration of the Kinta RCC dam crest. According to this analysis, the horizontal acceleration of the dam in Case 1 is 39.27 m/s^2. The acceleration response of the dam in Cases 2 and 3 is varied to be 27.51 m/s^2 and -31.94 m/s^2 (toward the downstream direction), respectively. It can be concluded that, the hydrodynamic water pressure has a favourable influence to reduce the dam acceleration; reducing the dam acceleration by 30% respect to Case 1. Taking the flexible foundation into consideration again increase the absorbed acceleration by 16% respect to Case 2, however, its effect mitigates the crest acceleration by 18.7% compared to Case 1. It can also from this figure be observed that, the period of the maximum acceleration attracted by the dam crest is delayed to 3.63 second when the dam is constructed on the flexible foundation.

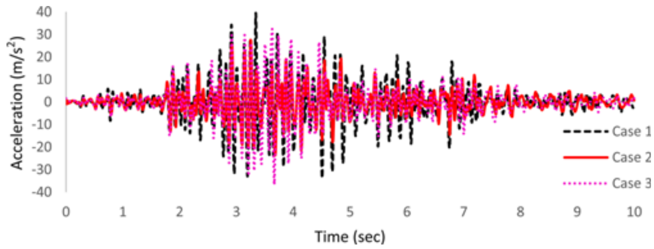

Fig. 10. Horizontal Acceleration of the Dam Crest In Three Cases

Figure 11 depicts the effect of flexible foundation on relative horizontal displacement of the dam. The displacement of the dam in Case 1 is only 2.32 cm when no hydrodynamic pressure and flexibility of the foundation is considered. Taking hydrodynamic water effect into consideration in Case 2 proves the effect of water pressure on the dam at upstream side by 20% increase in displacement, i.e. 2.79 cm. The dam experiences maximum movement at its crest when the foundation flexibility is taken

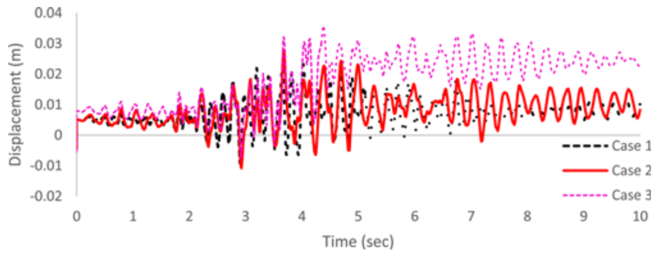

Fig. 11. Dam Crest Horizontal Displacements in Three Cases

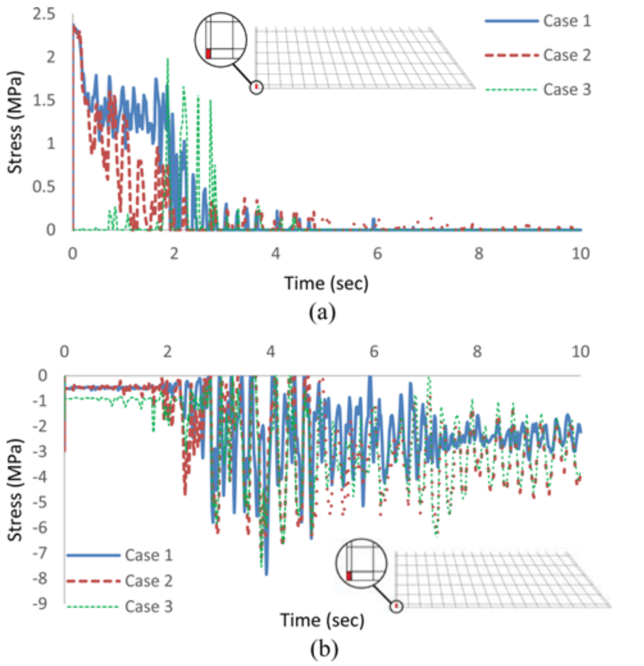

Fig. 12. Stress Attraction of the Heel Element During Seismic Analysis: (a) Maximum Principal Stress, (b) Minimum Principal Stress

into account and it is displaced 3.59 cm which means 54.7% more in compare to Case 1 and 28.7% more in compare to Case 2. In addition, the period of vibration for three cases are noticeable. The time of the maximum displacement occurrence is 4.38 second for when the foundation is considered in the analysis, whereas, for Cases 1 and 2 the peak displacement happens at 4 and 3.68 seconds, respectively. After 4.38 second, a remarkable residual displacement can be observed due to concurrent effect of the water and foundation during analysis in Case 3. These results show that, the interaction of the foundation with the dam and water influence the dam vibration period, dam displacement value and tendency. Also, the effect of flexibility of the foundation on the dam is a remarkable residue displacement. This phenomenon can expose the dam to a serious vulnerability in next seismic excitations. Based on this analysis, it is a crucial requirement to consider foundation effects in seismic analysis of RCC dams. Meanwhile, it should be noted the height and shape of the dams' crest may significantly affect their displacement response. In this study as demonstrated in Fig. 2, the crest height of the Kinta RCC dam is only 5 m and it does not have that sensibility like koyna dam with 36.5 m crest height (see Fig. 6) in dams' seismic analysis. For instance, the Koyna dam with crest height of 36.5 m has been displaced 6.13 cm (Sarkar *et al.*, 2007) under same earthquake that applied in the present study. Furthermore, the foundation materials such as young modulus of the foundation (E_f) can also influence the dam responses as researchers have proved that by increasing the E_f, total displacement of the dam crest can be decreased (Burman *et al.*, 2010; US Army Corps of Engineers, 2000).

It has confirmed that the heel element as an intersection element in dam-water-foundation interactive system can absorb a high value of stress, thus, this element is selected in order to investigation of stress attraction under the earthquake motion. This element is shown in the body of Fig. 12. According to the theory or criterion of the principal stresses, the failure of materials is arisen once the principal stresses distributed in a body surpasses uniaxial ultimate compressive or tensile strength of the materials. Based on this criterion, the materials are safe when $\sigma_{max} < \sigma_{ut}$ and $\sigma_{min} < \sigma_{uc}$ in which σ_{max} and σ_{min} are the maximum and minimum principal stress, respectively. σ_{ut} and σ_{uc} are the ultimate tensile and compressive strength, respectively.

The values of the time history responses of the maximum and minimum principal stresses of this element are plotted in Fig. 12(a) and Fig. 12(b), respectively.

It can be seen from Fig. 12(a) that, the maximum stress occurred with the same value of 2.38 MPa for Case 1 and Case 2. This amount was reduced by 17% to 1.98 MPa in Case 3 when the of flexible foundation is added to the system. Moreover, close to initial time of analysis the element absorbs the maximum stress, namely at time 0.001 second in both Cases 1 and 2. However, due to presence of flexible foundation, this period is delayed to 1.87 second for the element to attract its maximum value. The values of the minimum principal stress are depicted in Fig. 12(b). The stress values for Case 1, 2 and 3 are -7.84 MPa, -7.15 MPa and -7.55 MPa, respectively. These values show the effect of water pressure in Case 2 by 9% stress reduction and in Case 3 by 3.7 % stress reduction while the concurrent effect of flexible foundation and hydrodynamic pressure is considered. Table 3 summarizes the above expressions.

The contour lines and location of the occurred maximum and

Table 3. Stress Absorption Considering Dam Body Part

Cases	Case 1	Case 2	Case 3	Percentage of changes (%)		
				2 to 1	3 to 1	3 to 2
Max. Stress (Mpa)	2.38	2.38	1.98	0	17	-17
Min. Stress (Mpa)	-7.84	-7.15	-7.55	9	-3.7	5.6

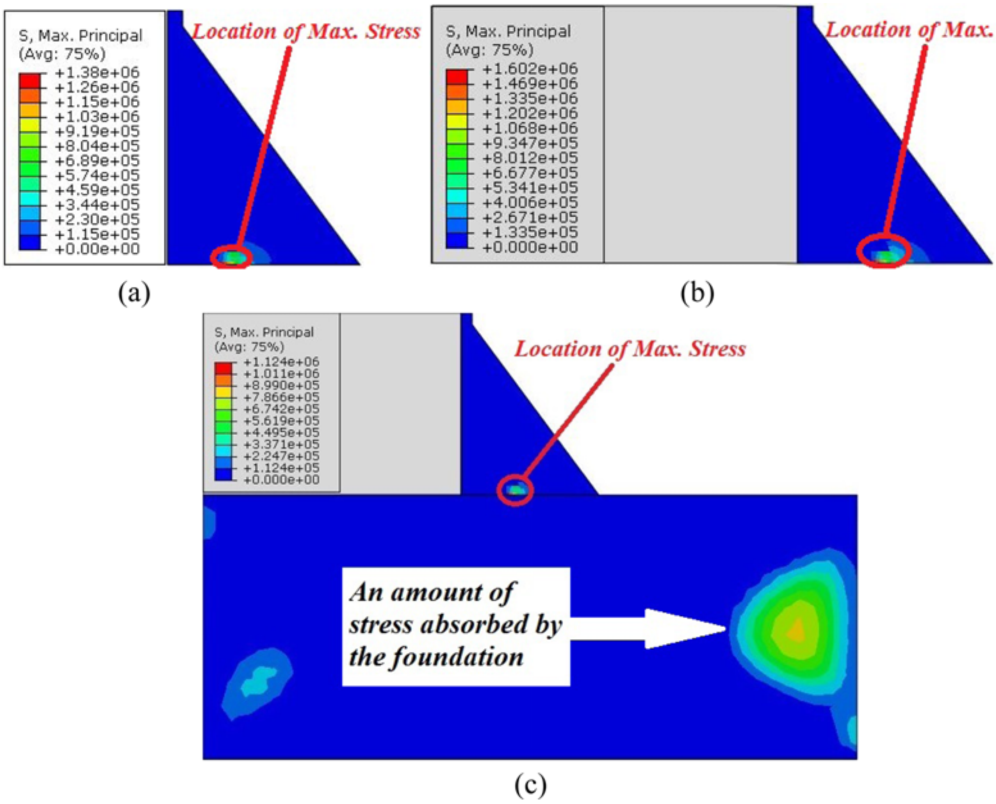

Fig. 13. Maximum Principal Stress in Different Cases: (a) Case 1, (b) Case 2, (c) Case 3

minimum principal stresses of the three cases are indicated in Fig. 13 and Fig. 14, respectively. Since the stresses inside the dam body are varied in each increment during time history analysis, therefore herein, the final frame of the analysis (end of the analysis), i.e. at 10 second, is selected to investigate the stress responses of the system and to show the effect of the foundation flexibility on changes of the dam stresses. As it is indicated in Fig. 13, in comparison to Case 1, the stress value in Case 2 rose up by 16% from 1.38 MPa to 1.60 MPa. By considering the foundation effect in Case 3, the maximum stress is rapidly declined by 18% and 29% from 1.38 MPa (in Case 1) and 1.60 MPa (in Case 2) to 1.124 MPa. This mitigation of maximum principal stress in the dam body in Case 3 is because of attracting a noticeable stress amount by the foundation as it can be seen in Fig. 13(c).

Based on the principal stress theory, it can be concluded from Fig. 13 and Table 1 that, $\sigma_{\max} < \sigma_{ut}$, therefore, the dam stands in a safe condition.

Figure 14 illustrates the contour lines and location of the minimum principal stresses of the three cases. This figure clearly shows that, how the hydrodynamic water pressure and foundation change the location of the occurred minimum principal stress in the system at the end of analysis (10 second). Presence of the water pressure at the upstream side of the dam changes the location of minimum stress from the downstream side to the left-bottom side of the upstream. This variation becomes more momentous when the location of the minimum principal stress goes to be out of the dam body and attracts by the flexible foundation. Based on these statements, the minimum stress value by -2.68 MPa is determined for Case 1. This value is increased by 46% to -3.91 MPa in Case 2. The significant stress absorption by the flexible foundation obviously shows its effect on the RCC dam system in Case 3. In Case 3 the minimum principal stress in comparison to Cases 1 and 2 is markedly declined by 2156% and 1446%, respectively in which the minimum stress value is approached to 60.45 MPa attracted by the foundation. The location of this huge amount of stress laid at the right-bottom corner of the foundation as indicated in the Fig. 14(c). Based on the principal stress theory, it can be concluded from Fig. 14 and Table 1 that, in all cases $\sigma_{\min} < \sigma_{uc}$. This states that, the Kinta RCC dam body stands in a safe condition. Note that, in Case 3 the minimum principal stress of the flexible foundation (60.45 MPa) exceeds from its ultimate compressive strength (18 Mpa). This means that, the foundation is out of the safety condition. As a result, it is observed from Fig. 13 and Fig. 14 that, the hydrodynamic pressure, however, has a remarkable effect in terms of the stress value and location but the effect of flexible foundation is enormously detected, particularly in relation to the minimum principal stress. The summary of the maximum and minimum principal stress results considering water pressure and foundation effect is tabulated in Table 4.

7. Seismic Damages Response

In this study, the Concrete Damaged Plasticity (CDP) model is

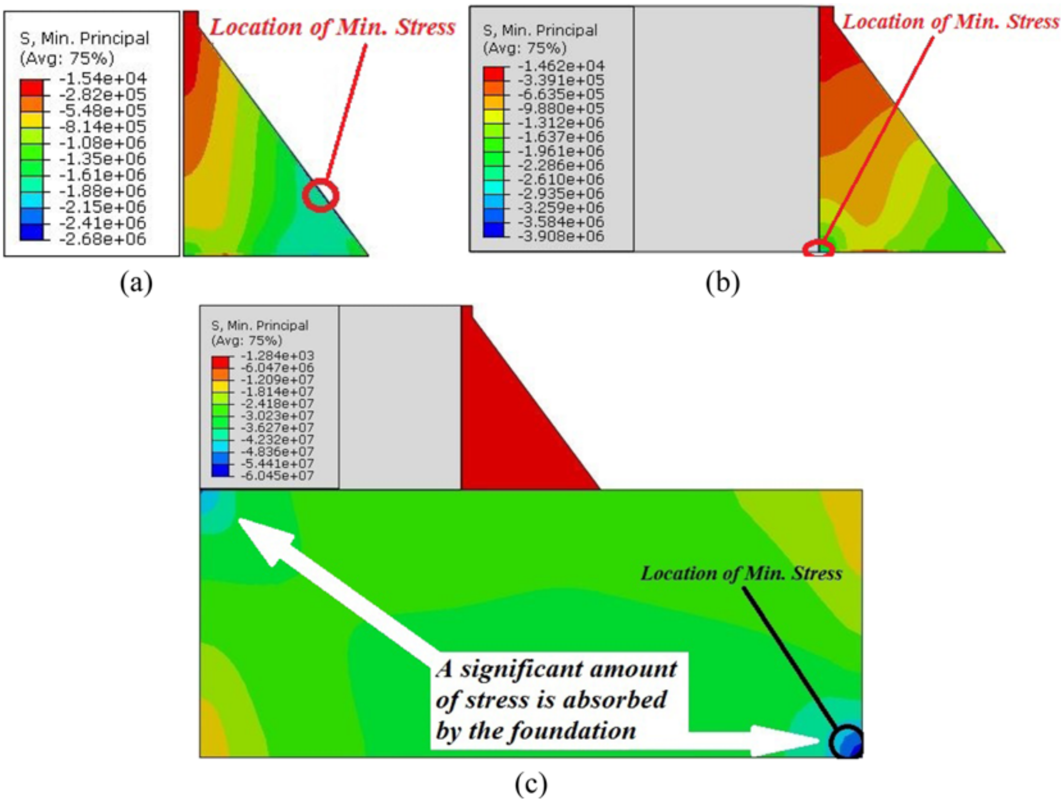

Fig. 14. Minimum Principal Stress in Different Cases: (a) Case 1, (b) Case 2, (c) Case 3

Table 4. Stress Absorption Considering Whole Parts

Cases	Case 1	Case 2	Case 3	Percentage of changes (%)		
				2 to 1	3 to 1	3 to 2
Max. Stress (Mpa)	1.378	1.602	1.124	16.2	-18.4	-29.8
Min. Stress (Mpa)	-2.68	-3.91	-60.4	6	2157*	1447*

*absorbed by the flexible foundation

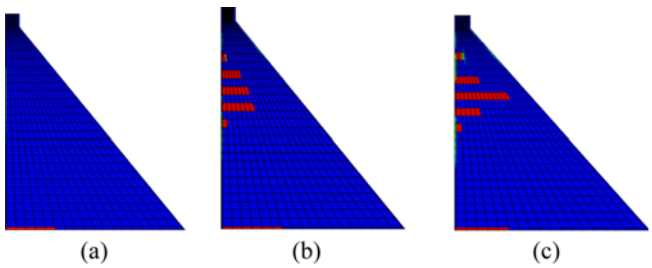

Fig. 15. Tensile Damage Response of Kinta RCC Dam Body in Three Cases: (a) Case 1, (b) Case 2, (c) Case 3

used in order to assess the dam cracking and its seismic performance. The hardening variables of damaged states in compression and tension are described independently by $\tilde{\varepsilon}_c^{pl}$ and $\tilde{\varepsilon}_t^{pl}$. These variables are known as equivalent plastic strains in compression and tension, respectively. The tensile damage is a non-decreasing quantity which is associated with the tensile failure. The variable of stiffness degradation, d or SDEG, may decrease or increase by reflecting the effects of the stiffness reparation associated with the cracks opening and closing. Therefore, by assuming no compressive damage, i.e. $d_c = 0$, the combination of SDEG with $d > 0$ and tensile damage with $d_t > 0$ in a certain point of material indicates an opened crack, whilst, the combination of SDEG with $d = 0$ and tensile damage with $d_t > 0$ indicates a closed crack. To evade unwarranted mesh-sensitive results, because of lacking reinforcement in the dam, a cracking criterion of fracture energy is utilized to specify the tensile post-failure behaviour determining a stress-displacement curve as an alternative for a stress-strain curve. This is done by means of post-cracking stress-displacement curve in present study.

By taking these explanations, Fig. 15 shows the tensile damage of the RCC dam. As illustrated in this figure, development of the cracks was only observed in the heel zone whilst the hydrostatic pressure effect was applied to RCC dam in Case 1. In Case 2, when the hydrodynamic pressure owing to the reservoir water is added and imposed to the model, the crack propagation increases and appears in the middle region of the dam in upstream face and tends toward the downstream face. The crack pattern became

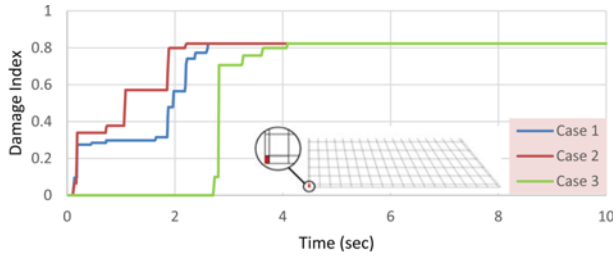

Fig. 16. Time History Tensile Damage Response for Heel Element

more severe in Case 3 compared to other cases, while the flexible foundation is taken into account. This occurrence is pertaining to the determination of young modulus of the foundation (see Table 1) which could be considered as a flexible foundation in the present study. Thus, the flexible foundation leads the dam to experience more displacement; causes more damage inside the RCC dam body.

The tensile damage is tabulated in terms of cracking displacement utilizing the post-cracking displacement curve. As mentioned earlier, the dam concrete crushing (compressive failure) or d_c which is equalled to zero causes the stiffness degradation damage. The provided range of the damage is calculated according to the concrete tension stiffening which is directly related to the tensile strength of the concrete materials of the dam and CVCs, as given in Table 1. Therefore, the damage range for concrete is specified between $d_t = 0$ (undamaged) and $d_t = 0.823$ (fully-damaged) as indicated in Fig. 16. It can be seen from Fig. 15 that, the heel element at the upstream face is the first damaged element. Hence, Fig. 16 demonstrates the time history analysis of the crack propagation for this element. This figure clearly shows the effect of flexible foundation to delay the cracking of the element. In other words, the onset of the element cracking is begun at 0.137 and 0.136 second for both cases 1 and 2, respectively, whilst, by considering the foundation effect this time is delayed to 2.72 second. In addition to this, the element becomes fully damaged at 2.62 second in Case 1. Although, the hydrodynamic water effect expedites the fully cracking time to 2.22 second, but once again the flexibility effect of the foundation lengthens the time to 4.09 second to be the element fully cracked.

Since the Stiffness Degradation (SDEG), Plastic Strain Magnitude (PEMAG) and Compressive Equivalent Plastic Strain (PEEQ) parameters are associated with the tensile damage, therefore, Case 3 herein is chosen to indicate the relation of the parameters at a certain time (3.673 sec) as presented in Figs. 17(b), 17(c) and 17(d). This consideration shows the response of the nonlinear dynamic analysis of the RCC dam under earthquake loading. The SDEG parameter plays an important role in the stiffness recovery associated with damage in the seismic analysis.

8. Conclusions

The responses of the RCC dam in 3 cases including dam (Case 1), dam-reservoir interaction (Case 2) and dam-reservoir-flexible foundation interaction (Case 3) have been considered with highlighting the nonlinear seismic analysis to investigate the dynamic response of the Kinta RCC dam. For easiness, the deepest section of the RCC dam has been idealized under plane stress (dam and foundation) and acoustic (reservoir water) condition respectively. To assess the dam cracking the CDP model has been utilized. There has no uplift pressure been considered during the analysis. According to these, the following conclusions are drawn:

- The flexible foundation effectively mitigates the acceleration response of the dam.
- The flexible foundation is more efficient to reduce the acceleration response of the dam rather that the displacement, as in Case 3 the crest displacement significantly increases in comparison to Cases 1 and 2.
- The flexible foundation arises a notable residue displacement after an earthquake excitation. This phenomenon can subject the dam to a drastic vulnerability during future seismic motions; prone to severe cracking.
- The foundation flexibility effect decreases the principal stress components of the dam body due to hydrostatic and hydrodynamic pressure.
- The flexible foundation causes the dam to have the severest damage (cracking) in its body.
- The flexible foundation increases the time period in terms of attracting the peak displacement and acceleration response, the peak stress absorption by the dam as well as cracking progress of the dam.

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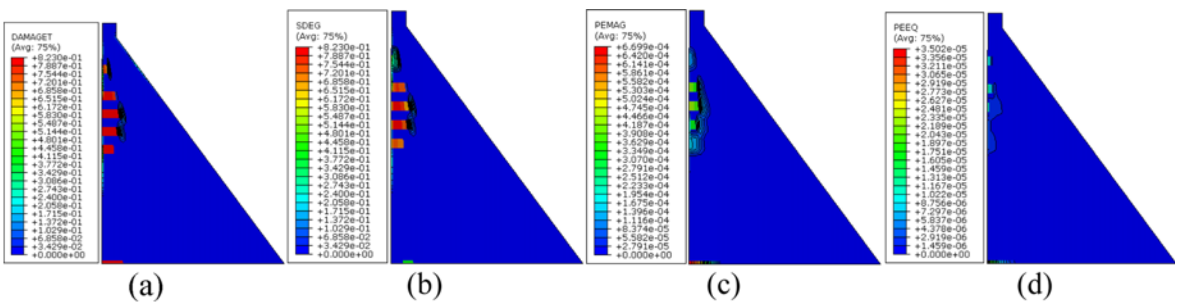

Fig. 17. The Damage Contours of the Kinta RCC Dam Body at 3.673 sec: (a) Tensile Damage, (b) SDEG, (c) PEMAG, (d) PEEQ

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