Structural Engineering and Mechanics, Vol. 33, No. 1 (2009) 31-46 31
DOI: http://dx.doi.org/10.12989/sem.2009.33.1.031

Damage mechanics approach and modeling nonuniform cracking within finite elements for safety evaluation of concrete dams in 3D space

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(Received April 16, 2008, Accepted July 22, 2009)

Abstract. An anisotropic damage mechanics approach is introduced which models the static and dynamic behavior of mass concrete in 3D space. The introduced numerical approach is able to model non-uniform cracking within the cracked element due to cracking in Gaussian points of elements. The validity of the proposed model is considered using available experimental and theoretical results under the static and dynamic loads. No instability and stress locking is observed in the conducted analyses. The Morrow Point dam is analyzed including dam-reservoir interaction effects to consider the nonlinear seismic behavior of the dam. It is found that the resulting crack profiles are in good agreement with those obtained from the smeared crack approach. It is concluded that the proposed model can be used in nonlinear static and dynamic analysis of concrete dams in 3D space and enables engineers to define the damage level of these infrastructures. The performance level of the considered system is used to assess the static and seismic safety using the defined performance based criteria.

Keywords: concrete dams; dam-reservoir interaction; damage mechanics; dynamic analysis; non-uniform cracking; performance level.

1. Introduction

The damage mechanics approach is one of the most recent models which have been used to consider the nonlinear behavior of mass concrete. Some of the main advantages of this theory in comparison with the elasto-plastic constitutive model or the fracture mechanics based models such as the smeared crack approach are given in Gunn (2001a).

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The damage mechanics theory was introduced for the first time in modeling the creep in metals, Ju (1990). Until now, various models based on this theory have been presented to predict internal damage in ductile and brittle materials. Also, several approaches have been developed to model the non-process and process damage in brittle materials (Kachanov 1980, Krajcinovic and Foneska 1981, Krajcinovic 1983, Kachanov 1987, Ju 1989, Pramono and William 1989, Ju 1991). In the field of dam analysis and performance evaluation, the damage mechanics theory has been used to analyze concrete gravity dams under the static and dynamic loading conditions as referred in Ghrib and Tinawi (1995a, 1995b), Mirzabozorg (2004). In the study represented by Ghrib and Tinawi (1995a, 1995b), the damage mechanics theory was developed to model the static and dynamic behavior of mass concrete in 2D space.

Gunn (2001a, 2001b) used the damage mechanics theory in 3D space for analyzing concrete structures under the static loads. The proposed model satisfies the fracture energy conservation principle and has appropriate criteria for both of the crack initiation and the crack propagation. Horii and Chen (2003) illustrated various methods to model the nonlinear behavior of mass concrete in gravity dams. In their study, the problems in crack modeling, computational algorithm and damping implementation are discussed in conjunction with safety assessment of concrete dam against large earthquakes. They showed that the formulation for the crack-embedded element has an analogy with that of computational plasticity. Oliver *et al.* (2003) presented a continuum strong discontinuity approach to consider cracking of concrete. Criteria for onset and propagation of material failure and specific finite elements with embedded discontinuities were sketched and some numerical simulations of cracking in plain and reinforced concrete specimens were presented. Pekau and Yuzhu (2004) presented a study on the dynamic behavior of the fractured Koyna dam during earthquakes using the distinct element method. They modeled the hydrodynamic effect using the added mass approach.

Several investigators such as Hall (1998), Malla and Wieland (1999), Espandar and Lotfi (2003), Lotfi and Espandar (2004) and Mirzabozorg and Ghaemian (2004) have represented models based on the smeared crack approach to study the nonlinear behavior of mass concrete in 3D space. Some of other investigators consider the effect of vertical construction joints on the nonlinear behavior of arch dams as referred in Mirzabozorg and Ghaemian (2004).

Calayir and Karaton (2005) presented a paper in which the earthquake damage response of concrete gravity dams is considered including the effect of reservoir interaction. In their work, 2D damage mechanics approach similar to that introduced by Mirzabozorg *et al.* (2004) utilized to model the nonlinear behavior of dam body in 2D space and the reservoir was modeled in the lagrangian space. Lotfi (2005) considered the accuracy and performance of simplified one dimensional model for fluid-foundation interaction in comparison with the rigorous approach. In his work, it is concluded that the errors due to approximate method could be very significant both for horizontal and vertical ground motions. Ardakanian *et al.* (2006) developed an anisotropic damage mechanics approach to consider the nonlinear seismic response of concrete dams in 3D space when the reservoir is assumed compressible. Oliveira and Faria (2006) studied the failure scenarios of concrete dams. In their work, a Continuum Damage Mechanics model that incorporates two independent scalar damage variables in tension and compression was adopted in which both in tension and compression material softening is reproduced.

Zhu and Pekau (2007) investigated the seismic response of concrete gravity dams in which the major task is the treatment of dynamic contact conditions at the cracks penetrated within the dam body. They modeled all the modes of motion along the cracks using FE and adopting incremental displacement constraint equations. The hydrodynamic effect was modeled using the added mass approach. Bayraktar *et al*. (2008) carried out an investigation on the effects of near-fault ground motion on the nonlinear response of dams including dam-reservoir-foundation interaction. In their work, four different types of dams, which are gravity, arch, concrete faced rockfill and clay core rockfill dams, were considered and the behavior of reservoir was taken into account utilizing lagrangian approach.

In the present study, the damage mechanics model introduced in Ardakanian *et al.* (2006) is developed so that the cracking within an element is non-uniform, means that cracks in the candidate element propagate within its Gaussian points. The major efficiency of the proposed numerical model is the ability of evaluating nonlinear seismic behavior of concrete dams in 3D space using large elements and also, its ability of more accurate tracing crack paths within the dam body and finally, reducing time and saving analysis requirements. It is worth noting that the main aspect of the damage mechanics theory is defining the damage level of the cracked elements which can be used as an index for safety assessment of concrete dams. Performance based approach and its criteria in design of new dams and safety assessment of existing dams is one of innovative methods which quantify the performance of the system under various loading combinations (USACE 2007).

Various performance levels are considered when evaluating the response of concrete hydraulic structures such as concrete dams. The performance levels commonly used are serviceability performance, damage control performance, and collapse prevention performance.

In *the Serviceability performance level* the structure is expected to be serviceable and operational immediately following earthquakes producing ground motions up to the OBE[1] level (USACE 2007).

In *the damage control performance level,* certain elements of the structure can deform beyond their elastic limits (non-linear behavior) if non-linear displacement demands are low and load resistance is not diminished when the structure is subjected to extreme earthquake events. Damage may be significant, but it is generally concentrated in discrete locations where yielding and/or cracking occur. The designer should identify all potential damage regions, and be satisfied that the structure is capable of resisting static loads and if necessary can be repaired to stop further damage by non-earthquake loads. Except for unlikely MCE[2] events, it is desirable to prevent damage from occurring in substructure elements, such as piling and drilled piers, and other inaccessible structural elements (USACE 2007). Finally, *the collapse prevention performance level* requires that the structure not to collapse regardless of the level of damage. Damage may be un-repairable. Ductility demands can be greater than those associated with the damage control performance. If the structure does not collapse when subjected to extreme earthquake events, resistance can be expected to decrease with increasing displacements. Collapse prevention performance should only be permitted for unlikely MCE events. Collapse prevention analysis requires a Nonlinear Static Procedure or Nonlinear Dynamic Procedure in accordance with the related guidance (USACE 2007).

Based on the present study, obtaining the damage indices of the cracked Gaussian points using the damage mechanics approach can help the designer to identify the performance level of the structure using given quantitative criteria.

[1]Operating Basis Earthquake
[2]Maximum Credible Earthquake

2. Constitutive law

The stress-strain relationship within the pre-softening phase and also, the softening initiation criterion are given in (Ardakanian *et al.* 2006). It should be noted that the softening initiation criterion is based on the elastic uni-axial energy of the considered Gaussian point (Ardakanian *et al.* 2006).

During the softening phase, when a Gaussian Point initiates softening, its elastic stress-strain relationship is replaced using the modulus matrix which is formulated based on the damage level in each of the three principal directions. In this study, the Secant Modulus Stiffness (SMS) approach is used for the stiffness matrix formulation in which, the constitutive law is defined in terms of the total stresses and strains. Based on the energy equivalence principle and neglecting the coupling between the three principal fracture modes, the modulus matrix is given as (Gunn 2001a)

$$[D]_d = \begin{bmatrix} [D]_d^t & 0 \\ 0 & [D]_d^r \end{bmatrix} \tag{1}$$

where,

$$[D]_d^t = \frac{E}{(1+\upsilon)(1-2\upsilon)} \begin{bmatrix} (1-\upsilon)(1-d_1)^2 & \upsilon(1-d_1)(1-d_2) & \upsilon(1-d_1)(1-d_3) \\ \upsilon(1-d_1)(1-d_2) & (1-\upsilon)(1-d_2)^2 & \upsilon(1-d_2)(1-d_3) \\ \upsilon(1-d_1)(1-d_3) & \upsilon(1-d_2)(1-d_3) & (1-\upsilon)(1-d_3)^2 \end{bmatrix} \tag{2}$$

$$[D]_d^r = \begin{bmatrix} \gamma_{12}G & 0 & 0 \\ 0 & \gamma_{23}G & 0 \\ 0 & 0 & \gamma_{13}G \end{bmatrix} \tag{3}$$

in which,

$$\begin{aligned} \gamma_{12} &= \frac{2(1-d_1)^2(1-d_2)^2}{(1-d_1)^2+(1-d_2)^2} \\ \gamma_{23} &= \frac{2(1-d_2)^2(1-d_3)^2}{(1-d_2)^2+(1-d_3)^2} \\ \gamma_{13} &= \frac{2(1-d_1)^2(1-d_3)^2}{(1-d_1)^2+(1-d_3)^2} \end{aligned} \tag{4}$$

where d_1, d_2 and d_3 are the damage variables corresponding to the principal strains in the local directions. Satisfying the principle of energy equivalence and assuming linear stress-strain curve in the post-peak phase, d_i is given as

$$d_i = 1 - \sqrt{\frac{\varepsilon_0}{\varepsilon_i} - \left(\frac{\varepsilon_i - \varepsilon_0}{\varepsilon_f - \varepsilon_0}\right) \cdot \frac{\varepsilon_0}{\varepsilon_i}} \tag{5}$$

where ε_0 and ε_f are the stains corresponding to the crack initiation and no resistance strain, respectively and ε_i is the principal strain of the element in the considered direction. The proposed modulus matrix includes all of the principal fracture modes.

The proposed modulus matrix given in Eq. (1) is in local coordinate which is corresponding to the direction of the principal strains. This matrix should be transformed to the global coordinate as follows

$$[D]_S = [T]^T[D]_d[T] \tag{6}$$

where, $[T]$ is the strain transformation matrix. Clearly, increasing the strain of the considered Gaussian point leads to increasing the corresponding damage variable. Finally, when the strain reaches to the fracture strain, the Gaussian point is fully cracked in the corresponding direction and the related damage variable sets to be unit. In fact, any change in principal strain or its direction leads to update requirement of the global constitutive matrix, $[D]_S$. Satisfying the fracture energy conservation principle in the static and dynamic loading conditions, the no resistance strain is given as

$$\varepsilon_f = \frac{2G_f}{\sigma_0 h_c} \quad \text{and} \quad \varepsilon_f' = \frac{2G_f'}{\sigma_0' h_c} \tag{7}$$

where, h_c is the characteristic dimension of the cracked Gaussian point and is assumed equal to the third root of the Gaussian point's contribution volume within the cracked element. The primed quantities show the dynamic constitutive parameters. The strain-rate sensitivity of the specific fracture energy is taken into account through the dynamic magnification factor DMF_f as follows

$$G_f' = DMF_f G_f \tag{8}$$

As in Ardakanian *et al*. (2006), in the present study, the shear stiffness factors γ_{12}, γ_{23} and γ_{13} in Eq. (4) are determined based on the state of the Gaussian point in each principal direction in the current time step. As softening within the considered element progresses, the shear stiffness factor in the cracked Gaussian points decreases corresponding to the state of the principle strains and may reach to zero value. Therefore, the constitutive matrices contributions of the cracked Gaussian points and finally, the constitutive matrix of the considered element must be updated as these factors are changed.

In the proposed formulation, the Co-axial Rotating Crack Model (CRCM) is used to model the behavior of the cracked Gaussian points within the cracked elements and the crack opening and closing criterion is based on the principal strains. In addition, it has been shown that under cyclic loading, there is residual strain in the closed Gaussian point. This concept has been used at the *element level* (Ghrib and Tinawi 1995a, 1995b, Mirzabozorg 2004, Mirzabozorg *et al.* 2004, Mirzabozorg and Ghaemian 2005, Ardakanian *et al.* 2006) and also, in the current study, in which the total strain in *each Gaussian point* is decomposed into two components of the elastic and the residual strain given as

$$\varepsilon = \varepsilon^e + \varepsilon^{in} = \varepsilon^e + \lambda\varepsilon_{\max} \tag{9}$$

where, $\varepsilon_{\max}$ is the maximum principal strain which the Gaussian point has reached during the previous cycles and λ is the ratio between the residual strain in the closed Gaussian point and the maximum principal strain $\varepsilon_{\max}$ and is normally given as 0.2. Fig. 1 shows the algorithm for the

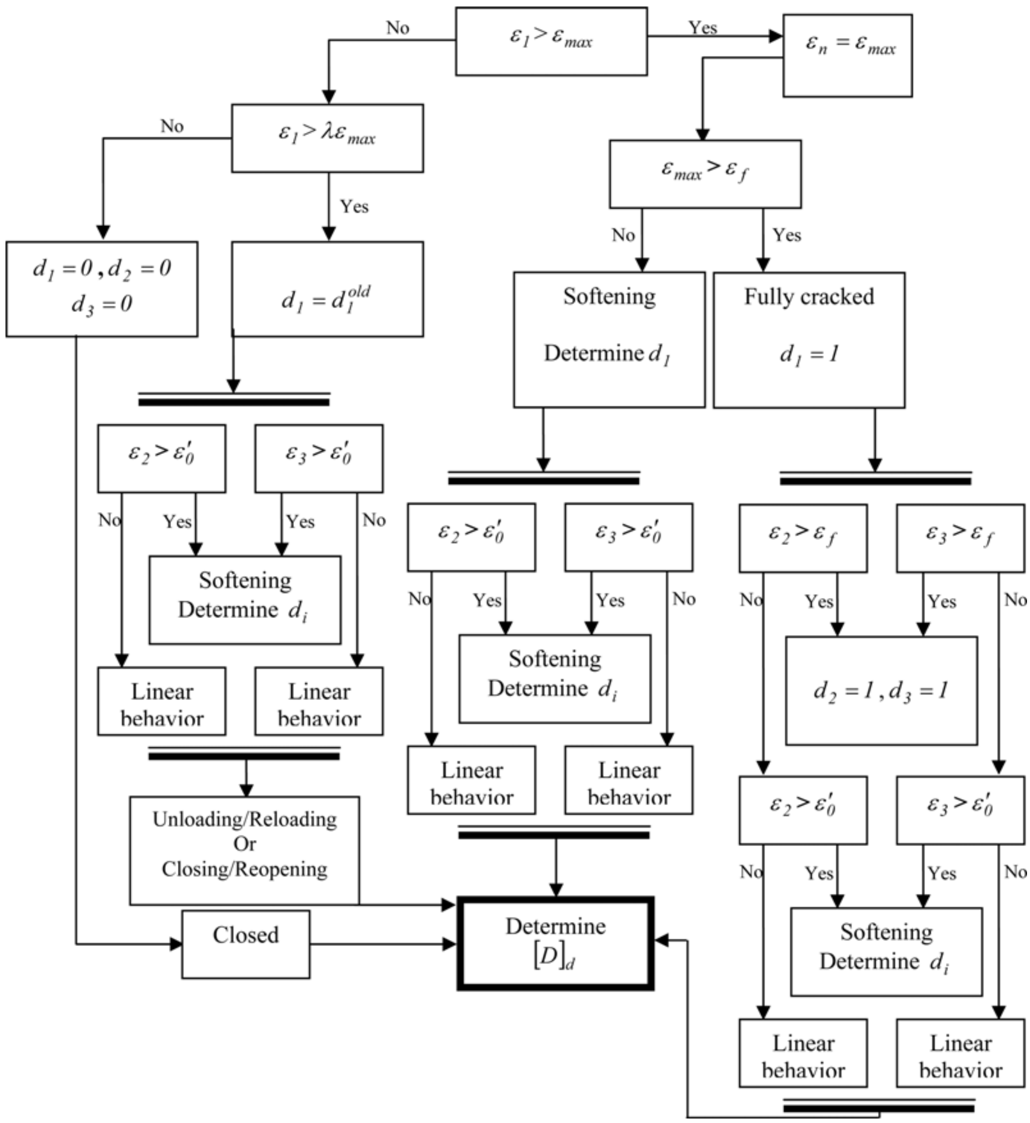


Fig. 1 State determination of the Gaussian Point, closing/reopening algorithm and fully crack state in the cracked Gaussian points

crack closing/reopening procedure which is utilized in the proposed numerical approach. In is worth noting that the shown algorithm in Fig. 1 has been used at the *element level* in reference (Mirzabozorg and Ghaemian 2005).

3. Validity and application in static and dynamic conditions

The 20-node iso-parametric brick finite element is utilized to model the structure, mathematically. This element is recommended in 3D fracture analysis of concrete dams under static and dynamic loading conditions. The requirement for integration and generation of the mass, stiffness and damping matrices for this type of element is 27 Gaussian points in 3*3*3 order within the element.

The main aspect of the proposed approach is in applying the cracking process on each Gaussian point within the element instead of cracking the element. This is based on the average of all the Gaussian point responses in which the energy dissipates within each cracked Gaussian point instead of the entire cracked element. The validity of the proposed model and numerical algorithms are considered using available experimental and theoretical results.

3.1 Direct displacement control using simple unit element

The utilized element is the 20-node iso-parametric simple element with unit dimension in each direction, shown in Fig. 2 (Mirzabozorg and Ghaemian 2005, Ardakanian *et al.* 2006, Mirzabozorg *et al.* 2007). In spite of simplicity, this example is able to show the validity of the proposed numerical model and the prepared finite element program. The modulus of elasticity, Poisson's ratio, the tensile strength and the specific fracture energy are assumed 20 GPa, 0, 2.0 MPa and 250 N/m, respectively.

The incremental displacement in steps of $1*10^{-5}$ is applied on the free face. The element cracks throughout 18 Gaussian points within its body on the two cracked parallel planes, which are parallel with the supporting face. The first cracked plane including nine Gaussian points is in vicinity of the loading face. The next cracked plane is in the middle plane within the element body.

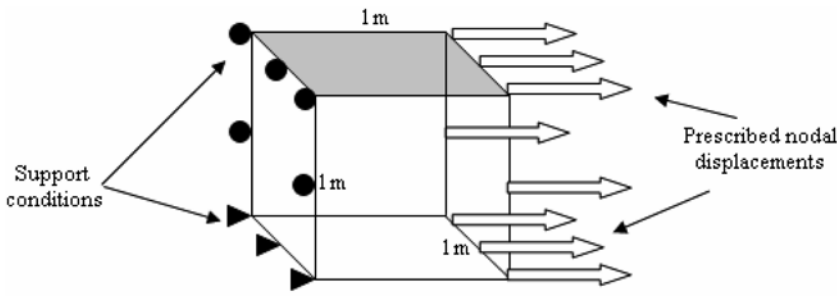


Fig. 2 Unit 20-node iso-parametric element, direct displacement on free face

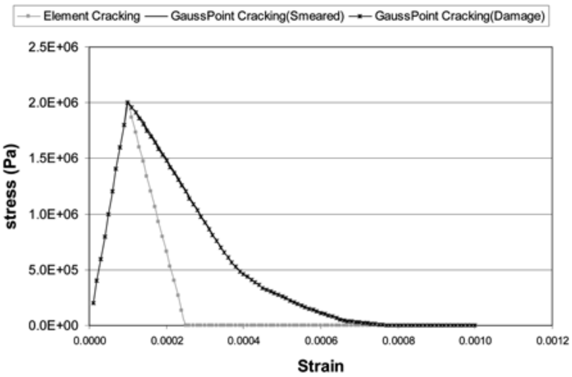


Fig. 3 Stress-strain curve resulted from Gaussian point cracking and element cracking level

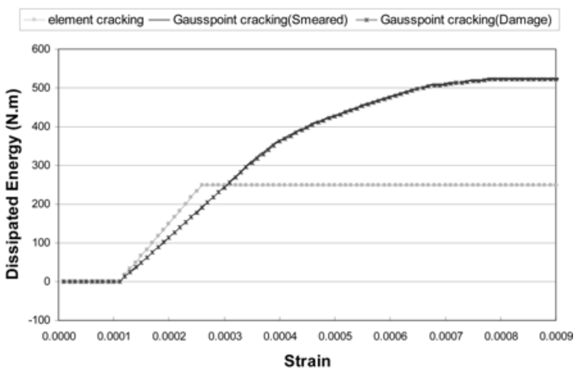


Fig. 4 Dissipated energy in the Gaussian point cracking and the element cracking level

The resulted stress-strain curve is shown in Fig. 3. The results obtained from the damage mechanics model coincide with those obtained from the smeared crack approach reported in Mirzabozorg *et al.* (2007).

Fig. 4 shows the energy dissipated within the element due to the fracturing process. The energy dissipation in each Gaussian point is initiated when the applied strain on the Gaussian point reaches 0.0001 which corresponds to the softening initiation. When the Gaussian Point is fully cracked, its total dissipated energy corresponds to its volume contribution within the element. It is found that in the conducted analysis, the dissipated energy is 520.67 N/m, which is a little more than 2 times of the corresponding value resulted from the element level cracking model. Obviously, this difference is due to cracking in the two fractured plane in the proposed model. It is worth noting that the results are the same as that obtained using the smeared crack approach (Mirzabozorg *et al.* 2007).

Similar to the results obtained using the smeared crack approach in Mirzabozorg *et al.* (2007), cracks cannot be localized at the Gaussian points within the element body, and their use makes sense only if the element size is equal to the material characteristic length. In fact, quadratic elements allow localization into a region smaller than the element. This can be shown analytically for a one-dimensional problem; in an element with quadratic displacement interpolation and three Gaussian points, strain is localized into two Gaussian points only and the dissipation is not captured correctly (Bazant and Planas 1998). This phenomenon can be seen in the conducted analysis in which there are just two parallel and adjacent fractured planes within the element body. A suggestion to solve the problem is to use the non-local or the crack band model (Bazant and Planas 1998).

It must be noted that the proposed numerical approach is also verified using the indirect displacement control algorithm and the resulted crack profiles within the body of the considered model is the same as that reported in Mirzabozorg *et al.* (2007).

3.2 Seismic analysis of koyna dam

Koyna dam in India is a classic example which has been used by several investigators as a benchmark for seismic analysis of gravity dams (Ghrib and Tinawi 1995a, 1995b, Mirzabozorg *et al.* 2004, Ardakanian *et al.* 2006). This structure is a concrete gravity dam which was designed based on the no-tension concept. The design seismic coefficient was 0.05 and the resulted seismic force was distributed uniformly over the height of the dam. The dam experienced an earthquake in 1967 with the magnitude of 6.5 on Richter scale which caused serious damages in various blocks. The tallest block experienced a crack at the upper part which passed through the entire thickness. Geometric properties of the tallest block and the position of the crack initiation on the downstream face of the dam can be found in Mirzabozorg *et al.* (2007).

The 3D unit-thickness finite element model of the tallest block includes 520 20-node iso-parametric elements and 3858 nodes. The upstream face is assumed vertical which has a negligible difference with the actual dam body. The modulus of elasticity, Poisson's ratio, the unit weight, the tensile strength and the specific fracture energy are taken as 31.027 GPa, 0.2, 25.920 KN/m^3, 1.5 MPa and 150 N/m, respectively. The dynamic magnification factor applied on the tensile strength and the specific fracture energy is 1.2. The two components of Koyna earthquake in 1967 are used to excite the system in the upstream-down stream and the vertical directions. These components are shown in Fig. 5.

The direct integration method presented in Mirzabozorg *et al.* (2003) is used to solve the problem

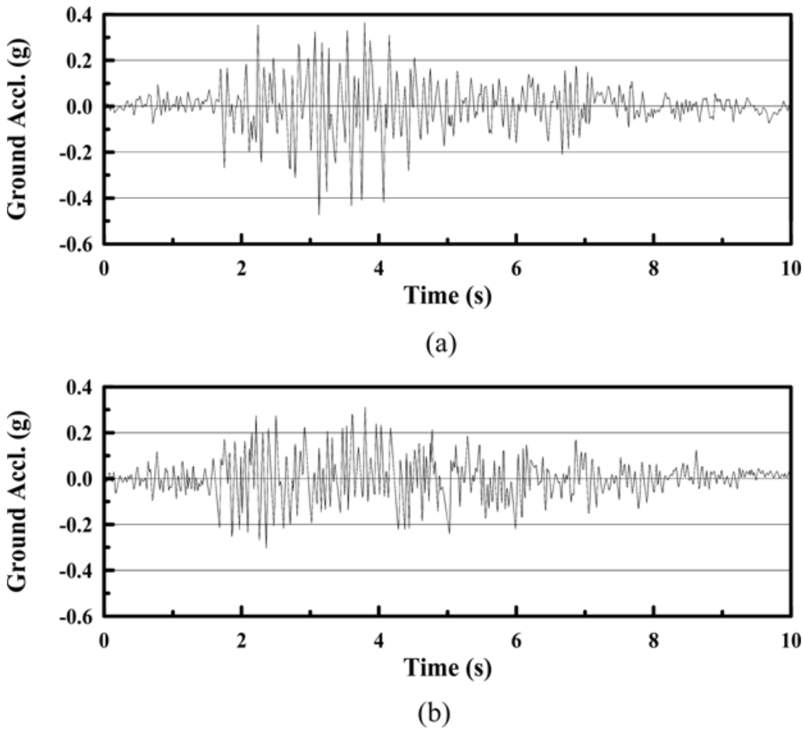


Fig. 5 Ground motion recorded at Koyna dam, Koyna earthquake 1967; (a) Stream component, (b) Vertical component

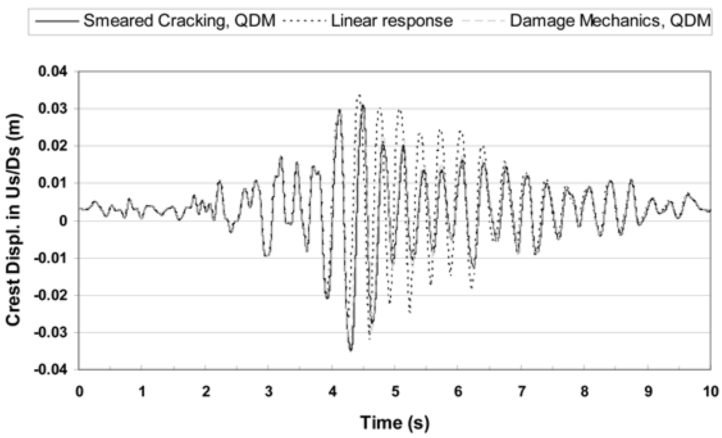


Fig. 6 Crest displacement time history of Koyna dam due to the stream and vertical excitations of Koyna earthquake record

under dynamic excitations. The integration parameters α, β and γ are assumed −0.2, 0.36 and 0.7, respectively. The integration time step is 0.001s. It must be noted that the time step of Koyna earthquake record is 0.01s. At the first step, the self weight and the hydrostatic loads are applied on the model. There is not any cracked element at the end of this stage. In the second step, dynamic analysis is conducted. The Quasi Linear Damping mechanism (QDM) is used in dynamic equations

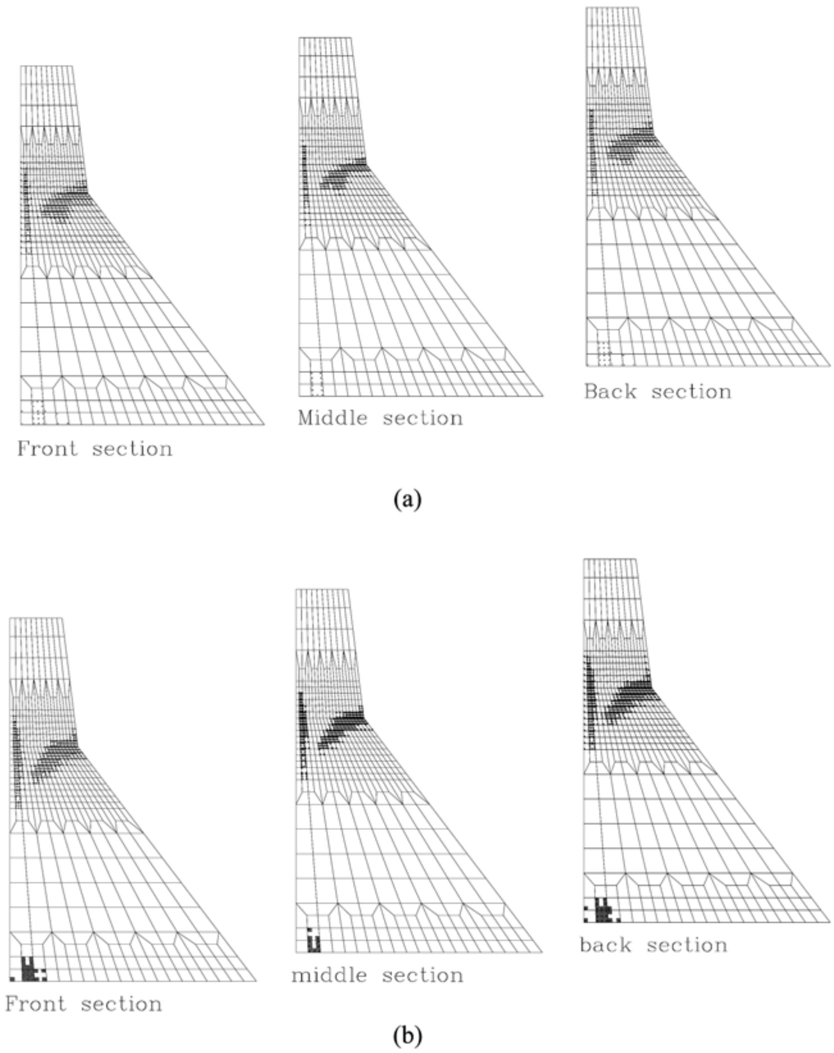


Fig. 7 Crack profiles within the dam body; (a) Damage mechanics approach, (b) Smeared crack approach (Mirzabozorg *et al.* 2007)

in which the stiffness proportional damping matrix of the element is computed based on the current stiffness matrix of the structure.

In Fig. 6, the time history of the crest displacement in the upstream-downstream direction is compared with that resulted from the smeared crack approach (Mirzabozorg *et al.* 2007) and the linear response of the system. No numerical instability was noted during the conducted dynamic analysis. The resulted crack profiles within the three parallel planes through the thickness of the model are shown in Fig. 7. There is excellent agreement between the resulted crack profiles with that obtained from the experimental work and the other available reported theoretical results (Mirzabozorg *et al.* 2007).

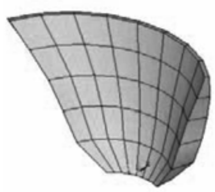
As discussed in USACE 2007, the acceptance criteria for linear-elastic time-history analysis of gravity dams is based on Demand-Capacity Ratios (DCRs) and cumulative inelastic duration. DCR for plain concrete structures are computed as the ratio of stress demands to static tensile strength of the concrete. A systematic interpretation and evaluation of the results of time history analysis in terms of the demand-capacity ratios, cumulative inelastic duration, spatial extent of overstressed regions, and consideration of possible modes of failure form the basis for estimation of probable level of damage or acceptable level of nonlinear response. The dam response to the MDE[1] is considered to be within the linear-elastic range of behavior with little or no possibility of damage if the computed stress demand-capacity ratios are less than or equal to 1.0. The dam would exhibit nonlinear response in the form of cracking of the concrete and/or opening of construction joints if the estimated stress demand-capacity ratios exceed 1.0. The level of nonlinear response or cracking is considered acceptable if demand-capacity ratios are less than 2.0 and limited to 15 percent of the dam cross-sectional surface area, and the cumulative duration of stress excursions beyond the tensile strength of the concrete falls below the performance curve given in USACE (2007).

Generally, a nonlinear time-history analysis might be required to estimate the damage more accurately. Using the proposed numerical approach, the damage index for each cracked Gaussian point is obtained and finally, the damage index of the dam body can be extracted more accurately than a linear analysis and therefore, judgment about the safety of the system is more precise.

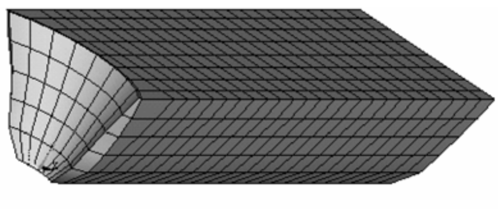
3.3 Nonlinear seismic analysis of morrow point arch dam

The Morrow Point dam is used to show the applicability of the proposed approach in modeling the nonlinear dynamic behavior of arch dams. The considered dam was constructed on Gunnison River in Colorado in a U-shape valley during 1963 to 1968. The height of the dam, the radius curvature at the crest level and the crest length of the dam are 145.74 m, 114.3 m and 220.67 m, respectively. The thickness of the central block is 3.66 m at the crest level and 15.85 m at the foundation level. Fig. 8 shows the considered system which includes the finite element model of the dam body and the reservoir with the length of about five times of the height of the dam. It is worth noting that the considered model has been used in Mirzabozorg and Ghaemian (2005), Ardakanian *et al.* (2006), Mirzabozorg *at al.* (2007).

The dam body is modeled using 40 20-node iso-parametric solid elements and the reservoir model includes 1000 8-node fluid elements. The modulus of elasticity, Poisson's ratio, the unit weight, the true tensile strength and the ratio of the apparent to the true tensile strength, the specific fracture energy and the dynamic magnification factor are 27.604 MPa, 0.2, 24027.15 N/m^3, 2.5 MPa, 1.25,



(a)



(b)

Fig. 8 FEM of the dam-reservoir system with rigid foundation; (a) Dam body, (b) Coupled dam-reservoir system

[1]Maximum Design Earthquake

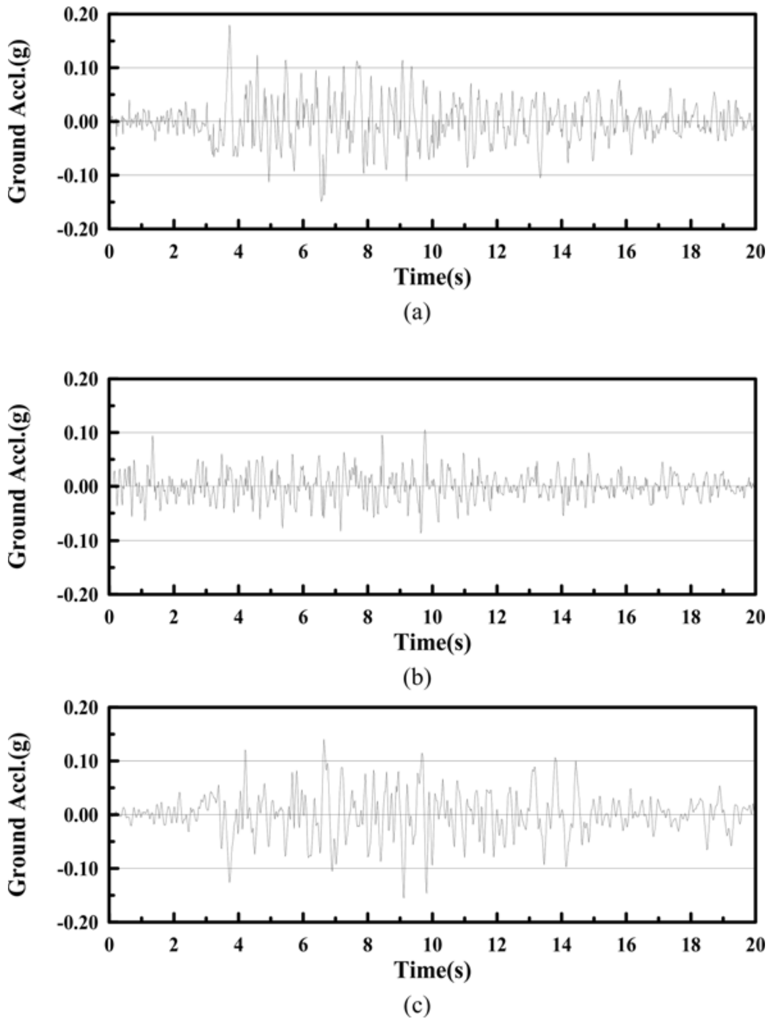


Fig. 9 Ground motion recorded at Taft Lincoln School Tunnel, California earthquake 1952; (a) Stream component, (b) Cross stream component, and (c) Vertical component

200 N/m and 1.30, respectively. The pressure wave propagation speed within the reservoir and the unit weight of the water are 1436 m/s and 9807 N/m^3, respectively. The wave reflection coefficient is taken as a conservational value of 0.8. The system is excited using the three components of the Taft earthquake in 21 July 1952 recorded at the Lincoln Tunnel School, shown in Fig. 9.

Applied loads on the system are the self weight, the hydrostatic pressure and the seismic load. The system is analyzed using the staggered displacement method (Mirzabozorg *et al*. 2003). The direct integration parameters α, β and γ are assumed −0.2, 0.36 and 0.7, respectively. The time integration step is 0.001s and the quasi linear damping mechanism is used to model the energy dissipation due to the damping.

At the first step, the dam body cracked at the heel due to the self-weight and the hydrostatic load as shown in Fig. 10. The initial crack profile, resulted from applying the static loads, propagates on the upstream face when the system excited using the three components of the earthquake record. At

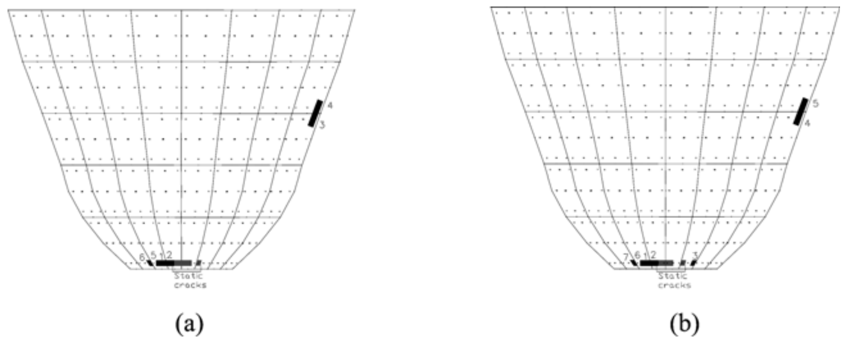


Fig. 10 Cracked elements on the upstream face of dam body including cracking sequence, (a) Damage mechanics approach, (b) Smeared crack approach (Mirzabozorg *et al.* 2007)

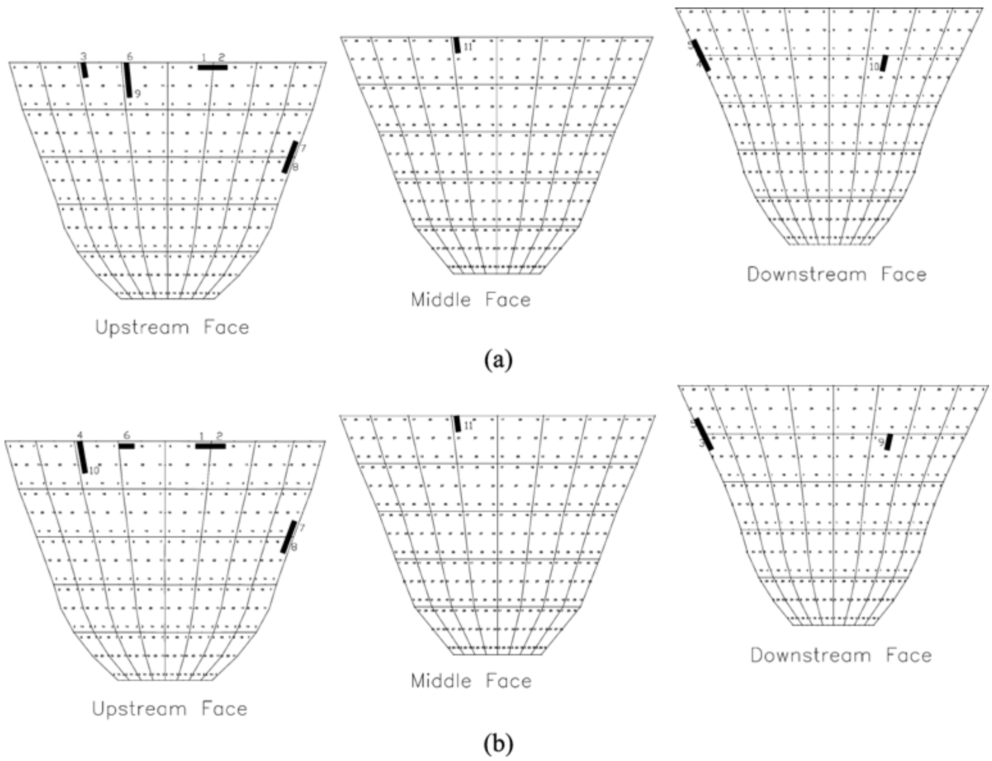
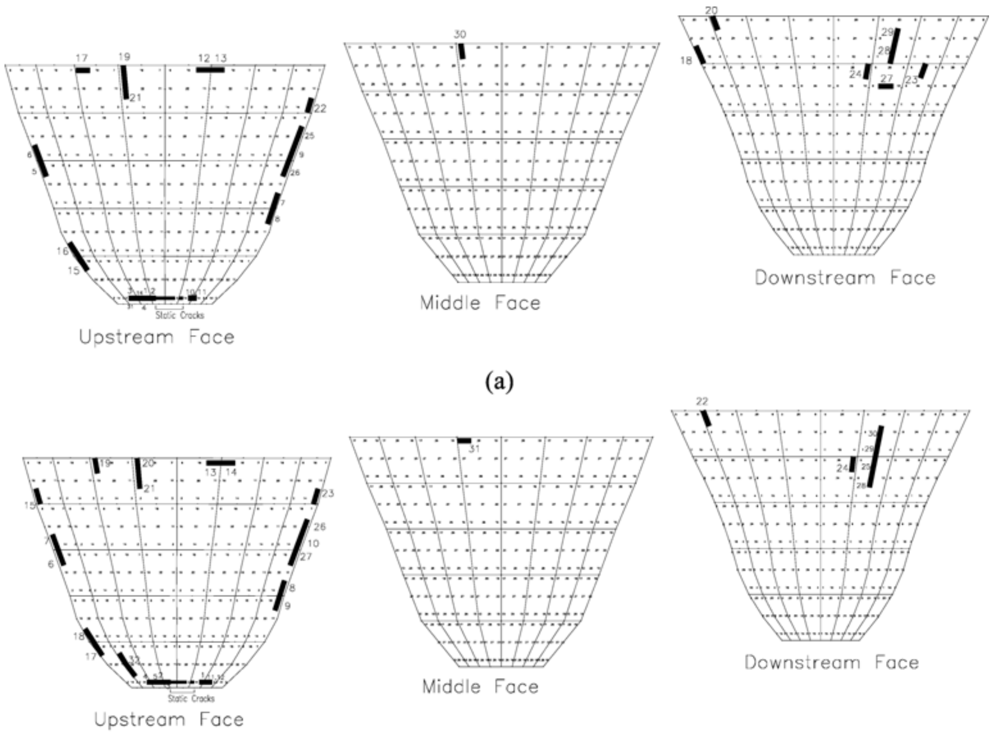


Fig. 11 Cracked Gaussian points within the dam body due to only the seismic loads including cracking sequence, (a) Damage mechanics approach, (b) Smeared crack approach (Mirzabozorg *et al.* 2007)

the end of the analysis, there are just two positions on the upstream face in which cracks propagate. Other sections through the thickness of the dam body experience no cracks. Clearly, because of few cracked Gaussian points within the dam body, the cracked elements do not affect the crest response of the dam body and the crest displacement time history is the same as that resulted from the linear analysis. In addition, as shown in Fig. 10, there are just negligible differences between the results obtained from the damage mechanics approach and the smeared crack approach proposed by the authors (Mirzabozorg *et al.* 2007).



(b)

Fig. 12 Cracked Gaussian points within the dam body including cracking sequence, due to applying the static and the scaled seismic loads by 1.7, (a) Damage mechanics approach, (b) Smeared crack approach (Mirzabozorg *et al.* 2007)

Fig. 11(a) shows the resulted crack profiles at the three layers of Gaussian points through the thickness of the dam body when the system is analyzed just under the seismic load (excluding any static loads). This analysis is conducted to consider cracking due to only dynamic loads and in addition, to consider the effect of static loads on the cracking pattern within the dam body. Comparing the crack profiles shown in Figs. 10 and 11, all the cracked Gaussian points at the higher levels of the dam body and the cracked points on both the middle plane and the downstream plane of the dam body are eliminated when the static loads are applied because of the stress distributed within the dam body resulting from the arch action of the structure. This phenomenon is common in arch dams. In Fig. 11(b), the results are compared with the crack profiles reported in Mirzabozorg *et al*. (2007) and there is excellent agreement between the obtained crack profiles using both the damage mechanics and the smeared crack approaches.

In the last step, the system is excited using the earthquake components scaled by a facor of 1.7 after applying the static loads, to compare with the same analysis reported in Mirzabozorg *et al.* (2007). Fig. 12(a) shows the resulted crack profiles in detail within the dam body and Fig. 12(b) shows the crack profiles obtained from the smeared crack approach. As shown, the resulted crack profiles for both models are in excellent agreement and both of them are in good agreement with the common seismic behavior of arch dams.

Similar to the gravity dams, there are acceptable criteria for safety evaluation of arch dams which are based on the Demand-Capacity Ratios (DCRs) and cumulative inelastic duration within the dam body (USACE 2007). Using the damage mechanics approach, the overstresses regions can be estimated more accurately and therefore, judgment about the safety of the considered system can be more realistic.

4. Conclusions

A 3D damage mechanics model was developed which is able to simulate cracking at the Gaussian point level. The validity of the proposed model due to the static and dynamic loading conditions were considered using available experimental and numerical results and also, using the smeared crack model proposed by the authors, in Mirzabozorg *et al*. (2007). It was found that the proposed model gives reasonable results using the direct and indirect displacement algorithms in the static conditions and the results are reasonable in comparison with the available data. In addition, dynamic analysis of Koyna dam using the proposed method shows that the pertinent numerical algorithms and the proposed model is stable in the dynamic conditions and gives excellent results compared with the experimental and the other numerical results. Finally, the Morrow Point dam was analyzed under static and dynamic loads. The conducted analysis includes the dam-reservoir interaction. It was found that using the proposed method, the resulted profiles within the dam body can be studied in detail and the crack propagation can be traced within the three layers of each element. Clearly, tracing cracks through the thickness of the dam body in arch dams is a major step in terms of both the dam safety evaluation and the dam design stage. It is worth noting that the results obtained form the proposed approach are similar to those obtained using the smeared crack approach introduced by the authors (Mirzabozorg *et al.* 2007).

The main aspect of the proposed method is the ability of tracing crack propagation within the mass concrete using coarse meshing of the finite element model. In addition, the stability of the proposed method is excellent because of gradual change in the stiffness matrix of the finite element model due to Gaussian point cracking instead of element cracking. The other aspect of the proposed damage mechanics approach is its ability to identify a damage index to the cracked dam body which is a significant factor in dam safety assessment based on the safety criteria.

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