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Comparative study of endurance time and time history methods in seismic analysis of high arch dams

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Abstract

In the present study, the application of Endurance Time Analysis (ETA) method is investigated on seismic analysis of a high arch dam. In this method the coupled system is excited using the predesigned intensifying acceleration functions instead of the real ground motions. Finite element model of an arch dam considering the dam-rock-water interaction effects was developed in which the concrete and rock were assumed to have linear elastic behavior. The effect of the large displacement in dam body was considered in numerical model using co-rotational approach. The coupled system was analyzed by conventional Time History Analysis (THA) method in various seismic performance levels and the results were compared with those obtained from ETA at the equivalent target time. It was found that ETA method provides the close results to THA with acceptable accuracy while it reduces the total time of the analyses considerably.

Keywords: *Endurance time analysis, Dam-rock-water interaction, Equivalent target time, Geometric nonlinearity.*

1. Introduction

Over the last few years, developing innovative methods for analyzing the structural systems and predicting the responses spending minimum time have been interested by structural engineers. Nonlinear static analysis or pushover analysis (POA) is one of these methods in which the seismic demand is computed by nonlinear static analyses of the structure subjected to monotonically increasing lateral forces until a target value of specific point displacement is reached [1]. Incremental Dynamic Analysis (IDA) is another approach in which the seismic load is scaled in different performance levels and several nonlinear dynamic analyses are performed to estimate dynamic performance of the structural system [2]. In this method Engineering Demand Parameter (EDP) is obtained at various Intensity Measures (IMs) and the performance of the structure is monitored more precisely. The main difficulty in IDA method is requisiteness for large number of nonlinear dynamic analyses which makes it practically impossible to use in the case of complicated structures.

Endurance Time Analysis (ETA) method is basically a simple dynamic pushover procedure that tries to estimate EDPs at different IM levels by exciting the structural system using a predesigned intensifying acceleration function [3].

Artificial excitations, in ETA method, are called acceleration function instead of accelerogram. Endurance Time Acceleration Functions (ETAFs) are designed in a way that their intensity increases through time [4]. Because of increasing demand of ETAF, structural system gradually goes from linear elastic to nonlinear phase, finally leading to global dynamic instability.

Application of ETA method in linear seismic analysis of structures has been studied before [5]. Ability of ETA method in nonlinear analysis of moment resisting and concentrically braced steel frames was investigated as well [6 and 7]. Valamanesh and Estekanchi [8] studied multi-component analysis of the frames using ETA method. Alembagheri and Estekanchi [9] studied seismic analysis of aboveground steel storage tanks using ETA method. They considered surface sloshing in numerical models and found that ETA is capable to estimate conventional nonlinear response history with reasonably good accuracy. Tavazo et al. [10] used ETA method for linear seismic analysis of several cases of shell structures and compared results with those obtained from THA and response spectrum analysis. Hariri-Ardebili and Mirzabozorg [11] compared the results of ETA method in elastic linear analysis of concrete arch dam with conventional THA method. They found that ETA is capable to estimate various responses of arch dams in low to high excitation levels with good accuracy. Hariri-Ardebili *et al.* [12, 13] studied the ability of ETA method in nonlinear seismic analysis and assessment of concrete arch dams considering

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the nonlinear behavior of mass concrete and the contraction joints effects. They found that there is good agreement between the general pattern of crack profile and also the maximum joint opening/sliding resulted from ETA and nonlinear THA methods.

In this paper, we investigate the ability of ETA method for seismic assessment of concrete arch dams and we consider the large deformation of the coupled system under the intensive dynamic excitations. Conventional time history analysis method using different site-specific ground motions is also implemented on the structural system for comparison purposes. A high double curvature arch dam is used as case study and the water-dam interaction effect is considered by Eulerian-Lagrangian approach. Flexibility of the rock is taken into account by modeling it as a massless medium. The material constitutive model of the both mass concrete and the foundation rock is assumed to be linear elastic, while the geometric nonlinearity effect of the dam body is modeled using co-rotational formulation. Finally the finite element model of the coupled system is excited at different seismic performance levels using real ground motions and ETAFs.

2. Concept of Endurance Time Analysis

ETA method is a dynamic pushover procedure to estimate seismic performance of the structures by analyzing their resilience when subjected to intensifying dynamic excitations [3]. Structural responses, such as displacements, accelerations, stresses or other EDPs are monitored up to the point where the structure collapses or the analysis does not converge. Time duration from start of the test or analysis to this collapse point is called "*Endurance Time*" [14]. Basically, longer the structure can endure imposed excitations, it is judged to have better performance.

Concept of ETA analysis can be described using hypothetical shaking table experiment on simple frames [5]. Three different structures with unknown structural properties are to be ranked according to their seismic resistance performance. All three structures are fixed on a shaking table and the test begins by subjecting the structures to an ETAF, as shown in Fig. 1. According to lapse of time, the amplitude of vibrations increases in the shaking table.

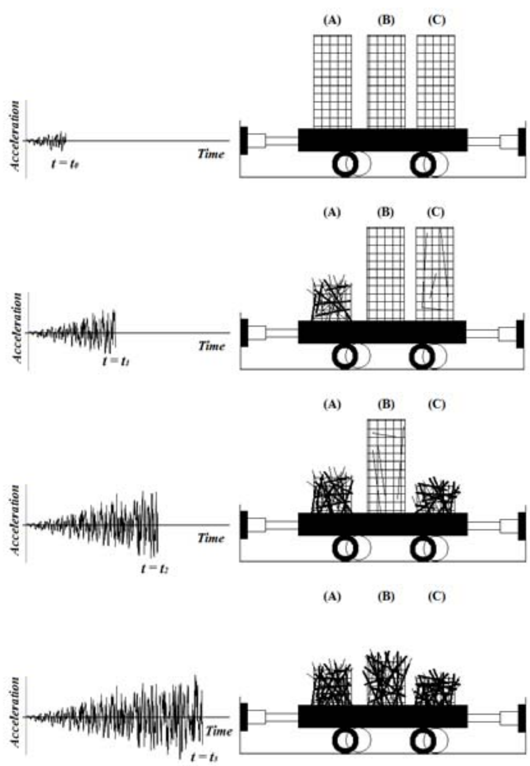


Fig. 1 Schematic of hypothetical shaking table experiment on frames

As the vibration amplitude increases, at $t = t_1$ one of the structures fails as shown in Fig. 1. Continuously, the amplitude of vibration is increased and at $t = t_2$ and $t = t_3$ the second and the third structures also fail. Based on these results and considering that lateral loads induced by shaking table somehow correspond with earthquake loads, structure "*A*", which failed earliest, is ranked as the worst and structure "*B*", which endured longest, is ranked as the best one. This hypothetical experiment describes the essence of ETA method.

Numerically, after modeling the structural system and specifying the suitable EDPs, ETA can be implement using appropriate ETAFs and the time history of selected EDP is recorded during analysis. Maximum absolute value of EDP is plotted for total duration of analysis. Then, Endurance Time Curve (ETC) is plotted for each desired EDP. In theory, ETC represents a special diagram that its vertical axe's values refer to the maximum absolute values of EDP during the time interval from 0.0 to t based on Eq. (1).

$$\Omega\left(EDP(t)\right) \equiv Max\left(Abs\left(EDP(\tau)\ :\ \tau \in \left[0,t\right]\right)\right) \qquad (1)$$

in which, Ω acts as maximum absolute operator and $EDP(t)$ represents time-history of the considered response.

There are three key factors for successful implementation of the procedure. The most important one is generation of an appropriate ETAF so that the results from ETA can be correlated reliably well with response of the structure subjected to real ground motions. For this purpose, concept of response spectra is utilized in ETA method [5]. The second factor is calculation of the appropriate equivalent target time for various seismic performance levels. The third factor is preparing a suitable numerical model or set up a reliable experimental test.

3. Finite Element Model and Formulation

Dez is 203m high double curvature arch dam which is located in a narrow gorge at Dez River, in Khuzestan Province in Iran, about 150km upstream of provincial capital of Ahwaz. Crest length is 240m and thickness at the crest level is 4.5m. Finite element idealization prepared for the dam and foundation rock are depicted in Fig. 2, which consists of 792 solid elements for modeling dam and concrete saddle and 3770 solid elements for simulation of rock. The eight-node solid elements have three transitional degrees of freedoms (DOFs) at each node. Water is modeled using 3660 fluid elements (Fig. 2). Utilized Eulerian fluid elements have three transitional DOFs and one pressure DOF at each node. It should be noted that transitional DOFs of fluid elements are active only on interface of solid elements. Also all nodes on far end boundary of the foundation rock are restricted in three transitional directions.

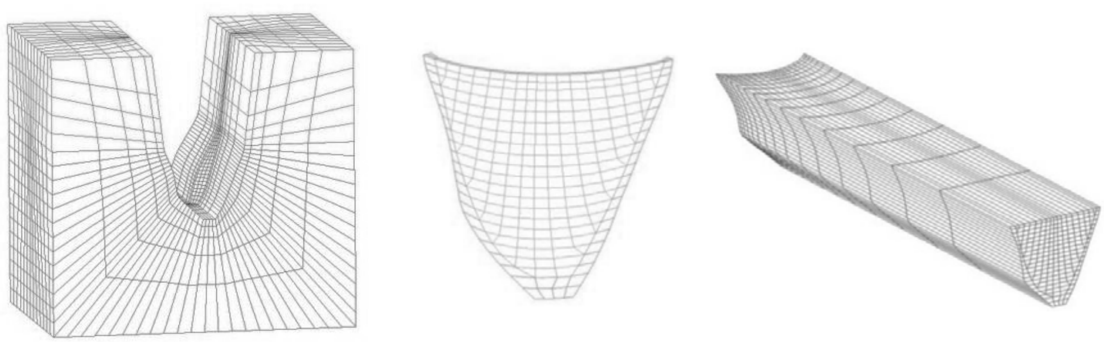


Fig. 2 Finite element model of dam, foundation rock and reservoir water

Isotropic elasticity for mass concrete in static and dynamic condition is 40GPa and 46GPa respectively and their corresponding Poisson's ratios are 0.2 and 0.14. In addition, density of mass concrete is taken as 2400kg/m^3. Deformation modulus of soil in saturated and dry conditions is 13GPa and 15GPa, respectively [15]. Reservoir water density is assumed 1000kg/m^3, sound velocity is 1440m/s in water and wave reflection coefficient for reservoir around boundary is supposed 0.8, conservatively. It is noteworthy that all properties for material were obtained from instrumentation and calibration of finite element model with geodesy in static and thermal conditions [15].

It is usually assumed that due to boundary condition of the dam-abutment, there are no large displacement and strain in the body. So, the geometry of the dam is assumed to remains unchanged during the static and even dynamic loading process and the linear strain approximation can be used [16]. It should be noticed that analysis of the coupled system subjected to very intense dynamic loading increase deformability of the dam and so considering the large-displacement/small-strain effects in the finite element formulation seems to be important in this condition.

There are different versions of the kinematic modeling of geometrically nonlinear problems such as Eulerian approach, Lagrangian approach, and arbitrary Eulerian-Lagrangian approach. Three Lagrangian kinematic descriptions which are used for finite element analysis of geometrically nonlinear structures are; total Lagrangian, updated Lagrangian and co-rotational formulation [17]. In recent years the use of co-rotational formulation has increased because it provides the simple solution for Lagrangian formulations (large-displacement and small-strain problems) without significant loss of accuracy [18]. In this formulation, rigid-body motion is eliminated and only element deformation is considered to obtain internal forces and the tangent stiffness matrix [19]. In fact the co-rotational nonlinearity is contained in strain-displacement relationship and changes the usual kinematic equations to advanced form. Fig. 3 summarizes the general procedure in order to implementation of the nonlinear geometric effects in finite element formulation. In this formulation small strain-displacement relationship in the original element coordinate system, $\left[B_v\right]$, is related to the one in the rotated element coordinate system, $\left[B_v\right]$, using the orthogonal (undated) transformation matrix, $\left[T_n\right]$.

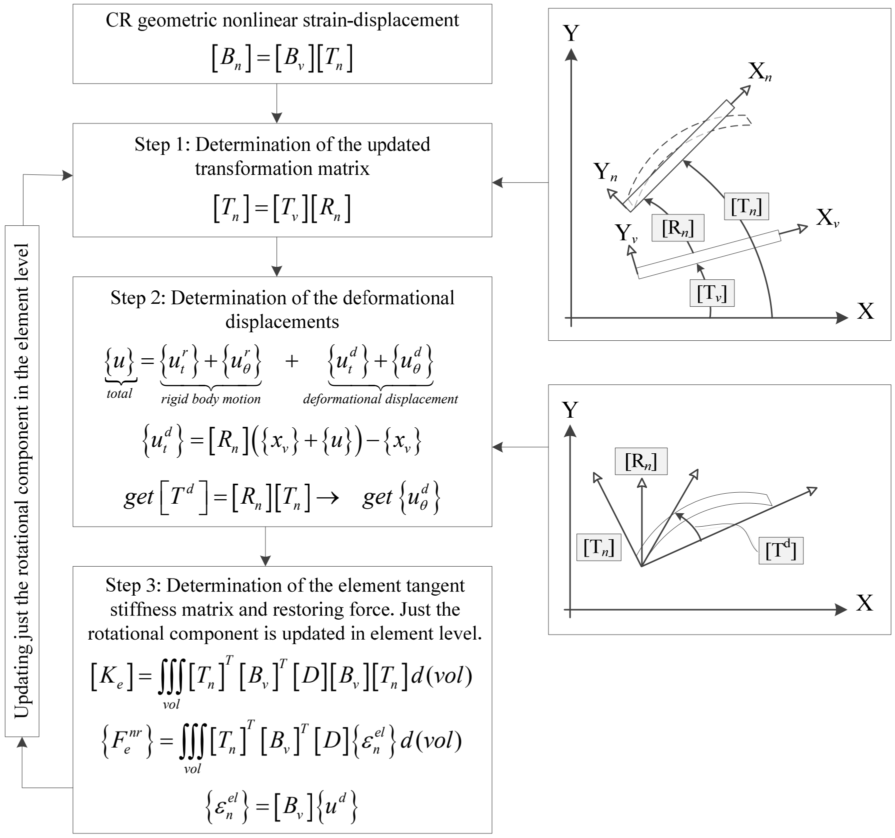


Fig. 3 Implementation of the geometric nonlinearity in the finite element analysis using co-rotational formulation

The undated transformation matrix is calculated using the original transformation matrix, $[T_v]$, and the rotation matrix, $[R_n]$. Total displacement vector of the element in global coordinates, $\{u\}$, is then calculated using summation on translational displacement of the rigid body motion, $\{u_t^r\}$, rotational displacement of the rigid body motion, $\{u_\theta^r\}$, translational deformational displacement , $\{u_t^d\}$, and rotational deformational displacement , $\{u_\theta^d\}$.

Finally the element tangent stiffness matrix, $[K_e]$, and the restoring force , $\{F_e^{nr}\}$, are calculated in which , $\{\varepsilon_n^{el}\}$, is the elastic strain vector. It should be mentioned that this is a loop over the entire load steps of the analysis where only the rotational component is updated in element level and the translational component is kept unchanged.

Finally the coupled equations of the dam-foundation (as the structure) and the reservoir take the form:

$$\begin{cases}[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{f_1\}-[M]\{\ddot{U}_g\}+[Q]\{P\}=\{F_1\}+[Q]\{P\}\\ [G]\{\ddot{P}\}+[C']\{\dot{P}\}+[K']\{P\}=\{F\}-\rho[Q]^T\left(\{\ddot{U}\}+\{\ddot{U}_g\}\right)=\{F_2\}-\rho[Q]^T\{\ddot{U}\}\end{cases} \tag{2}$$

where [M], [C] and [K] are the mass, damping and stiffness matrices of the structure including the dam body and its foundation media and [G], [C′] and [K′] are matrices representing the mass, damping and stiffness equivalent matrices of the water, respectively. The matrix [Q] is the coupling matrix; $\{f_1\}$ is the vector including both the body and the hydrostatic force; {P} and {U} are the vectors of hydrodynamic pressures and displacements, respectively and $\{\ddot{U}_g\}$ is the ground acceleration vector. A detailed definition of matrices and vectors used in Eq. (2) has been provided in Mirzabozorg et al. [20]. The coupled equations are solved using the staggered displacement method in which the direct integration scheme is used to determine the displacement and hydrodynamic pressure.

4. Loading the Coupled System

Applied loads on the system are dam body self-weight, hydrostatic pressure in Normal Water Level (NWL) and seismic load. In addition, thermal load corresponding to summer condition is applied on the dam body. It is worthy to note that thermal load applied on the structure has been extracted from calibrated thermal transient analyses conducted using real data at the dam site taking into account the solar radiation on the exposed surfaces of the dam body [15].

The system is excited at foundation boundaries using ETAFs and scaled ground motion records and Newmark-β method is utilized to solve the coupled dam-rock-water model. Moreover based on engineering judgment, structural damping is taken to be 5% of critical damping in all excitation levels. It should be mentioned that all seismic inputs are applied to the structural system in only one major direction which is the Upstream/Downstream (US/DS) direction. It is obvious that seismic analysis of arch dams should be utilized considering appropriate three-component ground motion records, but in present paper, ETA and THA methods were compared in seismic assessment of arch dams considering only one major direction of seismic input which is US/DS direction. Although using one-component ground motion instead of three-component one reduces results obtained from analysis of system, it has no effect in general methodology. In multi-component analysis of structures based on ETA method it is just needed that ETAFs be scaled based on horizontal and vertical design response spectrums and suitable reduction factor be used for second horizontal component as described in [8].

4.1. Characteristics of ETAFs

As mentioned before, an important issue in successful implementation of ETA method is the generation of appropriate ETAFs. There are different generations for ETAFs with different lengths and characteristics produced by Estekanchi *et al.* [14, 21]. A summary of first and second generations of acceleration functions are presented in Fig. 4. As far as ETAFs be able to satisfy the characteristics of real ground motions, they can provide appropriate responses of structural systems under simulated dynamic excitations.

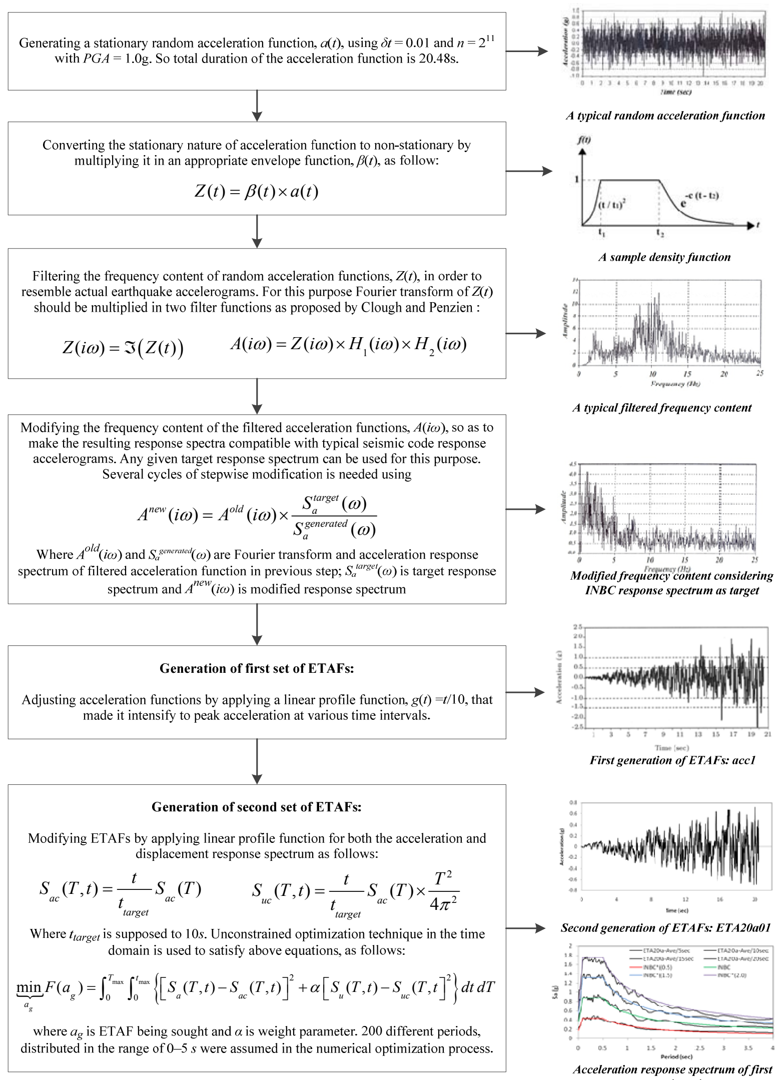


Fig. 4 Summary of the procedure for generation of the first and second generation of ETAFs

To investigate potential of ETA method in comparison with THA, a set of ETAFs called as ETA20e01-03 was used in the present paper. This set of ETAFs which also called *e*-series, were generated using the average response spectrum of real ground motions. To reach this goal, 20 accelerograms which were defined by the NEHRP and used in FEMA-440, were selected as base ground motions [14]. From these ground motions, seven records whose response spectra shapes were more compatible with the response spectrum of soil type *II* of INBC (Iranian National Building Code) were selected as reported by Estekanchi *et al.* [14]. These seven accelerograms were scaled to produce a response spectrum that is compatible with the INBC spectrum. Finally, the average of the pseudo acceleration spectrum of these scaled accelerograms is obtained and smoothed. The smoothed spectrum was used as the base target spectrum in generating this set of ETAFs. These ETAFs are generated in such a way that their response spectra increase by the time, hence response of the structure under this kind of accelerograms gradually increases with time.

It is important to note that ETAF's response spectrum remains proportional to the target spectrum in any lapse of time. For example response spectrum at time t=10s (which is considered as base target time) is twice of the response spectrum at time t=5s and half of it at time t=20s. Fig. 5 shows average response spectrum extracted from ETA20e01-03 at various times. Also Fig. 6 represents ETA20e01-03 acceleration functions, velocity functions and displacement functions time-histories.

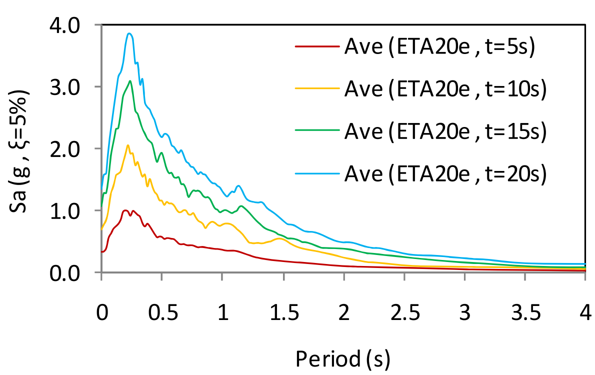
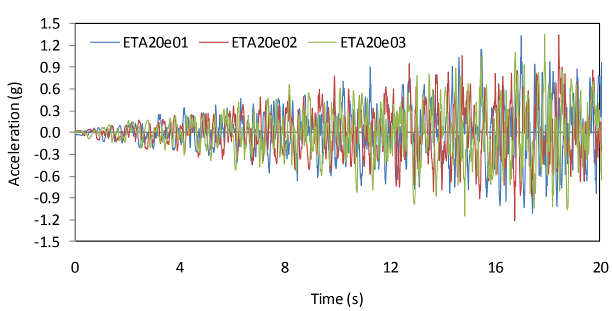


Fig. 5 Average acceleration response spectrum of ETA20e01-03 at various times



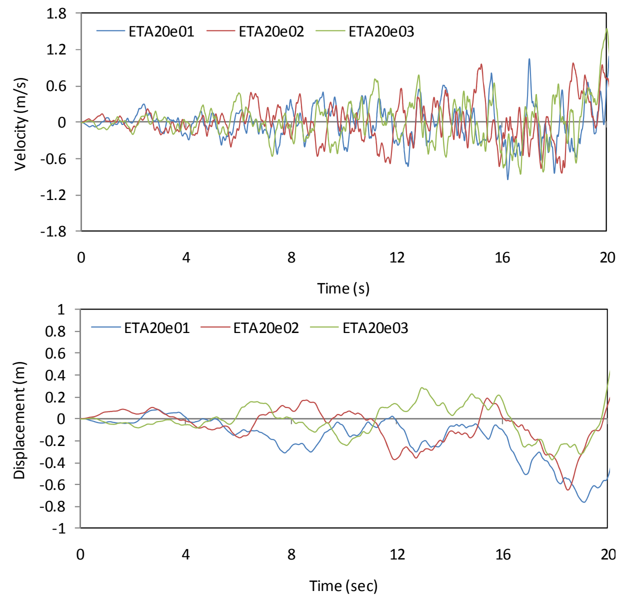


Fig. 6 Acceleration functions, velocity functions and displacement functions for e-series of ETAFs

4.2. Characteristics of site-specific ground motions

Based on ICOLD [22] three seismic performance levels should be considered for design and assessment of dam-rock-water system that are Design Base Level (DBL), Maximum Design Level (MDL) and Maximum Credible Level (MCL). In DBL, the structure should stays safe without structural damage and all vital components of the system must remain functional and/or operable. Design Base Earthquake (DBE) is defined to have a 20%-64% probability of occurrence in a 100-year-exposure period, which is equivalent to a recurrence interval (return period) of 100-450 years. In MDL, the earthquake would generate the most critical ground motions for evaluation of seismic performance of the structure among those loadings to which the structure might be exposed. In this level structure may have some structural damages but are repairable and the system is operable after renovation. Maximum Design Earthquake (MDE) is defined to have a 10%-20% probability of occurrence in a 100-year-exposure period, which is equivalent to a recurrence interval of 450-950 years. In MCL, the structure may have severe and un-repairable damages but it must not lead to sudden release of the reservoir water and loss of life in downstream. Maximum Credible Earthquake (MCE) is the largest earthquake that a fault or other seismic source can produce under current tectonic setting. This level usually has recurrence interval more than 950 years.

Generally after hazard analysis of the dam site, design spectrum for three performance levels are extracted considering ξ = 5% as shown in Fig. 7. Many factors like source depth, size of the rupture area, style of faulting, shear-wave velocity, damping characteristics of crustal rock, rock properties, local soil conditions at the site and topography of the site must be considered for extracting site-specific ground motions [23, 24]. Fig. 8 shows the horizontal components of the selected ground motions used in THA pertinent to each of the three performance levels. The intent of ground motion selection is to obtain a set of motions that will produce unbiased estimates of structural response when used with nonlinear response-history analysis. Use of few numbers of motions is only allowed if the goodness of fit of the spectra of the selected motions to that of the target. In this condition the means spectra of the scaled motions should matches well to that of target spectra at the desired period range of the structural system as shows in Fig. 9(a) [25, 26].

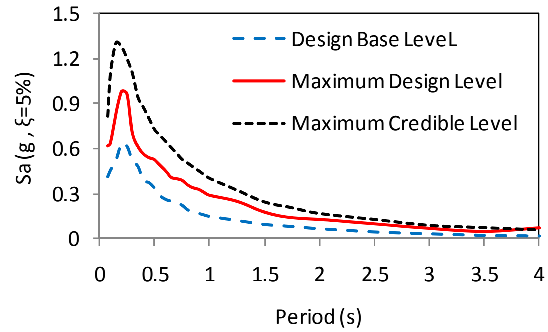


Fig. 7 Design spectrum for various performance levels in Dez Dam site

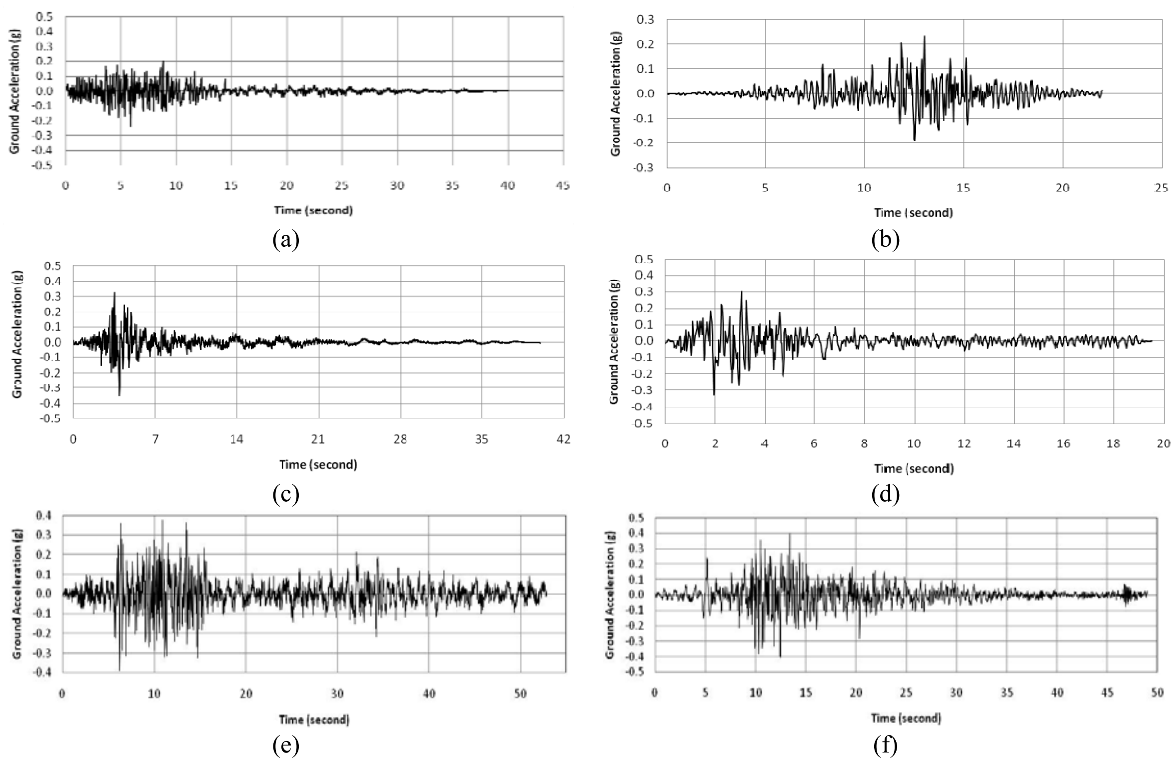


Fig. 8 Horizontal components of accelerograms (a) Northridge earthquake at 24088 PKC station - component 90, DBL; (b) Spitak earthquake at Gukasyan station - component NS, DBL; (c) Loma Prieta earthquake at 47006 Gilroy-Galivan coll station - component 337, MDL; (d) Qaen earthquake at Qaen station - component T, MDL; (e) Manjil earthquake at Abbar station - component L, MCL; (f) Tabas earthquake at Tabas station - component L, MCL

5. Calculation of Equivalent Target Time

ETA is in fact an incremental dynamic pushover procedure which can be used as alternative method for several sets of conventional THAs. As mentioned in previous sections, the third key factor in prosperity of ETA method lay on determination of the suitable equivalent target time for various seismic performance levels. Response of the dam in each time window is corresponding with a certain performance level. Ordinary low target times are corresponding with low performance levels and vice versa.

Considering the wonderful characteristics of acceleration functions that their response spectrum until base target time, $\left(t_{eq}\right)_0$, is similar to base target spectrum (base spectrum for generation of acceleration functions) and for any time like i, response spectrum of acceleration function from t_0=0 to t_1=i resembles that of the base target spectrum with a scale factor that is proportional with time i. So equivalent target time for any performance level can be calculated as:

$$t_{eq} = \psi \times \left(t_{eq}\right)_0 \tag{3}$$

where $\left(t_{eq}\right)_0$ is the base target time, t_{eq} is equivalent target time for desired ground motion (or seismic excitation level as it's in the present paper) and ψ is the scalar scale factor which called spectrum ratio. In fact, the

Eq. (3) seeks for an appropriate t_{eq} so that the response spectrum of ETAF is as similar as possible to that from desired performance level. Spectrum ratio is especial fraction of acceleration response spectrum in any desired performance level (or any desired earthquake ground motion) to ETAF's response spectrum at base target time and can be written as:

$$\psi = \frac{S_a^{EQGM}}{S_a^{ETAF}} \tag{4}$$

where S_a^{EQGM} is response spectrum of desired ground motion (or seismic performance level) and S_a^{ETAF} is response spectrum of ETAF in base target time (or base spectrum for generation of acceleration functions). As it is obvious, the above equation is a general definition and for exact results of spectrum ratio, two parameters should be determined which are: the period rage of interest and also the scaling method. Generally there are two approaches in calculation of equivalent target time: i.e. methods based on concept of spectrum matching and the statistical methods.

Considering the fact that the ETA fundamental concepts is based on the response spectrum, it is possible to use response spectrum of multi-degree-of-freedom (MDOF) systems for calculation of the equivalent target time, t_{eq}, for the specific seismic performance level. In this approach, equivalent target time is calculated using Complete Quadratic Combination (CQC) mode superposition method for the selected EDP taking into account the dam-rock-water correlated modes effects. Based on modal analysis, we know:

$$\begin{cases} \left(u_i\right)_{\max} = \Gamma_i\left((S_d)_i\right) \\ \left(\dot{u}_i\right)_{\max} = \Gamma_i\left((S_v)_i\right) \\ \left(\ddot{u}_i\right)_{\max} = \Gamma_i\left((S_a)_i\right) \end{cases} \tag{5}$$

where $\left(u_i\right)_{\max}$, $\left(\dot{u}_i\right)_{\max}$ and $\left(\ddot{u}_i\right)_{\max}$ are maximum displacement, velocity and acceleration for i^{th} mode; $(S_d)_i$, $(S_v)_i$ and $(S_a)_i$ are maximum spectral displacement, velocity and acceleration for single-degree-of-freedom (SDOF) system and Γ_i is participation factor (PF) in i^{th} mode that can be written as:

$$\Gamma_i = \frac{\tilde{L}_i}{\tilde{M}_i} \tag{6}$$

where $\tilde{L}_i$ and $\tilde{M}_i$ are excitation factor and generalized mass for i^{th} mode. Also we know that [27]:

$$(S_a)_i = \omega_i \times (S_v)_i = \omega_i^2 \times (S_d)_i \tag{7}$$

where ω_i is angular frequency in i^{th} mode. Based on Square-Root-of-Sum-of-Squares (SRSS) mode superposition method that is suitable for systems with fully separated modes and also considering the Eqs. (5), (6) and (7), the maximum displacement (this rule is feasible for velocity and acceleration with some changes) for MDOF system using linear combination can be written as:

$$u_{\max} = \sqrt{\sum_{i=1}^{n} u_i^2} = \sqrt{\sum_{i=1}^{n}\left[\Gamma_i\left((S_d)_i\right)\right]^2} = \sqrt{\sum_{i=1}^{n}\left\{\frac{\Gamma_i}{\omega_i^2}(S_a)_i\right\}^2} \tag{8}$$

where $u_{\max}$ is the maximum displacement for MDOF system. Substituting of the Eq. (8) into Eq. (4) yields:

$$\psi_{SRSS}^{u} = \frac{\sqrt{\sum_{i=1}^{n}(u_i^{EQGM})^2}}{\sqrt{\sum_{i=1}^{n}(u_i^{ETAF})^2}} = \frac{\sqrt{\sum_{i=1}^{n}\left\{\Gamma_i\left((S_d)_i^{EQGM}\right)\right\}^2}}{\sqrt{\sum_{i=1}^{n}\left\{\Gamma_i\left((S_d)_i^{ETAF}\right)\right\}^2}} = \frac{\sqrt{\sum_{i=1}^{n}\left\{\frac{\Gamma_i}{\omega_i^2}(S_a)_i^{EQGM}\right\}^2}}{\sqrt{\sum_{i=1}^{n}\left\{\frac{\Gamma_i}{\omega_i^2}(S_a)_i^{ETAF}\right\}^2}} \tag{9}$$

where ψ_{SRSS}^{u} is the spectrum ratios based on SRSS mode superposition for displacement in desired seismic performance level and *n* is the number of all effective modes of the dam-rock-water coupled system. In many of structures like bridges, dams, tall buildings and towers, higher modes have significant effect in seismic behavior of the system. In these types of structures many of the modes are correlated with each other and so using CQC mode superposition is reasonable for calculation of the spectrum ratio. Eq. (9) can be rewritten based on CQC linear mode combination method as follow:

$$\psi_{CQC}^{u}=\sqrt{\frac{\sum_{i=1}^{n}(u_i^{EQGM})^2+2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n}\rho_{ij}(u_i^{EQGM})(u_j^{EQGM})}{\sum_{i=1}^{n}(u_i^{ETAF})^2+2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n}\rho_{ij}(u_i^{ETAF})(u_j^{ETAF})}}$$

$$=\sqrt{\frac{\sum_{i=1}^{n}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_i^{EQGM}\right)\right\}^2+2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n}\rho_{ij}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_i^{EQGM}\right)\right\}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_j^{EQGM}\right)\right\}}{\sum_{i=1}^{n}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_i^{ETAF}\right)\right\}^2+2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n}\rho_{ij}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_i^{ETAF}\right)\right\}\left\{\frac{\Gamma_i}{\omega_i^2}\left((S_a)_j^{ETAF}\right)\right\}}} \tag{10}$$

where ψ_{CQC}^{u} is the spectrum ratio based on CQC mode superposition for displacement in desired seismic performance level, n is the number of all effective modes for the dam-rock-water coupled system and ρ_{ij} is the cross-modal coefficient and defined as:

$$\rho_{ij}=\frac{8\xi^2(1+r)r^{3/2}}{(1-r^2)^2+4\xi^2 r(1+r)^2} \tag{11}$$

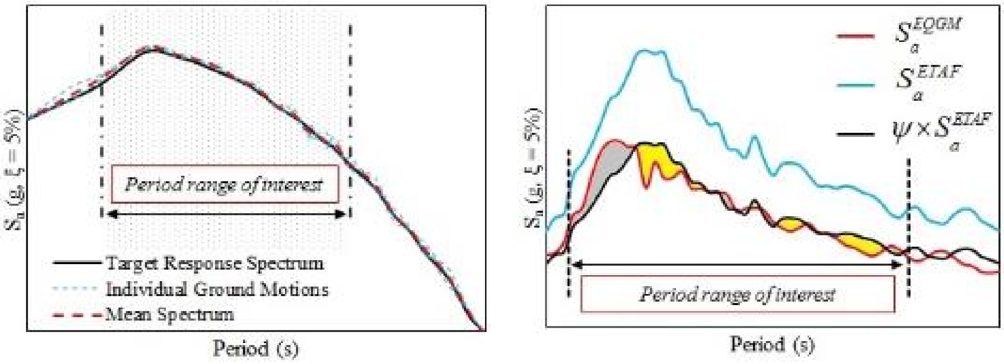
where $r=T_i/T_j$ is the proportion of the two modes and ξ is the damping ratio. For the composite structure with obviously different damping characteristics more complicate methods should be used as reported by Ruifang and Xiyuan [28]. Using this method the equivalent target time for various performance levels is obtained as shown in Table 1. In this method the possible EDPs are displacement, velocity and acceleration.

Table 1 Calculated equivalent target time for the different seismic performance levels at Dez Dam site

Method	Calculated Equivalent Target Time		
Displacement-based	$\left(t_{eq}\right)_{CQC}^{u}(MCL)=6.25\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{u}(MDL)=4.01\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{u}(DBL)=3.31\,\mathrm{s}$
Velocity-based	$\left(t_{eq}\right)_{CQC}^{\dot{u}}(MCL)=6.51\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{\dot{u}}(MDL)=4.08\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{\dot{u}}(DBL)=3.32\,\mathrm{s}$
Acceleration-based	$\left(t_{eq}\right)_{CQC}^{\ddot{u}}(MCL)=6.84\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{\ddot{u}}(MDL)=4.33\,\mathrm{s}$	$\left(t_{eq}\right)_{CQC}^{\ddot{u}}(DBL)=3.30\,\mathrm{s}$
Response spectrum-based	$t_{eq}^{*}(MCL)=6.36\,\mathrm{s}$	$t_{eq}^{*}(MDL)=4.17\,\mathrm{s}$	$t_{eq}^{*}(DBL)=3.25\,\mathrm{s}$

Another method for calculation of the equivalent target time is direct implementation of the response spectrum concept in conjunction with statistical methods. In this approach, the first step is determination of the period range of interest for the structural system. In this research the period range is selected in a way that includes at least 90% of the total mass of the structural system [29, 30]. In the proposed technique at the current study, 10th second of ETAFs is selected as the base target time and therefore, the average response spectrum resulted up to the 10th second of ETAFs is interpreted as the base response spectrum. In the second step the base response spectrum of ETAF is multiplied in the ψ factor, spectrum ratio, in order to generate the scaled ETAF response spectrum. The scaled ETAF response spectrum, $\psi\times S_a^{ETAF}$, is then compared with the response spectrum of the desired ground motion (or desired seismic performance level), S_a^{EQGM}, and the optimization function seeks to find the optimum value for the ψ factor in a way that the sum of the positive and negative areas enclosed by the two curve set to zero. It's noteworthy that the positive finite area is referred to the condition in which the S_a^{EQGM} has the higher value than the $\psi\times S_a^{ETAF}$ and vice versa.

After calculation of the ψ factor, the equivalent target time for the desired ground motion (or seismic performance level) can be calculate using Eq. (3). Fig. 9(b) shows schematically the proposed method which uses the arithmetic mean weight concept and hereafter the spectrum ratio and also the equivalent target time calculated by this method are referred as ψ^* and t_{eq}^* respectively. Table 1 (last row) also presents the calculated equivalent target times using the proposed algorithm. Based on analyses conducted by the authors, the appropriate equivalent target time in linear analysis of the concrete dams is suggested as follows: $\left(t_{eq}\right)_{CQC}^{u}$ for extracting the displacement response; $\left(t_{eq}\right)_{CQC}^{\dot{u}}$ for extracting velocity response; $\left(t_{eq}\right)_{CQC}^{\ddot{u}}$ for extracting the acceleration response; and t_{eq}^* for extracting the stresses within the dam body.



(a) (b)

Fig. 9 (a) Scaling the ground motions to target spectrum in time-history analysis; (b) Determining the optimum value for the ψ factor using the response spectrum of the desired ground motion (or desired seismic performance level)

6. Results and Discussion

Flowchart shown in Fig. 10 describes the proposed methodology for comparing the results obtained from ETA and THA methods. Crest displacement at the end of static and thermal analyses is 2.66mm in downstream direction. It is noteworthy that for obtaining the extreme results of the structural system either in THA or ETA methods, seismic excitation should be applied to the system in two opposite directions to find out the most critical direction that would cause the largest structural response. For one-component excitation a total of two permutations are required which are shown as +US/DS and -US/DS in which (+) and (-) signs indicate that earthquake record is multiplied by +1 (zero phase) or -1 (180 phase) to account for the most unfavorable earthquake direction [23].

Determine the design spectrum pertinent to site seismic performance levels

↓

Select a bunch of appropriate ground motions scaled based on the design response specrtum

↓

Generate ETAFs based on the design response spectrum

↓

Calculate the equivalent target time corresponding to desired seismic performance level considering the appropriate EDP

↓

Analyze the structural system based on THA method and extract the maximum absolute values of the desired EDP

↓

Analyze the structural system based on ETA method and determine the maximum absolute value of the desired EDP for time interval [0, t_{eq}] using ETC

↓

Compare the results of the THA and ETA method for the desired EDP

Fig. 10 Methodology of comparing the results obtained from THA and ETA methods

In the following subsections, the ability of ETA method in estimation of various parameters such as displacement, velocity, acceleration and stress at the crest is considered. Obviously, parameters like velocity and acceleration at the crest are not utilized directly for seismic analysis and safety evaluation of concrete arch dams. However, investigation on these parameters can be valuable mathematically.

6.1. Displacement as EDP

Fig. 11 shows displacement time history at the crest in which excitation is in +US/DS direction for instance and also the absolute extreme values are shown in this figure. On the other hand Fig. 12 shows the absolute values of displacement time history extracted from ETA method.

ETC, its Linear Trend Line (LTL) and the average ETCs are shown in this figure. Detailed values for displacement at equivalent target time, $\left(t_{eq}\right)_{CQC}^{u}$ for various performance levels were represented in Tables 2 to 4. In theses tables, the percentage of the errors can be calculated as follow:

$$Err\% = \left(\frac{Response_{ETA} - Response_{THA}}{Response_{THA}} \right) \times \left(100\%\right) \qquad (12)$$

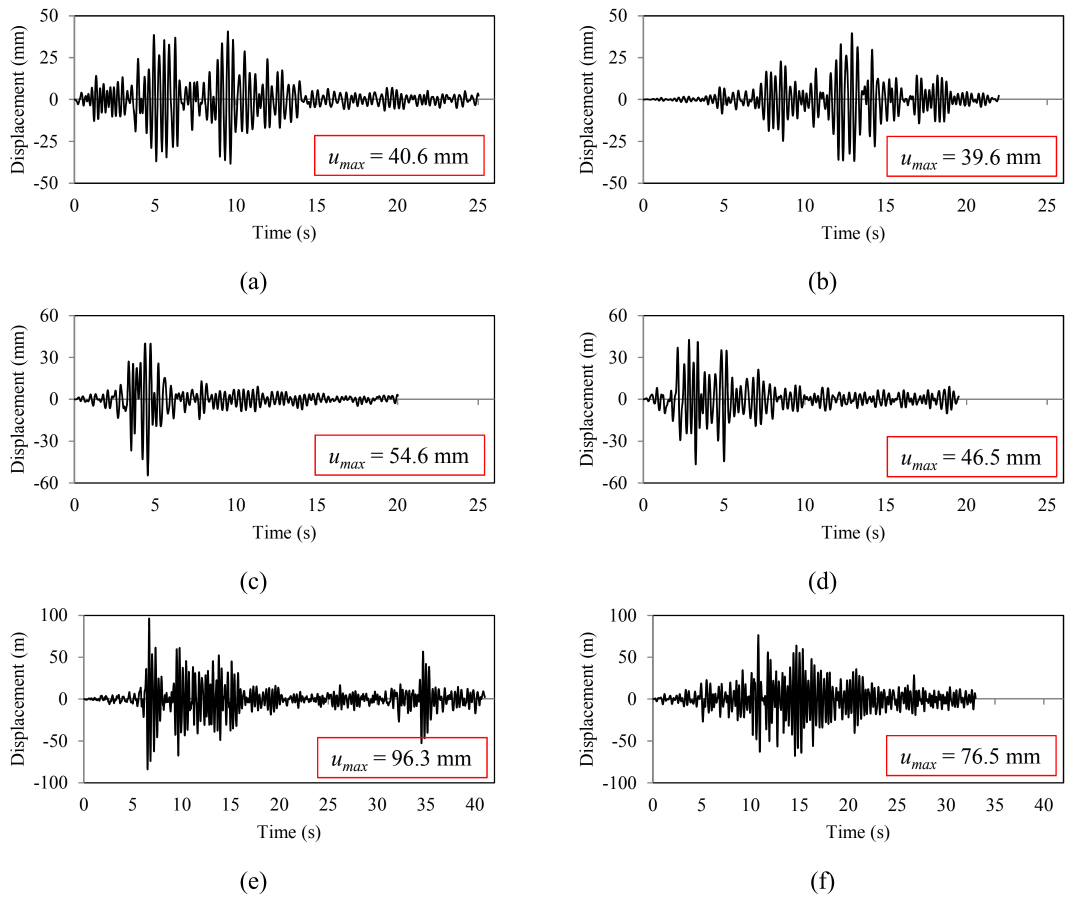
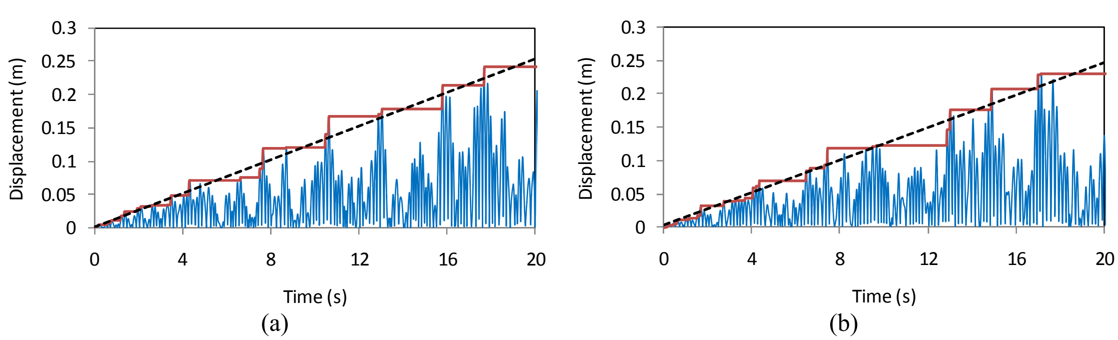
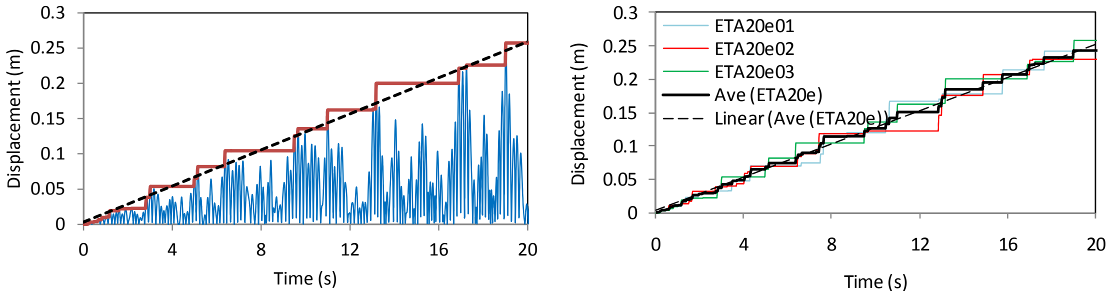


Fig. 11 Displacement time history at the crest point in +US/DS direction (a) Northridge, DBL; (b) Spitak, DBL; (c) Loma Prieta, MDL; (d) Qaen, MDL; (e) Manjil, MCL; (f) Tabas, MCL





(c) (d)

Fig. 12 Time history of absolute displacement in +US/DS direction, ETC and its LTL for (a) ETA20e01; (b) ETA20e02; (c) ETA20e03 (d); Ave (ETA20e)

Table 2 Maximum absolute displacement estimated by ETA method at MCL and pertinent errors

ETAF No.	Type of curve	Response at $\left(t_{eq}\right)^{u}_{CQC}(MCL)=6.25\text{s}$	ETAFs to MCEs Err (%)		
			Manjil	Tabas	AVE
ETA20e01	ETC	70.3mm	-27.0	-8.0	-18.6
	LTL	80.8mm	-16.1	5.7	-6.4
ETA20e02	ETC	70.1mm	-27.2	-8.3	-18.8
	LTL	79.4mm	-17.6	3.9	-8.1
ETA20e03	ETC	81.9mm	-14.9	7.2	-5.2
	LTL	83.2mm	-13.7	8.8	-3.7
ETA20eAVE	ETC	74.1mm	-23.0	-3.0	-14.2
	LTL	81.1mm	-15.8	6.1	-6.1

Table 3 Maximum absolute displacement estimated by ETA method at MDL and pertinent errors

ETAF No.	Type of curve	Response at $\left(t_{eq}\right)^{u}_{CQC}(MDL)=4.01\text{s}$	ETAFs to MDEs Err (%)		
			Qaen	Loma-Prieta	AVE
ETA20e01	ETC	47.6mm	2.3	-12.7	-5.8
	LTL	52.5mm	12.8	-3.8	3.9
ETA20e02	ETC	44.6mm	-4.1	-18.1	-11.7
	LTL	52.3mm	12.4	-4.1	3.5
ETA20e03	ETC	53.8mm	15.6	-1.3	6.5
	LTL	54.4mm	17.0	-0.2	7.7
ETA20eAVE	ETC	48.8mm	4.8	-10.5	-3.5
	LTL	53.1mm	14.1	-2.7	5.0

Table 4 Maximum absolute displacement estimated by ETA method at DBL and pertinent errors

ETAF No.	Type of curve	Response at $\left(t_{eq}\right)^{u}_{CQC}(DBL)=3.31\text{s}$	ETAFs to DBEs Err (%)		
			North-Ridge	*Spitak*	AVE
ETA20e01	ETC	33.3mm	-18.0	-15.8	-16.9
	LTL	43.6mm	7.3	10.3	8.8
ETA20e02	ETC	40.6mm	-0.1	2.7	1.3
	LTL	43.4mm	6.8	9.8	8.3
ETA20e03	ETC	53.8mm	32.4	36.1	34.2
	LTL	45.6mm	12.2	15.3	13.7
ETA20eAVE	ETC	42.6mm	4.8	7.7	6.3
	LTL	44.3mm	9.0	12.0	10.5

It should be pointed out that the considered node is the most critical one within the dam body yet all the errors are in reasonable range. In addition, when three ETAFs are used and compared with the average results of THA, the calculated errors decrease meaningfully. Based on these tables, maximum errors are obtained for MCL and the minimum ones are pertinent to MDL. It is notable that all the errors in average MCL (last column of table 2) are negative, which means the estimated results by ETAFs for crest displacement are less than its real values obtained from THA method. On the other hand, almost all the errors in average DBL (last column of table 4) are positive, meaning that the estimated responses by ETAFs are more than their real values obtained from THA method. Finally the errors in average MDL (last column of table 3) are sometimes positive and sometimes negative, which means the estimated results by ETAFs fluctuate around the real values.

Fig. 13 shows the average non-concurrent envelopes of displacement along the height of the crown cantilever extracted from THA and ETA method in various performance levels. As can be seen, there is great compatibility between the results of ETA method and THA in DBL and MDL. Although there is good compatibility in lower part of the crown cantilever in MCL, some differences can be observed in upper part especially in vicinity of the crest. Moreover, percentages of the errors between each of ETAF and also average of them (ETA20eAVE) with average of values extracted from THA method in each performance level are shown in right-side column. It is obvious that using average of three ETAFs can reduce the errors considerably. It is shown that maximum percentage of the errors, which belongs to MCL, is less than 16% for upper 1/3 part of the cantilever.

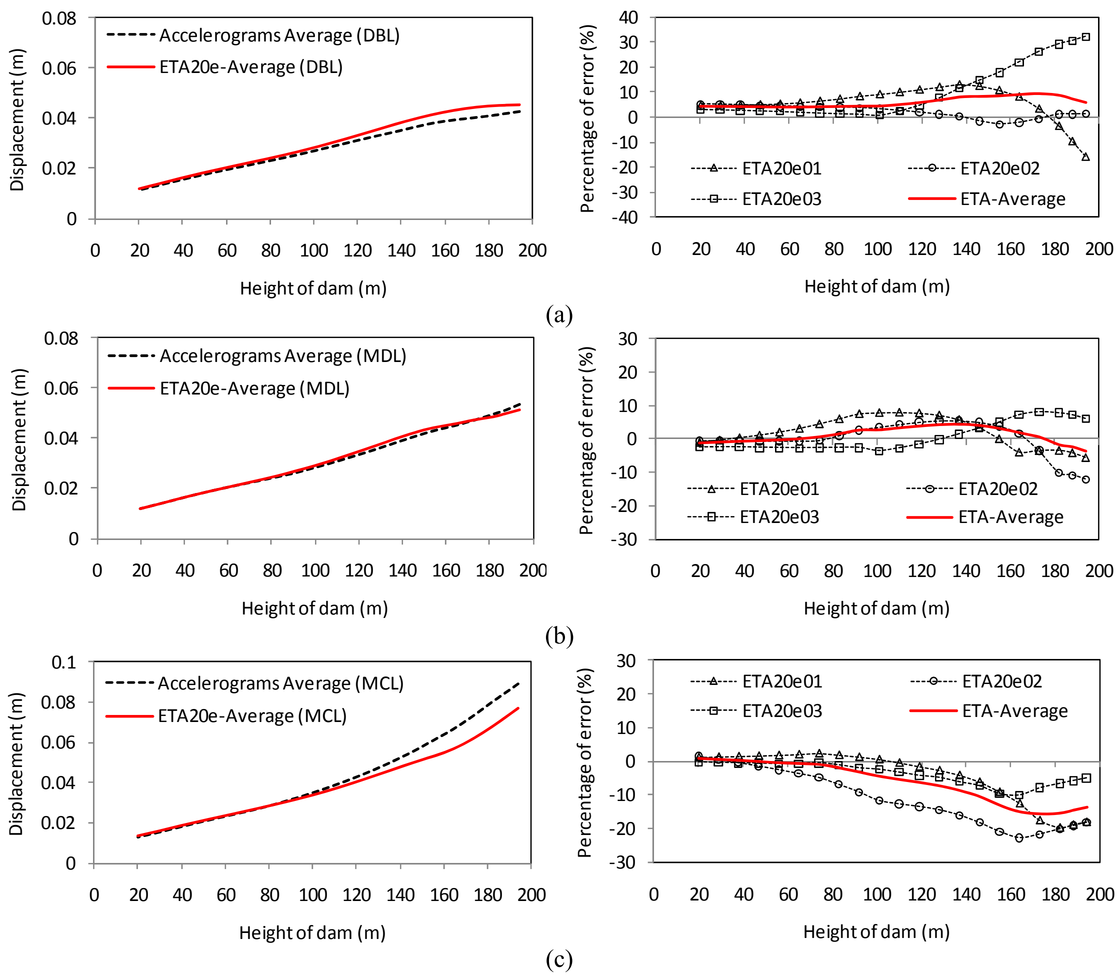
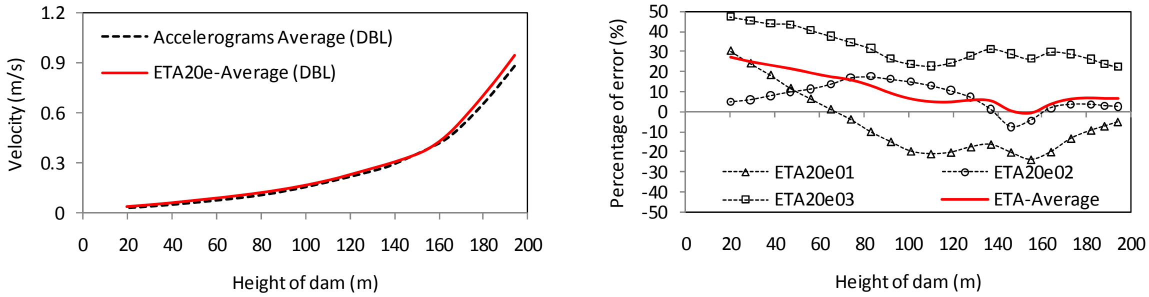


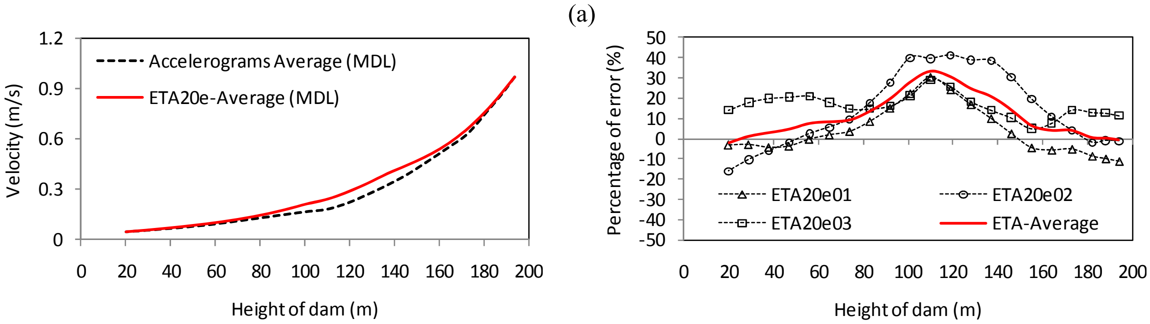
Fig. 13 Non-concurrent envelope of displacements and the percentage of the errors along the crown cantilever, (a) DBL; (b) MDL; (c) MCL

6.2. Velocity as EDP

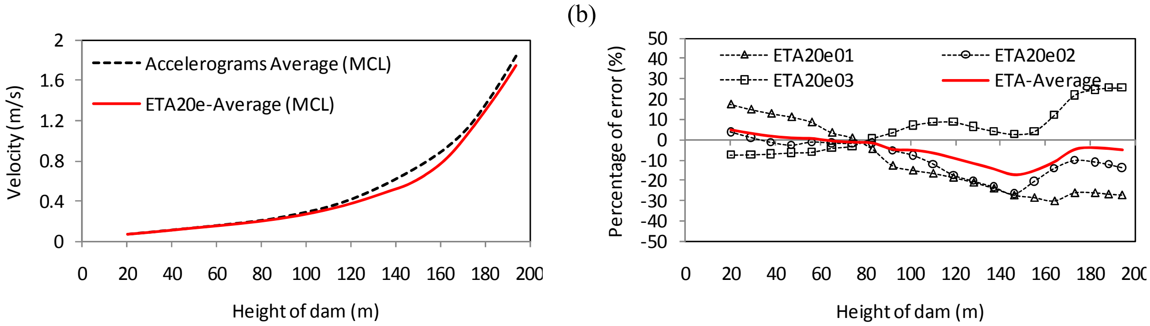
Fig. 14 shows the average non-concurrent envelope of velocity along the height of the crown cantilever extracted from ETA. As can be seen, there is good compatibility between the results of ETA method and THA in three performance levels. In DBL, there is notable error in lower 1/3 part of the cantilever, however, for upper 1/3 of the crown cantilever there is good agreement between two methods. Although in MDL some high errors are observed in middle 1/3 of the height, for the rest of the cantilever the percentage of the errors are reasonable. In MCL, some errors are shown in upper half of the crown cantilever but all the errors are limited to 17%.



(a)



(b)

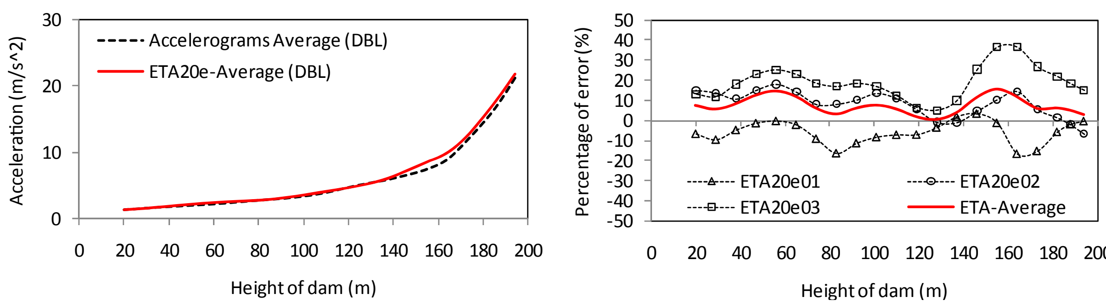


(c)

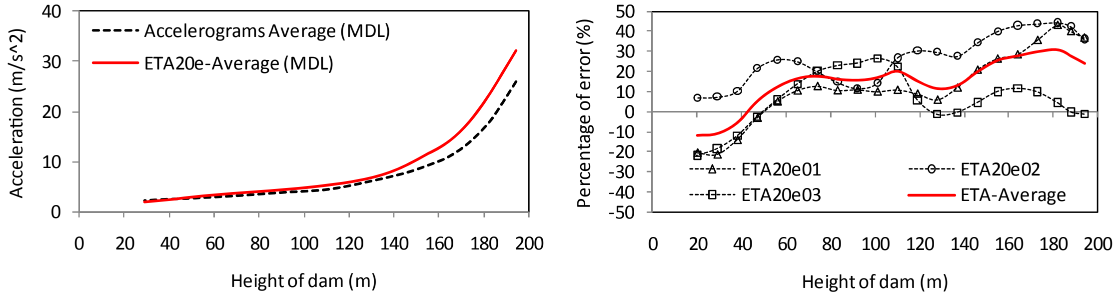
Fig. 14 Non-concurrent envelope of velocities and the percentages of the errors along the crown cantilever, (a) DBL; (b) MDL; (c) MCL

6.3. Acceleration as EDP

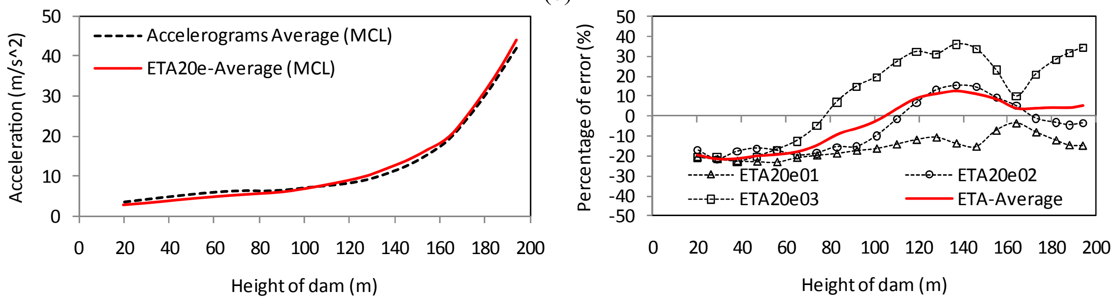
Fig. 15 shows the average of non-concurrent envelope of accelerations extracted from ETA and THA along the height of the crown cantilever and also the pertinent errors. In DBL, error of ETA20eAVE fluctuates along the cantilever height. In MDL, responses have the same trends but ranges of fluctuations are more than those observed in DBL results. In MCL, ETA method underestimates the results for lower half of the cantilever and overestimates them for the upper half.



(a)



(b)



(c)

Fig. 15 Non-concurrent envelope of accelerations and the percentages of the errors along the crown cantilever, (a) DBL; (b) MDL; (c) MCL

6.4. Stress as EDP

One of the important factors in seismic assessment of arch dams is non-concurrent stress envelopes in US and DS faces of the dam body. Subsequently this section investigates capability of ETA method in estimation of the non-concurrent stress envelopes in both faces. Fig. 16 shows the average of non-concurrent envelope of first principal stresses extracted from ETA and THA on both faces of the dam body. As is visible on the figure, the maximum stress values in DBL (either in THA or ETA method) is about 8.2MPa; in MDL is about 9.0MPa and in MCL it increases to 17MPa suddenly. It shows that dam will experiences severe damage in MCL level. On the other hand, Fig. 17 shows the average of non-concurrent envelope of third principal stresses in two aforementioned methods. The minimum stress values in this condition (also can be interpreted as compressive stress) are -15MPa, -16MPa and -23MPa for DBL, MDL and MCL respectively. Based on these figures, it is found that ETA method can estimate non-concurrent stresses envelopes and extreme values with an acceptable approximation in arch dams.

6.5. Comparing the computational efforts in ETA vs. THA

Table 5 shows the general specifications of the utilized acceleration time histories for THA. Considering time step of 0.02s for earthquake records, the number of the load steps required for analyses can be obtained by dividing the selected significant duration to time step as shown in table. Finally, the number of total load steps required for two series of analyses (corresponding to +US/DS and –US/DS directions), based on THA method is 2×8025=16050.

Table 5 Some specification of utilized records in THAs

Ground motion	Total Duration	Significant Duration	Time Step	Number of Load Steps
Tabas-L	48.96s	33.00s	0.02s	1650
Manjil-L	52.92s	41.00s	0.02s	2050
Loma-Prieta-337	39.94s	20.00s	0.02s	1000
Qaen-T	19.50s	19.50s	0.02s	975
North-Ridge-90	40.00s	25.00s	0.02s	1250
Spitak-NS	22.00s	22.00	0.02s	1100
SUM		160.50s		8025

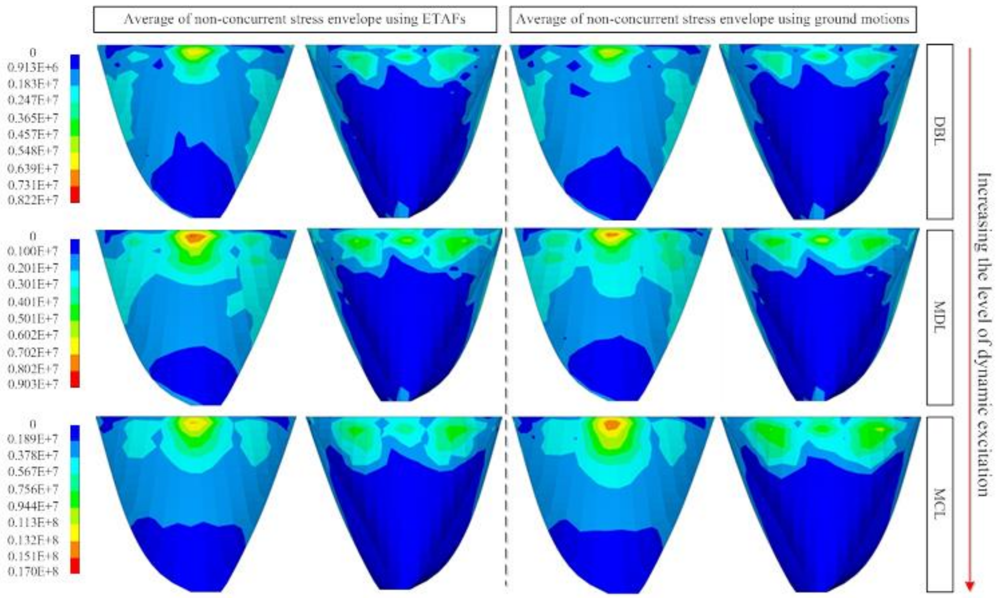


Fig. 16 Average of non-concurrent envelopes of first principal stresses on US and DS faces, (Pa)

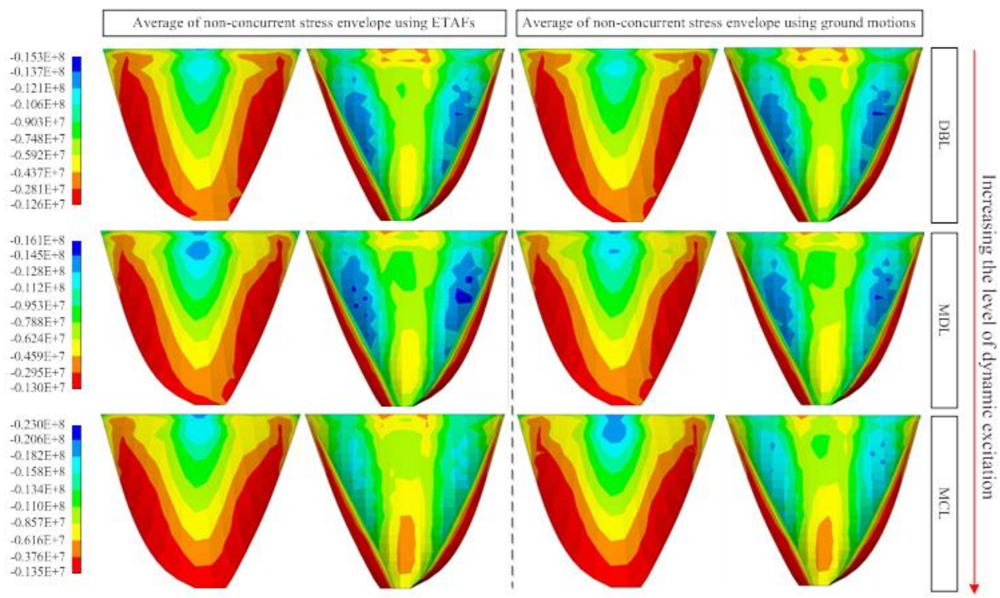


Fig. 17 Average of non-concurrent envelopes of third principal stresses on US and DS faces, (Pa)

On the other hand, referring to table 1, it is obvious that the maximum target time which has been calculated for ETA is less than 7.00s (exactly 6.84s). So analyses of provided finite element model using the first 7.00s of the three ETAFs in the two +US/DS and –US/DS directions lead to an analysis with total time of 42.00s. Considering that time step in ETAFs is 0.01s, the number of total load steps required for ETA, is 42.00÷0.01=4200. As can be seen, ETA method reduces the computational efforts and subsequently the cost of analyses about 75% in comparison with THA method in linear analysis of *Dez* Dam.

Although the main time-consuming aspect in seismic assessment of an arch dam is development of 3D model, analysis of such huge systems by different ground motions for reduction of dam responses dependency to selected ground motions can be a great concern for dam analyzers. On the other hand any probable changes in intensity of the ground motions, due to complementary hazard analysis of the dam site, can lead to totally re-analysis of structural system in THA method, while in ETA method it is just required to calculate the modified equivalent target time and evaluate the results at this new time without any more analysis.

7. Conclusion

In the present study, the application of Endurance Time Analysis (ETA) is investigated on seismic analysis of coupled arch dam-rock-water system. The constitutive model for the mass concrete, foundation rock and the reservoir water is assumed to be linear elastic. Rock is modeled to be a massless medium and the reservoir is taken to be compressible. Large deformation effects of the finite element model under the extreme dynamic excitation are also considered using the co-rotational formulation. *Dez* Dam in Iran was selected as case study and the dam body, soil medium and the reservoir was modeled using appropriate finite element model. In static loading phase, the applied loads on the system were self weight, hydrostatic load in normal water level and the thermal load corresponding to summer condition. All excitations in ETA and THA for the three performance levels (specific for the dam site) were applied in only US/DS direction.

Based on the conducted dynamic analyses, it was found that all responses of the dam extracted from ETA method up to the equivalent target time are in good agreement with the extreme values extracted from THA method. Finally, the great advantage of ETA method lays on lower computational efforts with respect to group of THA especially when the system should be analyzed in different seismic performance levels. Considering benefit over cost (B/C) theorem for analysis of dam-rock-water system using ETA method, it was found that the proposed method has the great capability for seismic assessment of the coupled system.

Abbreviations

ETA	Endurance Time Analysis
THA	Time History Analysis
POA	Pushover Analysis
IDA	Incremental Dynamic Analysis
EDP	Engineering Demand Parameter
IM	Intensity Measure
ETAF	Endurance Time Acceleration Function
SDOF	Single Degree of Freedom
MDOF	Multi Degree of Freedom
ETC	Endurance Time Curve
DOF	Degree of Freedom
NWL	Normal Water Level
US/DS	Upstream/Downstream
NEHRP	National Earthquake Hazard Reduction Program
FEMA	Federal Emergency Management Agency
INBC	Iranian National Building Code
DBL	Design Base Level
MDL	Maximum Design Level
MCL	Maximum Credible Level
DBE	Design Base Earthquake
MDE	Maximum Design Earthquake
MCE	Maximum Credible Earthquake
CQC	Complete Quadratic Combination
SRSS	Square Root of Sum of Squares
PF	Participation Factor
LTL	Linear Trend Line
B/C	Benefit over Cost

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