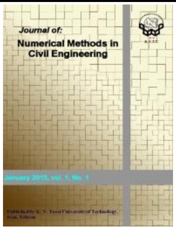

Numerical Methods in Civil Engineering

Application of endurance time method in damage assessment of concrete moment frames

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Abstract:

Nonlinear Time History (NTH) analysis is currently the most reliable method for estimating structural behavior. Considerable computational demand and complexity of this method may cause difficulty for its routine practical application. Based on the Methodology of Endurance Time (ET) method, it can estimate the nonlinear response of structures with a much lower computational cost. In this research, the reliability of the ET method in the analysis of concrete moment frames will be discussed. The results of the ET method are compared with those acquired from the NTH method by considering some energy Engineering demand parameters(EDP) like base shear roof displacement. Furthermore, the accuracy of this method for estimating damage in structures was evaluated by considering inter story Drift, Park-Ang, and Bozorgnia-Bertero models as samples of damage indices. It is observed that the nonlinear dynamic response of structures and damage indices can be estimated by the ET method with reasonable accuracy.

1. Introduction

Non-linear response history analysis (RHA) or time history analysis (NTH) is the most rigorous procedure to compute the seismic demands. In current civil engineering practices, it is more acceptable to use the non-linear static procedure (NSP) or pushover analysis [1]. The seismic demands are computed by non-linear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a predetermined target displacement is reached. Both the forces distribution and target displacement are based on the assumption that the response is controlled by the fundamental mode and that the mode shape remains unchanged after the structure yields [1]. Obviously, after the structure yields, both assumptions are approximate, but investigations have led to good estimates of seismic demands. Nevertheless, such satisfactory predictions of seismic demands are mostly restricted to low and medium-rise structures provided, the inelastic response is distributed throughout the height of the structure [1].

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Non-linear time history analysis of a detailed (and often quite complex) analytical model subjected to a suite of representative site-specific ground motions using a well-calibrated analysis tool, is likely the best option for the estimation of these demands [2].
One of the advantages of structural analysis like static-pushover is that it produces a complete picture of the structure from start to collapse. This methodology is considered in RHA as Incremental Dynamic Analysis (IDA). It involves subjecting a structural model to one (or more) ground motion record(s), each scaled to multiple levels of intensity, thus producing one (or more) curve(s) of structural response parameterized versus intensity level [3]. Each dynamic analysis can be characterized by at least two scalars, an intensity measure (IM), which represents the scaling factor of the record, and an engineering demand parameter (EDP), which monitors the structural response of the model [4]. From the results of such computations, it is possible to determine structural capacities (or ground motion intensities) corresponding to various limit states such as; immediate occupancy (IO), collapse prevention (CP), or global instability (GI) [5].

The IDA method requires great computational effort and time. The developers of the IDA method have introduced an approximate method of using a single-degree-of-freedom (SDOF) system to estimate the static pushover curve for multi-degree-of-freedom (MDOF) [4]. In another effort, modal pushover analysis (MPA) [1,6,7] is used instead of real RHA [5].

Endurance Time (ET) method is a response history based analysis procedure that can be used for estimating the seismic response of structures at different excitation levels in each response history [8]. In the ET method, a particular ground motion named acceleration function, tries to simulate minor, moderate, and major earthquakes throughout its duration in a single record [9]. By correlation of Engineering demand parameter (EDP) and intensity measure, ET method provides seismic demand estimation with considerably lower computational demand compared to NTH methods.

The key factor of the ET method is producing a proper acceleration function that corresponds with EDPs obtained from IDA analysis at each intensity measure. Optimization procedures are used to produce such excitation functions [12,30,31].

In this method, an acceleration function is produced so that structural demand increases from elasticity to global dynamic instability. Therefore, the concept of this method can be used to compute the IDA curve with little computational effort. If the response spectrum of selected EDP is plotted using the ET method, the value at each time corresponds with the maximum response of that EDP in one RHA analysis. Thus, many non-linear RHAs required in IDA is replaced by one ET analysis.

2. Basic Concept

In order to explain the ET concept, a prototype with three alternative designs corresponding to different levels of performance is considered (Fig. **1**). To assess the performance level of each model, the models are placed on a shaking table and a gradually intensifying acceleration function named ET function is exerted until a failure occurs. This function is set to match a predefined hazard level at a specific time and corresponding performance level as shown in Figure 1. If the value of EDP response in a model for a hazard level is less than the minimum of design criteria for that performance level, the model can meet building performance objectives for that seismic hazard level. Fig. **1** shows that design C-B-A cannot meet IO-LS-CP performance levels relatively, so we can conclude that design A has the best performance.

To compare the results of the ET method with NTH analysis, seven records are selected and scaled in an intensifying manner to produce the IDA curve. These IDA curves are averaged to produce the average IDA performance curve which is estimated with ET analysis. Since ET excitation functions (ETEFs) are produced by a numerical calculation, to lessen dispersion of each record response from the target, the average of 3 ETEFs is used for comparison. See reference [12] and [13] for more detail about how ET records are produced based on given natural records.

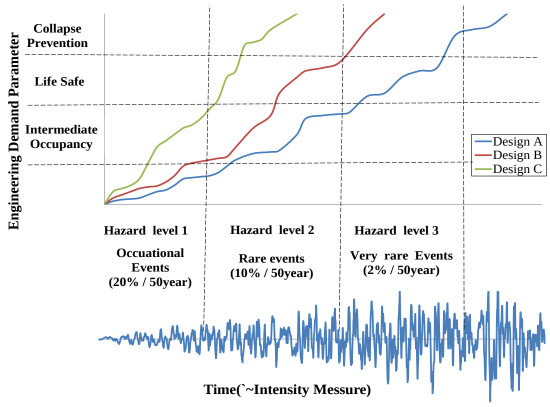


Fig. 1: Concept of ET methods

Now, the question is whether it is possible to establish a meaningful correlation between the intensity of an intensifying excitation and that of ground motions or not. It turns out that the concept of response spectrum can be used quite effectively in producing intensifying excitation functions. The point is that the response spectrum strongly reflects two major characteristics of any ground motion, i.e., the intensity and the frequency content. Two dynamic excitations with similar response spectrums produce almost similar responses in most structures. Thus, if the response spectrum of the ETEFs at a particular time is generated to match a particular response spectrum corresponding to say the average response spectrum of a set of ground motions, the produced response at that time can be considered as a good estimation of the expected average response of the structure when subjected to those ground motions [10].

Note that the ET record is not just an artificial record and its purpose is different from an artificial record production. Because of randomness, uncertainty, and site-dependency of actual earthquake records, artificial records are produced to represent specific sites including randomness. On the other hand, ET record is produced to lessen the cost and complexity of the calculation of the NTH method. The ET records can be produced from an artificial record too. In this way, ET can predict the response of a structure to that artificial record within an accepted range of errors.

3. ET Excitation Functions

The first step of implementing the concept of ET is to produce practical intensifying ETEFs that in this case, are translated into a meaningful correspondence between the responses of a structure at a particular time in ET analysis and the average response to ground motions representing the seismicity of a particular site at a certain hazard level. As explained in the previous section, the concept of response spectrum can be used effectively in providing a preliminary formulation of the problem. A typical code design spectrum can be considered as a good starting point. In this way, the problem is defined as generating an intensifying acceleration function with a response spectrum that is matched with the code design spectrum at a particular time. This particular time will be named hereafter as the target time, i.e., t_{target}. In every specific time before t_{target}, the response spectrum produced by ETEF should be less than the considered design spectrum, and after the target time, it should be greater than the considered design spectrum at all times. It is possible to consider different target spectra at different times pertaining to different hazard levels. As a preliminary trial, the same target spectrum, which is linearly scaled with the time, will be assumed here. This means that the overall shape of the target spectrum is presumed to remain unchanged, and target spectrums at various times are scaled versions of the same spectrum, referred to as the template spectrum here. This means that the response produced by ETEF at target time should match the considered design spectrum at a time equal to ½ of the t_{target}, it should produce a response spectrum that matches the design spectrum with a scale factor of ½. Similarly, a linear scaling should be applied at all other times. This requirement can be formulated as follows [8]:

$$S_{ac}(T,t) = \frac{t}{t_{target}} S_{ac}(T) \tag{1}$$

$$S_{uc}(T,t) = \frac{t}{t_{target}} S_{ac}(T) \times \frac{T^2}{4\pi^2} \tag{2}$$

In which S_{ac} (T) is the template spectrum, S_{ac} (T, t) is the target spectrum to be approached at time t of ETEF and S_{uc} (T, t) is the target displacement spectrum to be induced at time t by ETEF. This formula simply states that the acceleration response produced by ETEF at a particular time t should remain proportional to the considered template spectrum and scaled in a linear manner as a function of time. This assumption causes continuity between different steps. Obviously, these simplifications are not an inherent part of the concept behind the ET method, but are just being made in order to synthesize a preliminary ETEF function [8].

Analytical approaches to find acceleration functions that satisfy conditions such as Equation (1) are formidably complicated [12, 30, 31]. Therefore, the problem solving approach is by formulating it as an unconstrained optimization problem in the time domain, as follows [13,32]:

$$\text{Minimize}\, F(a_g) = \int_{T_{min}}^{T_{max}} \int_0^{t_{max}} \left(\left[S_a(T,t) - S_{ac}(T,t) \right]^2 \right) \text{dtdT} \tag{3}$$

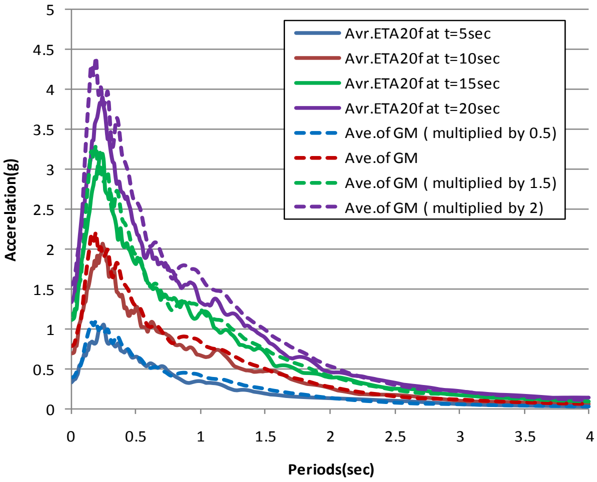
There are some differences between results of ETEF and ground motions which occur mostly due to the incompatibility of response spectra of ET acceleration functions and design spectrum. This setback is caused by the roughness of the target spectrum and optimization problems in generating ET acceleration functions [11].

ET acceleration functions that are compatible with the real earthquakes are used for verification of the applicability of this new procedure in concrete moment frame analysis. Properties of the ground motions are listed in Table 1. These ground motions are scaled so that their acceleration spectrum matches reasonably to that of the average of seven ground motion records on a stiff soil condition [15].

The response spectra of a typical ETEF produced by the above procedure are shown in Fig. 2. As this figure demonstrates, the resulting ETEF fits with the target spectrum in a reasonably well manner. The response spectrum of any window of the ETA20f set of acceleration functions from t_0=0 to t_1=t resembles that of the averaged response spectrum of the seven ground motions with a scale factor that is proportional with time (t) [16]. This scale factor is equal to 1.0 for t_{target}=10s in this study.

Table 1: Properties of selected ground motions

Record ID	Date	Event	Record Name	MAGNITUDE (Ms)	PGA (g)	Station Number
1	06/28 /92	Landers	LADSP 000	7.5	0.17	12149
2	10/17 /89	Loma Prieta	LPSTG 000	7.1	0.50	58065
3	10/17 /89	Loma Prieta	LPGIL0 67	7.1	0.36	47006
4	10/17 /89	Loma Prieta	LPLOB 000	7.1	0.44	558135
5	10/17 /89	Loma Prieta	LPAND 270	7.1	0.24	1652
6	04/24 /84	Morgan Hill	MHG06 090	6.1	0.29	57383
7	01/17 /94	Northridge	NROR R360	6.8	0.51	24278



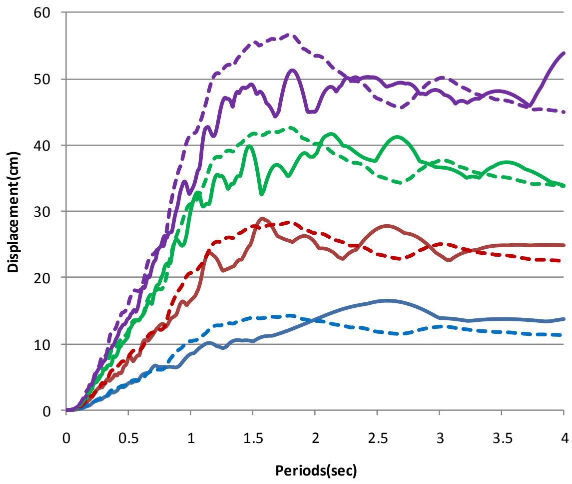
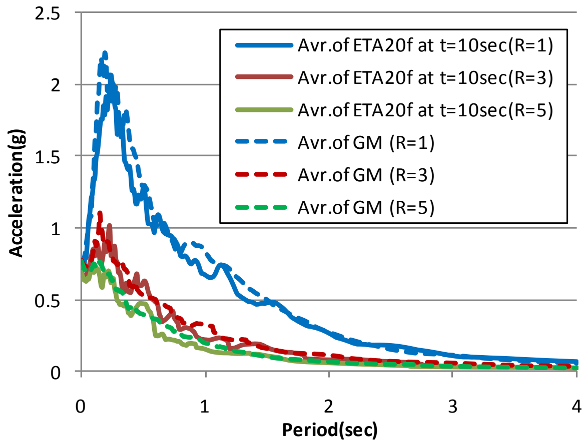


Fig. 2: Average response spectra of ETA20f set in comparison with ground motion set, Linear at 5th, 10th, 15th and 20th seconds

It may be a question as to why the 10th second is used for a target time. The ET acceleration functions and ground motions should be compatible with other characteristics such as the number of vibration cycles and strong motion duration since these parameters are among the most significant parameters in the nonlinear behavior of the structures [15,27,28,29]. The target time could influence the duration and frequency content of the records. Since earthquakes with larger magnitude actually have a longer duration, this characteristic of ET acceleration functions can make ET analysis results more compatible with the results of seismic analysis of ground motions. Undoubtedly, more research is required before passing any judgment on this issue. Basic studies show that the strong motion duration of the ETA20f set of acceleration functions up to t_{target}=10s is compatible with the average strong motion durations of the GM1 set of ground motions [15]. As described, changing 10 sec for target time during the calibration algorithm, may increase or decrease errors. This item needs further research for finding a solution that considers the duration and frequency content of each record, which is out of the scope of this paper. In this research, previously produced ET records, according to ref [15], are used for comparison.

Although the ETA20f set of ET acceleration functions is generated based on linear response spectra, its performance in estimating the nonlinear response of SDOF systems has been satisfying. Fig. 2 compares the average of the total acceleration and displacement response spectra of the ETA20f set at t=10s with the corresponding average spectra of the GM set for different strength ratios (R) [15]. As mentioned before, the ETAF goal is to estimate the average response of ground motions. In this study, the efficiency of this method is evaluated.



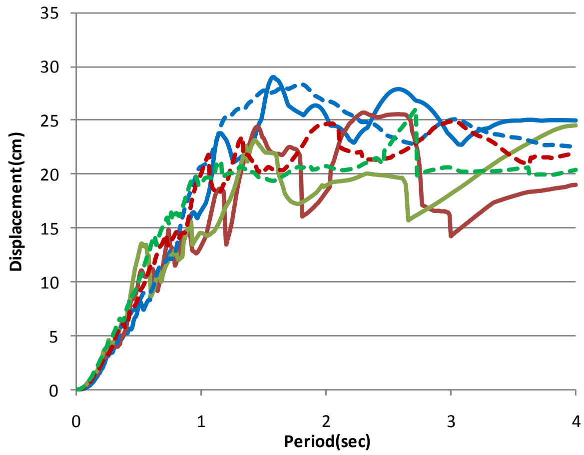


Fig. 3: Average response spectra of ETA20f set in comparison with ground motion set Nonlinear at 10th second

4. Estimation of damage indices

Structural parameters can reflect damages if their variations are calculated during an earthquake excitation. For example, plastic rotation or lateral displacement can be used as damage parameters. To quantify damages in a structure, a parameter called damage index is defined. This index takes a value between zero and one. Zero reveals no damage and one shows complete damage happening. According to used parameters in the damage index formula, a special analysis is needed. As the required analysis tends to a nonlinear dynamic method, it will become more complicated, and more time and cost will be needed for calculations.

In this research, Drift, Park-Ang [17] and Bozorgnia-Bertero [18] models are used as samples of damage indices. Structural drift is defined as ratio of maximum displacement to the story height. Park-Ang is a combined model which was first defined for RC material [17]. The form used in this paper is as given in Equation (4).

$$\mathrm{DI} = \frac{\phi_m - \phi_y}{\phi_u - \phi_y} + \frac{\beta_e\left(\int dE\right)}{M_y \phi_u} \tag{4}$$

Where ϕ_m is the maximum curvature in the member, ϕ_u is ultimate curvature capacity under static loading (assumed to be 20ϕ_y), ϕ_y is calculated energy (excluding potential energy), β_e is the coefficient for cyclic loading effect (a function of structural parameters), and M_y is yield moment of the element. Also, Bozorgnia and Bertero defined two improved damage indices [18]. For the special case of elastic-perfectly-plastic system, the two indices are as below:yield strength, dE is incremental absorbed hysteretic

$$\mu_H = \frac{E_H}{M_y \phi_y} + 1 \tag{5}$$

$$DI_1 = \frac{(1-\alpha_1)(\mu-\mu_e)}{\mu_{mon}-1} + \alpha_1 \frac{\mu_H - 1}{\mu_{Hmon}-1} \tag{6}$$

$$DI_2 = \frac{(1-\alpha_2)(\mu-\mu_e)}{\mu_{mon}-1} + \alpha_2 \left(\frac{\mu_H - 1}{\mu_{Hmon}-1}\right)^{\frac{1}{2}} \tag{7}$$

Where $\mu = u_{max}/u_y$ is displacement ductility and $\mu_e = u_{elastic}/u_y$ is maximum elastic portion of deformation which is equal to 1 for inelastic deformation and equal to μ if the response remains elastic.

Park-Ang and Bozorgnia damage indices are calculated for five integration points in each element. Maximum damage in these points is assumed as element damage. The damage model can be extended to the story and overall level by weighting the damage according to the dissipation energy as follows:

$$SDI_i = \sum_{K=1}^{m_i} \lambda_{ki}.DI_{ki}, \quad \lambda_{ki} = \frac{E_{ki}}{\sum_{k=1}^{m_i} E_{ki}} \tag{8}$$

In which SDI_i is the damage index of the i-th story, DI_{ki} is the damage index of the k-th element of the i-th story, E_{ki} is the hysteretic energy of the k-th element of the i-th story, $E_i = \sum_{k=1}^{mi} E_{ki}$ is the hysteretic energy of the i-th story, and m_i is the number of the elements of the i-th story. Also, the overall damage index is:

$$ODI = \sum_{i=1}^{N} \lambda_i.SDI_i, \quad \lambda_i = \frac{E_i}{\sum_{s=1}^{N} E_S} \tag{9}$$

Where ODI is the overall damage index $E_T = \sum_{s=1}^{N} E_s$ is the overall hysteretic energy, and N is the number of stories. Dissipated energy is calculated in each level of frames. The area between hysteretic loops of moment-curvature in each integration point is calculated. After that, the dissipated energy of the element is computed using equation (10). The last integral is evaluated using five points Gauss-Lobatto quadrature. Total story energy is calculated by summing the energy of all story elements.

$$E = \int M(x)\, d\theta = \int_L M\varphi\, dx = \int_L \left(\int_0^t M(x,t)\, d\varphi \right) dx \tag{10}$$

5. Model definition

For comparing results from ETEF and real ground motions, concrete moment frames with various bays and stories are considered as depicted in Fig. 4. Frames are selected so that they would cover a broad range of concrete frames. They are named based on abbreviation αS-βB where α is the number of stories, S stands for story, β is the number of bays and B stands for bay.

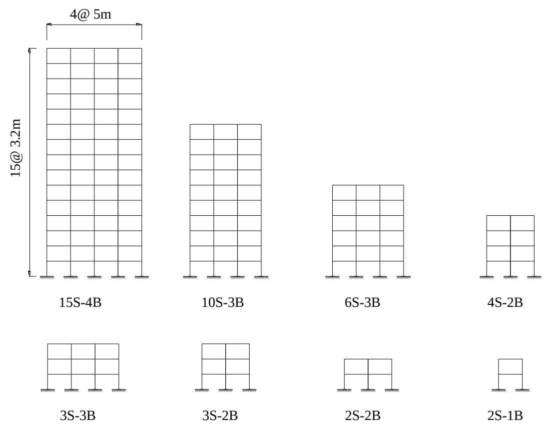
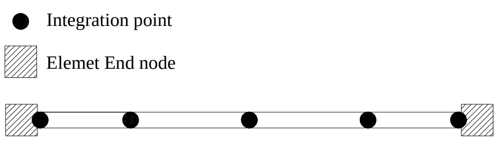


Fig. 4: Geometry of selected frames for this study

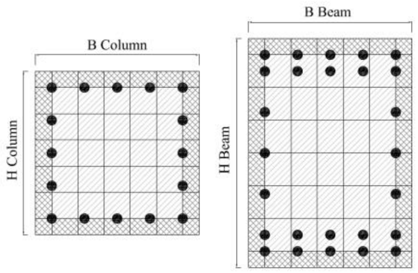
These frames are designed based on ACI318-05 [19] design code. All of the bay widths are 5 m and the story height of all frames is 3.2 m, which is common in buildings. Frames are designed considering a response reduction factor R of 8, corresponding to Special moment frames.

6. Structural Analysis

Analyses are performed in OPENSEES [20] which is a finite element open-source software for dynamic analysis of the structure. Beam and column are defined as force-based nonlinear beam-column elements that consider a spread of plasticity along the element length [21]. Five integration points with Gauss-Lobatto distribution along each element is considered as shown in Fig. 5-(a). To assess dissipated energy in the element, each section of the elements is divided into uniaxial fiber sections (Fig. 5-(b)). As in this research, we aim to compare results of ET with NTH methods in the same model, joint shear and reinforcement bond-slip are not considered in both analyses for simplicity.



(a)



(b)

Fig. 5: Element Modeling, (a) Distribution of integration point along the element, (b) Beam and column fiber section

It is interesting to note that the fiber discretization approach can consider simultaneous effects of axial load and flexural moment in a more accurate distribution of plasticity along the member length and cross sectional area [22]. Concrete material is modeled using Concrete02 objects in OPENSEES with tensile strength and linear tension softening. This material is based on the Kent-Scott-Park model [23], and strength and stiffness degradation are considered in the material model. Steel material is modeled by Steel02 material in OPENSEES, which is used to construct a uniaxial bilinear steel material object with strain hardening. Material definition parameters are depicted in Table 2 and monotonic envelope parameters of concrete and steel are shown in Fig. 6. Rayleigh damping model, taking the modal damping ratio as 5% for the first and the fourth modes, is considered in the model.

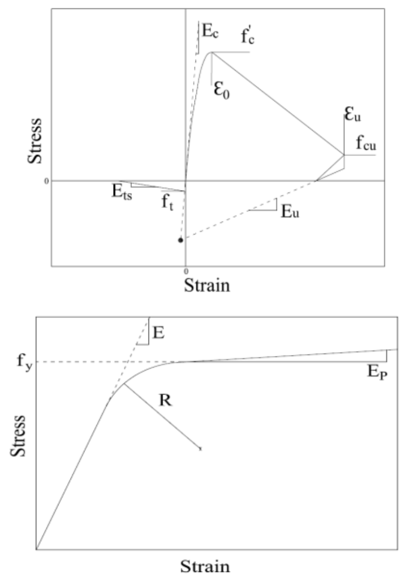


Fig. 6: Material parameters of monotonic envelopes of concrete and steel

Table 2: Material-defining parameters

Parameters	Value
-Concrete parameters	
f_c (Concrete Compressive Strength)	28(Mpa)
E_c (Concrete Young's Modulus)	$4700\sqrt{f_c'}$ (Mpa)
K_{fc} (ratio of confined to unconfined concrete section)	1.3
K_{res} (ratio of residual/ultimate to maximum stress)	0.2
f_{c1c} (confined concrete defined in mander model)	$K_{fc} \times f_c$
ε_{1c} (Strain at maximum stress)	$2\times f_{c1c}' / \varepsilon_{1c}$
f_{c2c} (ultimate stress)	$K_{res} \times f_{c1c}$
ε_{2c} (strain at ultimate stress)	$20\times\varepsilon_{1c}$
λ (ratio between unloading slope at ε_{2c} and initial slope E_c)	0.1
f_{c1u} (unconfined concrete maximum stress)	f_c
ε_{1cu} strain at maximum strength of unconfined concrete)	0.003
f_{c2u} (ultimate stress)	$K_{res} \times f_{c1u}$
ε_{2u} (strain at ultimate stress)	0.01
f_{tc}', f_{tu}' (tensile strength of confined and unconfined concrete)	$0.14\times f_{c1c}, 0.14\times f_{c1u}$
E_{ts} (tension softening stiffness)	$\frac{f_{tu}}{0.002}$
-Steel parameters	
F_y (steel yield stress)	400(Mpa)
E_s (Modulus of steel)	$2100000 kgf / cm^2$
B_s (Strain hardening ratio)	0.01
Parameters that control transition from elastic to plastic branches	$R_0 = 0.18$ $C_{R1} = 0.925$ $C_{R2} = 0.15$

7. Calibration of Earthquake accelerations

Seven earthquakes are selected from a group of twenty records used in FEMA 440 [24]. These records are for soil type C which is similar to soil type II in Iranian code 2800 [25]. According to ASCE7-05, the ground motions shall be scaled so that the average value of the 5 percent damped response spectra for the suite of motions is not less than the design response spectrum for the site for periods ranging from 0.2T to 1.5T, where T is the natural period of the structure in the fundamental mode for the direction of the response being analyzed [26]. So, for each frame, these records are scaled by a scale factor S_1 to match this design spectrum. The design response spectrum is selected to be soil type II in Iranian code 2800 [25]. Therefore, at t_{target}=10, response of 7 records match the design response spectrum. These records are applied to the structure at different hazard levels specified by scale S_2 from 0.1 to 2.5. As a result, IDA curve can be plotted using these data and by multiplying each natural record by its scale factor S_1, the average response of the records matches with the design response spectrum. Multiplying all the records by a scale factor S_2, we can identify the response for different intensities.

ET records are produced in a manner that at 10 seconds the response matches with the average response of seven records and are linearly scaled with the time. To compare results of each scale factor from NTH methods to correspond with the results from the ET method, the equivalent time in which the value of the target spectrum should be calculated is obtained from Equation (11) [4]. In this equation 10 is a constant that reflects the time ET response spectrum matches the target spectrum.

$$t_{eq} = 10 \times S_1 \times S_2 \tag{11}$$

8. Result Comparison

As mentioned before, each ET acceleration function contains the results of multilevel dynamic excitations. So Average IDA curve of seven records can be compared with the average response of three ETEFs. EDPs, which are used for this purpose, include normalized base shear, top story displacement, inter story drift, and total dissipated energy.

8.1 Analysis results comparison

Analysis results from IDA and ETEF are compared with each other using base shear, drift, roof displacement, and dissipated energy as EDPs. Base shear is calculated in each model for seven scaled earthquake accelerations and their average is compared with corresponding results from the ET method. Base shear is normalized by the weight of each frame. A typical IDA curve for base shear is depicted in Fig. 7 for the 6s-3b frame. It should be noted that the IDA scale factor S_2=1, indicates the acceleration response of an average of seven scaled records corresponding to design response Spectrum (t_{target}=10s). The results from other models are calculated similarly and acquired data is depicted in Fig. 8. As the results show, they match well with each other, and the coefficient of the trend line is near one.

Inter story drift is calculated using NTH and ET analysis. The result for model 15s-4b for two scale factors is shown in Fig. 9. The total comparison between drift results from NTH and ET method is shown in Fig. 10. Drift results for lower stories are estimated with more accuracy in comparison with upper stories. Roof maximum displacement is also calculated using the NTH and the ET method. As it is shown in Fig. 11, the results correlated well with each other. According to the results, average of results from three acceleration functions can be a good estimate for the response of structures subjected to the ground motions.

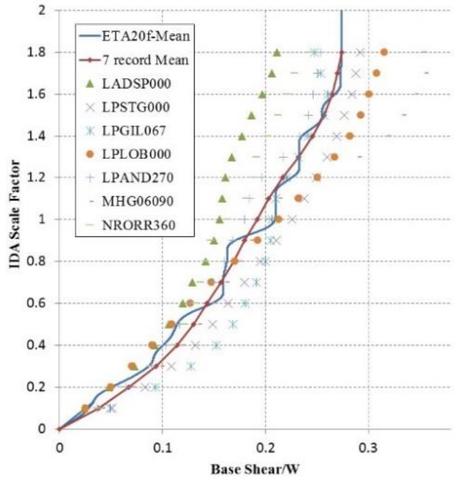


Fig. 7: Typical normalized base shear (model 6s-3b)

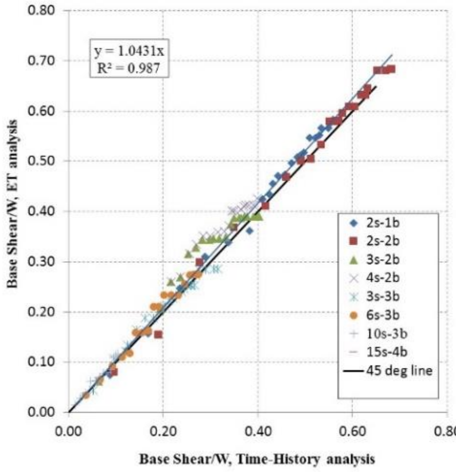


Fig. 8: Correlation of base shear results from NTH and ET methods

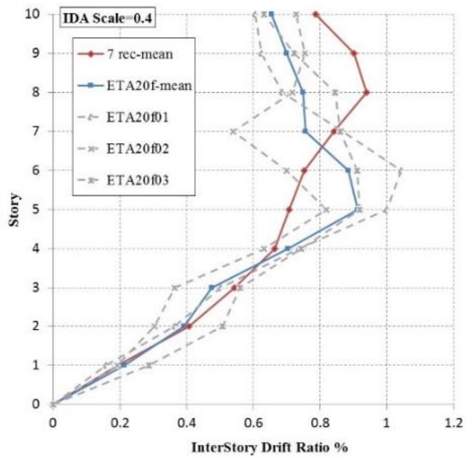


Fig. 9: Typical Inter story drift for different IDA scale factor by ET and NTH method (model 15s-4b)

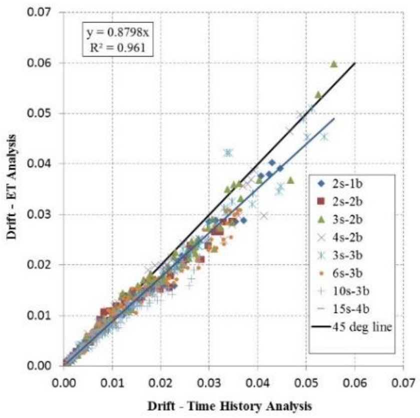


Fig. 10: Correlation of Inter Story drift results from NTH and ET methods

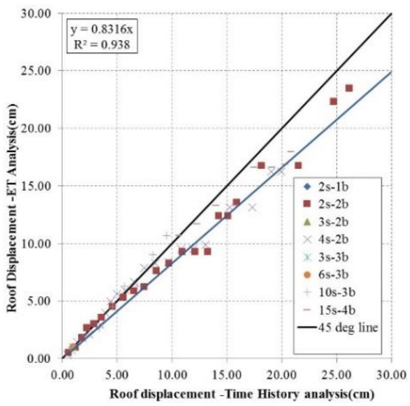
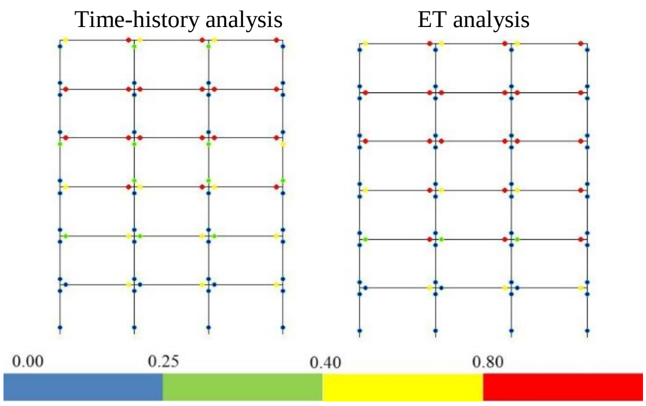


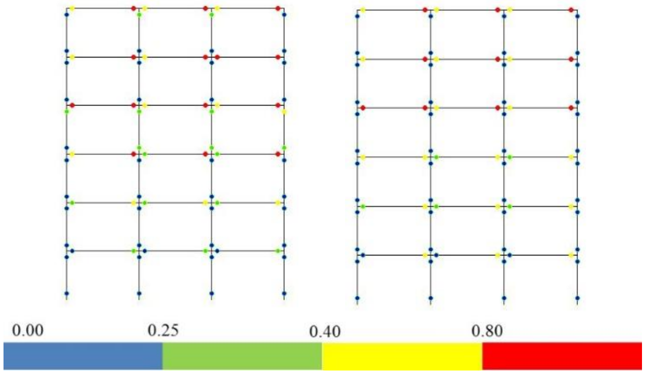
Fig. 11: Correlation of Roof Displacement results from NTH and ET methods

In the next step, Park-Ang and Bozorgnia-Bertero damage indices are calculated. Fig. 12 compares the local damage value obtained from ET and NTH analysis. The estimation of damage value in the beams and columns correlated well with each other for both damage indices.



(a)

Time-history analysis ET analysis



(b)

Fig. 12: local damage index results from time-history analysis and ET analysis for IDA scale factor equal to 1 (model 6s-3b) a) Park-Ang damage index b) Bozorgnia Damage index 1

Overall, Park-Ang damage index in an intensifying IDA analysis of sample 15s-4b model is depicted in Fig. 13. As it is seen, the damage index can be estimated well due to the proper evaluation of dissipated energy and element deformation.

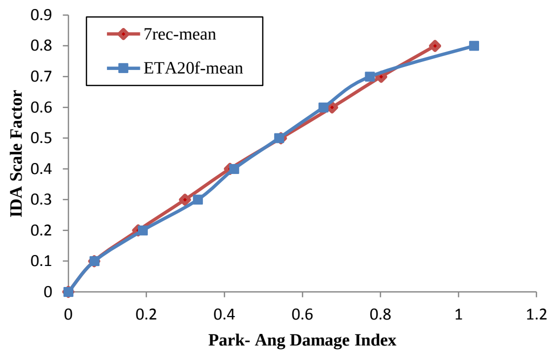


Fig. 13: overall Park-Ang damage index of 15-4b model for different IDA scale factor

In Fig. 14 an effort is made to find a relationship between Park-Ang and two Bozorgnia-Bertero damage indices. As it is seen, the r-squared value is near 1 which shows the fitness of the regression line to the data. Also, Park-Ang Damage index results are slightly higher in comparison with Bozorgnia-Bertero's two equations. Also, the correlation between two Bozorgnia-Bertero damage indices indicated that the equation damage index 1, equation (6), results in more damage to the structure.

8.2 Summary of results

Error in average response for each stated EDP is estimated using the response of each sample frame. Also, another important parameter is the divergence of three ETEFs results from the average of seven records. As the target of the ET method is to estimate NTH average response, it is important to know the value of response dispersion in the ET method. Therefore, the standard deviation of three ETEFs is calculated and the percentage of seven record averages, which are in ETEF response average plus one or two standard deviation ranges, are determined. Summary of results is cited in Table 3.

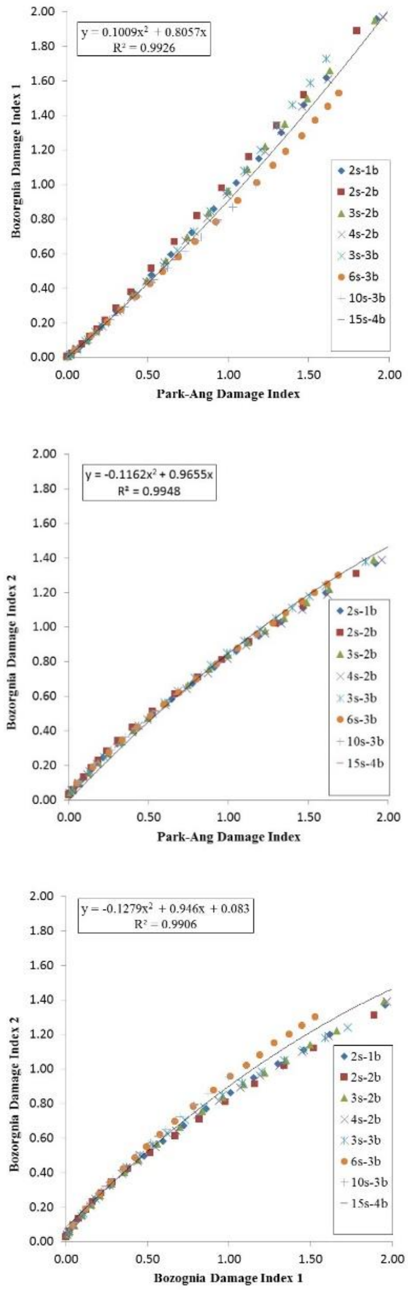


Fig. 14: relation between damage indices

According to the results, the error in the ET method is low and dispersion of records shows that this method estimates average NTH responses with great accuracy. In comparison with other EDPS, error in the estimation of dissipated energy is the highest. This has occurred because the energy term is not considered in optimization target, equation (3). As three ETEFs are produced with the same optimization processes, their input energies are almost the same and the standard deviation of dissipated energy is small. Therefore, the percentage of average NTH in the range of ETEF average plus one or two standard deviation(s) is low in comparison with other EDBs. Also, results from the Park-Ang damage index show that the estimation error is 13.55 %, which is an acceptable value.

Table 3: General EDPs result comparison

EDP	ET error in calculating average of seven records (%)	Percent of average NTH in the range of ETEF average $\pm 1\sigma$	Percent of average NTH in the range of ETEF average $\pm 2\sigma$
Normalized base shear	7.33	60.4	86.6
Inter story drift	13.22	63.51	83.8
Roof displacement	13.64	63.92	84.17
Dissipated energy	28.29	19.5	41.6

9. Summery and conclusion

In this paper, the application of the Endurance Time (ET) method, an intensifying dynamic analysis, in the nonlinear analysis of concrete moment frames was investigated. Sample models of concrete moment frames were considered, and stiffness and strength degradation were exerted. Seven earthquake accelerations were considered. By using incremental dynamic analysis (IDA), responses of selected records were compared with the response of the ET method at different excitation levels. Some engineering demand parameters (EDPs) i.e. base shear, roof displacement, and dissipated energy were used to compare the results. Furthermore, the accuracy of this method for estimating damages in structures was checked by evaluating inter story drift, Park-Ang, and Bozorgnia-Bertero as well-known damage indices.

A major question was how well ET excitation functions calibrated to match average seven earthquake response spectra in periods of 0.2 to 1.5 times the first period of the structure, can estimate the behavior of concrete moment frame, considering strength and stiffness degradation material model. Results show that NTH response for EDPs such as base shear, inter story drift, and roof displacement can be estimated by using this procedure with reasonable accuracy. As the response intensity increases, the method error increases due to the nonlinear response of the material and the different number of cycles in ETEFs in comparison with the average of seven records. As ET excitation functions (ETEFs) are produced by numerical optimization, energy content is not directly considered in the target function of optimization, and estimating this parameter indicates more errors in comparison with other EDPs.

Besides, results from sample damage indices show that ET responses are correlated well with NTH values and this method can estimate damage indices with acceptable accuracy. Furthermore, using the strength and stiffness degradation model for material shows that the ET method can respond well in the nonlinear region.

According to the results, the ET method can estimate NTH responses of concrete moment frames with good accuracy. As the number of analyses and consequently run time in this method is much less than those of NTH, engineers can use this method as an alternative to NTH analysis for seismic design and investigation of these types of structures.

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