ASIAN JOURNAL OF CIVIL ENGINEERING (BUILDING AND HOUSING) VOL. 10, NO. 5 (2009)
PAGES 559-577

ENDURANCE TIME METHOD: EXERCISE TEST APPLIED TO STRUCTURES

H.E. Estekanchi*, H.T. Riahi and A. Vafai
Department of Civil Engineering, Sharif University of Technology, Tehran, Iran

ABSTRACT

Endurance Time (ET) method is a new dynamic analysis procedure that aims at estimating seismic performance of structures by subjecting them to predesigned intensifying acceleration. These acceleration functions are designed in a way that they approximately simulate earthquakes with different intensities through their duration. Performance of the structure is judged based on its behavior for different intensity measures through time. A set of three acceleration functions have been produced using the ET concept by applying optimization techniques. These acceleration functions are designed in such a way that their response spectrum, while complying with typical code design spectra, intensify uniformly with their duration. Potential applications of this method are described by evaluating the performance of sample SDOF and MDOF systems. The pros and cons of ET method are investigated. The feasibility of application of ET method as a tool in performance based seismic analysis and design is discussed.

Keywords: Endurance time method, earthquake engineering, seismic design, time history analysis, design criteria, dynamic pushover

1. INTRODUCTION

The basic objective of earthquake engineering is to provide structures with appropriate safety margin against failure when subjected to earthquakes. The common philosophy of seismic design codes and standards is to achieve the dual goal of preventing structural failure at collapse level earthquakes and also keeping the non-structural damages to a minimum amount in case of service level earthquakes. According to Performance Based Earthquake Engineering (PBEE) philosophy, these goals can be extended to cover a broader range of design objectives [1].

Apparent limitations of simple seismic analysis procedure, along with significant progress in computational technology, have encouraged researchers in developing more precise and consistent analysis methods for seismic design. Development of various nonlinear static and dynamic procedures, such as pushover and incremental dynamic analysis (IDA), are examples of new developments in this area [2-9]. The concept of Performance Based Seismic

* E-mail address of the corresponding author: stkanchi@sharif.edu (H.E. Estekanchi)

Engineering (PBSE) is gaining increased interest among practitioners [10]. Pushover analysis is becoming a standard practice in design offices and nonlinear time history analysis is also gaining popularity [11]. As a result of these new developments, the structural engineer can incorporate significant sources of nonlinear behavior in the analytical model. Thus he can make a more conclusive judgment on different design choices.

In this research, a new dynamic analysis method for evaluating the seismic performance of structures is proposed. The primary objective of ET method is to provide an ideal input motion for dynamic analysis that can be used in assessment of the seismic performance of different structures. This ideal input motion is designed in a way that it simulates earthquakes with different intensities through its duration. The concept of ET method is similar to the exercise test used by cardiologists for assessing the condition of cardiovascular system of patients. ET method can be useful when comparing the relative performance of two different structures or design alternatives for the same structure.

The basic idea of ET method was originally introduced by Estekanchi *et al* [12] where heuristic method was used in order to produce the intensifying acceleration functions that could roughly meet the requirements of a successful ET analysis. Even though some applications of the ET method have been presented in available literature, the basic concepts of the method still deserve further explanation and discussion [13]. In this research the idea of ET method is first explained by a hypothetical shaking table experiment. The analogy between ET method and exercise test used by cardiologists is also discussed. Then, a numerical optimization procedure for generating ET acceleration functions that significantly improves the practical significance of ET acceleration functions is explained. To evaluate the capability of this method in assessing the performance of the structures three acceleration functions have been produced using this procedure. The response spectrum of these acceleration functions, while complying with typical code design spectra, intensify uniformly with their duration. Application of ET method is described by evaluating the performance of sample SDOF and MDOF systems. Use of different design criteria and damage indices in determination of endurance time has been explained. The pros and cons of ET method are explained. The feasibility of application of ET method as an effective tool for performance based seismic analysis and design of structures is discussed.

2. CONCEPTS

The idea behind the ET method can be described by a hypothetical experiment. Consider that two buildings are to be compared regarding their potential seismic resistance. A shaking table test is to be used for this purpose. Consider that representative models of the buildings are set on the shaking table. The experiment is started by applying a white noise type random vibration with small magnitude to the models. As the amplitude of the applied ground motion is gradually increased, the response amplitude of the models also increases as shown schematically in Figure 1. The structures gradually go through elastic to yielding and inelastic phases until a point is reached when the induced motions are beyond the ultimate resistance of one of the models, say, structure A, in this hypothetical experiment. At this point, structure A will fail as a result of the applied dynamic forces. As the experiment continues, the second structure will also fail at a later time. Now, based on the results of this hypothetical experiment, it is judged that Structure B, which endured longer and sustained higher levels of dynamic excitation, has a better performance as compared to structure A. Note that this judgment has been made purely based on the direct observation of dynamic response without any particular reference to the dynamic characteristics of the structures, such as period of natural vibration, mass, damping, structure system and material, etc.

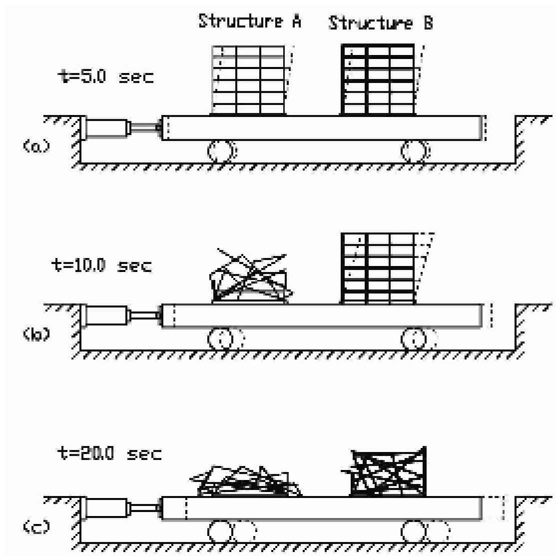

Figure 1. Hypothetical Shaking Table test

At the first glance, this judgment may seem to be too primitive and rough for any practical seismic assessment application. Different structures are vulnerable to different earthquakes with quite different characteristics. Thus, normally, structural engineers are not interested in how long a building can survive a particular earthquake but in fact they want to understand the performance of the structures subjected to different earthquakes. By the way, if a particular acceleration function can be made to include some major characteristics of real earthquakes, then it might be able to reveal some interesting differences in dynamic characteristics of different structures. The design objectives in current building codes address life safety, control damage in minor and moderate earthquakes, and prevent collapse in a major earthquake [14]. In ET method a particular ground motion, named acceleration function, tries to simulate minor, moderate and major earthquakes through its duration in a single record. Of course possibility of achieving this goal is the most important challenge of a successful ET analysis. ET acceleration function should be generated in a manner that it each time window of it presents major characteristics of earthquakes like the number of cycles and duration of them, their acceleration levels, etc. If an acceleration function conforming reasonably to these characteristics can be made, ranking of structures based on the results of ET analysis can be successful. Obviously, a single acceleration function cannot be made to fully implement all characteristics of a set of different earthquakes. The major question is how good a single acceleration function can be made to include the most significant aspect of intended earthquakes. In this paper, it is shown that by using numerically intensive optimization procedures, some very useful acceleration functions can actually be produced. As will be shown, some major seismic performance characteristics of different structures can be estimated with reasonable accuracy using these intensifying acceleration functions.

In the ET method, structures are rated according to the length of the time interval during which they can resist a calibrated intensifying acceleration function. This time duration is called Endurance Time and is in direct correlation with the magnitude of the applied dynamic excitation. Higher endurance time is interpreted as better performance and a shorter endurance time is interpreted as poor performance. A standard minimum endurance time is specified, based on the performance objective as the design criteria. Structures that fail to satisfy the performance objective before standard time are rejected while those that endure longer than the minimum time are considered to have satisfied the design criteria.

The concept of endurance time is very similar to the exercise test used by cardiologists in order to evaluate the condition of the cardiovascular systems of athletes and patients [15]. In this test, the subject who's general cardiovascular condition is to be evaluated is asked to run on a treadmill. The treadmill is set to the standard starting speed and slope. As the test continues, the speed and slope of the treadmill is increased as a predefined function of time. Vital biological measurements, such as blood pressure, heart beat rate and other physical parameters are continuously monitored and the test is commenced until abnormal conditions are observed or the patient is exhausted. From the total running time, an estimate of the patient's condition can be made.

The interesting point about the exercise test is how a relatively simple procedure is used for assessing the condition of a complex system. In ET method, it is intended to use a conceptually similar procedure for measuring the fitness of structures to endure the dynamic demands imposed on them by earthquakes. In order to achieve consistent and meaningful quantitative results, ET acceleration functions should be appropriately generated and calibrated. This will be discussed in the next section.

The endurance criterion is also an important consideration. In the hypothetical shaking table test, the endurance measure was the stability of the structure, i.e. the endurance time was measured up to the apparent collapse time for the structure. Even though collapse can be considered as an ultimate measure of endurance, it is not necessary to base the endurance time measurement on complete collapse. Depending on the model specifications, performance level requirements and other experimental or analytical considerations, different endurance criteria can be applied. For example, in the simple case of linear analysis, the design stress ratio of structure members and maximum story drift ratios, which are routinely used as design criteria, can be considered as the endurance criteria. These are the basic codified criteria for seismic design and their application will be demonstrated in the next sections. For nonlinear analysis, various criteria, such as plastic hinge rotations or damage indexes, can be used. For experimental implementations, collapse can be considered as a criterion for extreme performance levels. Evidently, different performance levels can dictate the usage of different endurance criteria. The monitoring of structural behavior does not need to be limited to only one criterion. Multiple criteria can be specified and endurance time can be measured, based on the most critical limit states or on a combination of various parameters and indices that are observed during the analysis or experiment.

The structural model is another fundamental requirement for ET analysis. Potentially, ET method can be equally well applied in experimental and analytical setups. Since the ET method is based on time history analysis, there is no apparent limit for consideration of the complex geometric or material behaviors such as buckling, material degradation, cyclic behavior, loss of stability, yielding, fracture, energy losses and cracking, etc. The concept is equally applicable to simple and complex models, ranging from ordinary moment frames to complicated systems with active control or other seismic mitigation devices. It will be up to the analyst to make sure that the proposed model adequately implements the special features of the intended actual prototype.

3. ET ACCELERATION FUNCTIONS

A key factor in the successful implementation of the ET method lies in the characteristics of the intensifying acceleration functions that are used as dynamic input. ET acceleration functions should be defined so that maximum consistency between ET analysis results and actual seismic performance can be achieved. However, this criterion cannot be readily employed in generating the acceleration functions because of the fact that the definition and determination of actual seismic performance is still under dispute itself. Additionally, gathering all major characteristics of real ground motions in a single acceleration function is not possible.

In order to simplify things and reach a practical initial solution, it is rational to start from a proven design criterion, which has already been implemented in seismic design codes. A key concept in the codified design procedure is the use of response spectrum in determination of expected peak accelerations for seismic design. If ET acceleration functions are to be used in order to predict structural performance under earthquake induced excitations, then the response induced by them should correspond to those induced by real earthquakes. An important difference between ET acceleration functions and earthquake accelerograms is in that ET acceleration functions are intensifying accelerograms with theoretically infinite duration [16]. Thus, the definition of spectral response should be interpreted in a special way. In the ET method, spectral maximum relative displacement ($S_u(T,t)$) and absolute acceleration ($S_a(T,t)$) at time t are defined as the peak value of these variables in time-history for a time interval from $\tau = 0$ to $\tau = t$. Spectral relative displacement and spectral absolute acceleration in ET analysis are thus defined as follows:

$$\begin{aligned} S_u(T,t) &= \max(|u(\tau)|) \quad \tau \in [0,t] \\ S_a(T,t) &= \max(|a(\tau)|) \quad \tau \in [0,t] \end{aligned} \tag{1}$$

Spectral responses for other parameters are also defined in a similar manner. A major difference between an ET acceleration function and an earthquake accelerogram is that in an ET acceleration function, time is used as a significant dimension that is related to the amplitude of excitation, while, in ordinary accelerograms, time is only an independent variable that defines ordinate of acceleration data points and is not directly involved in the evaluation of analysis results. Based on the above discussion, spectral response acceleration and displacement will be considered as the intensity measure in this preliminary implementation of ET procedure. ET acceleration functions will be designed so that at some predefined time, t_{Target}, their response spectrum reaches the codified value corresponding to the sought performance level. Because of the intensifying trend of ET acceleration functions, their response spectra should also increase through time. It means that at t < t_{Target} their response spectra should be less than the codified value and at t > t_{Target} their response spectra should be more than the codified value. The term of acceleration function is used here to distinguish it from synthetic accelerograms that are compatible with a predefined spectrum and do not have an increasing pattern [17]. In this study, t_{Target} has been set to 10 seconds, so that effective duration of ET acceleration functions is approximately compatible to strong ground motions for design level earthquakes. A question now arises about how the intensification should evolve with time. Based on engineering judgment and until more research has been done, a linear function will be used to define the evolution profile of the acceleration and displacement response spectrum function at this preliminary stage. Optimal intensification profiles are to be investigated in a separate study. Thus, target response displacement and acceleration are defined as follows:

$$
\begin{aligned}
S_{aC}(T,t) &= \frac{t}{t_{Target}} S_{aC}(T) \\
S_{uC}(T,t) &= \frac{t}{t_{Target}} S_{aC}(T) \times \frac{T^2}{4\pi^2}
\end{aligned}
\qquad (2)
$$

The objective is to find the ground acceleration function, $a_g(t)$, such that the response at all times is as close as possible to the target values defined by Eqns 2. Analytical approaches to find acceleration functions that satisfy conditions such as Eqns 2 are formidably complicated [18]. The problem has been formulated as an optimization problem and a numerical approach has been used for the purpose of generating the acceleration functions in this research. This problem can be formulated as an unconstrained optimization problem as follows:

$$
\text{Minimize } F(a_g) = \int_0^{T_{\max}} \int_0^{t_{\max}} \{[S_a(T,t) - S_{aC}(T,t)]^2 + a\,[S_u(T,t) - S_{uC}(T,t)]^2\}\, dt\, dT \qquad (3)
$$

In which a_g is the ET acceleration function to be calculated and a is a weight factor for relative error of displacements and accelerations.

For numerical determination of the target response values, the response spectrum of the Iranian National Building Code (INBC) standard 2800 (BHRC 2005), considering soil type II (relatively stiff soil, V_s=375~750 m/s, T_0=0.1 sec, T_s=0.5 sec, S=2.5) and a very high seismicity area (A=0.35g), has been used for the purpose of illustration [19]. The design response spectrum can be described by Eqns 4. Importance factor (I) and response reduction factor (R) have not been applied (assumed equal to 1.0) in generating the acceleration functions. These factors can be applied more conveniently during analysis time, based on each specific case. Response histories for SDOF systems, $S_a(T,t)$ and $S_u(T,t)$, have been calculated using a direct integration method.

$$\begin{cases} B = 1 + S(\frac{T}{T_0}) & T < T_0 \\ B = 1 + S & T_0 \le T < T_s \\ B = (1+S)(\frac{T_s}{T})^{\frac{2}{3}} & T_s \le T \end{cases} \tag{4}$$

$$S_{aC} = \frac{ABI}{R}$$

In order to proceed with the optimization problem, a set of first generation ET acceleration functions proposed by Estekanchi ***et. al.*** [12] is used as the initial starting point. This set of acceleration functions was produced from a random series with Gaussian distribution with zero mean value and a variance of unity that was filtered in the frequency domain using a Kanai/Tajimi filter function and then scaled using a linear profile function. The number of acceleration series elements is 2048 (i.e. 2^{11}) [20]. Considering a time step of 0.01 seconds, this defines an acceleration function up to about 20 seconds. An unconstrained linear optimization procedure is applied in order to solve the optimization problem, as defined in Eqn 3. Acceleration data points have been considered as the variables and the problem is solved considering 200 different periods distributed in the range of 0.0 to 5.0 seconds using more densely spaced points in the low period region where higher accuracy is desirable. The objective of optimization has been to fit the acceleration and displacement response spectra to target design spectrum values defined by Eqns 2. The problem is highly demanding considering the computational effort and each acceleration function takes about 120 hours on a Pentium IV processor to complete 200 iteration cycles. Three acceleration functions have been generated using the above mentioned procedure. These are named ETA20a01-03, respectively. Resulted acceleration functions are depicted in Figure 2.

Typical convergence histories are depicted in Figure 3 using a logarithmic scale. It does not seem to be possible to improve the acceleration functions by continuing the iteration cycles using the current procedure. However, the theoretical minimum of the target function is an open question and is the subject of ongoing research.

The response spectra of all acceleration functions at target time (t = 10 sec) are depicted in Figure 4. The response spectra of the ETA20a01 acceleration function at specified times are depicted in Figure 5. It is interesting to note how the response evolves with time, i.e. the profile of responses increases in a linear manner, as required by Eqns 2. As can be seen in Figure 5, the convergence of solutions in the optimization problem has been quite satisfactory and the response spectra intensify in a uniform manner with time as expected.

Considering the relatively intensive computational effort required in producing appropriate ET acceleration functions, a question may arise on whether the effort is worthwhile. It should be noted however that once the ET acceleration functions have been generated for a given response spectra, they can be used in all cases where similar response spectra can be considered as valid, thus resulting in considerable savings on time and computational effort. In cases where appropriate ET acceleration functions are not readily available, the extra effort required to generate ET acceleration functions can be justified only if the number of design alternatives that need to be compared is very high [21].

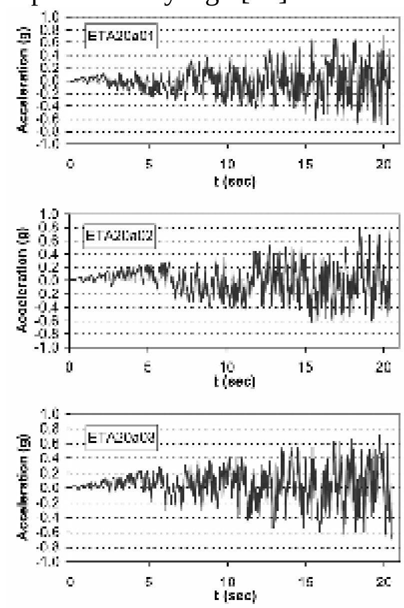

Figure 2. ETA20a01-03 acceleration functions

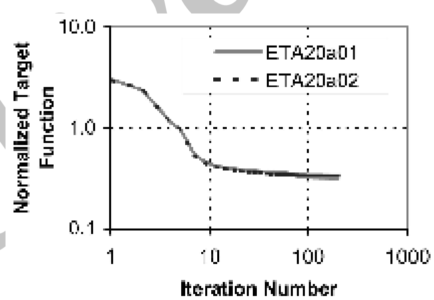

Figure 3. Typical convergence histories

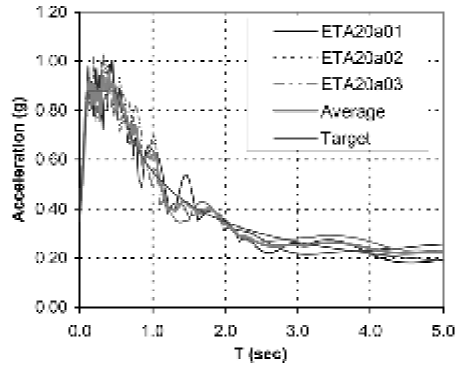

Figure 4. Acceleration response of ETA20a01-03 at target time (t = 10 sec)

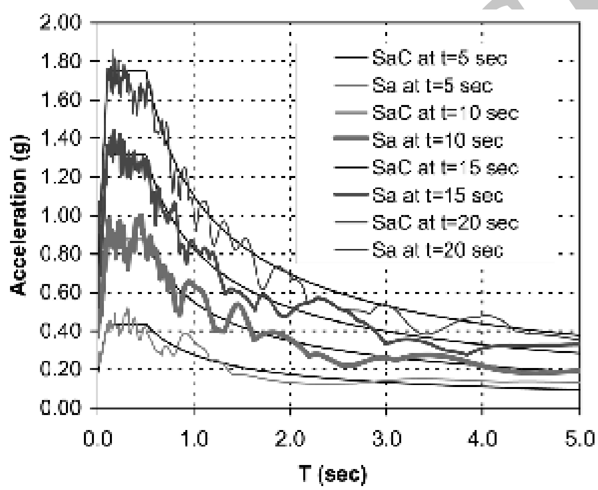

Figure 1. Acceleration response at different times for ETA20a01

4. APPLICATION TO SDOF SYSTEMS

Acceleration response histories for SDOF systems with various periods of vibration, subjected to the ETA20a01 acceleration function, are depicted in Figure 6. It should be noted that in the curves resulted from ET analysis, the maximum absolute value of the required parameter in the time interval [0, t_{cur}] is calculated and defined as the parameter value at $t=t_{cur}$. As expected, the maximum acceleration that the structure experiences, is an increasing function with time. The acceleration response satisfactorily follows the target line corresponding to the codified values that are scaled with time (Eqns 2). The error margin is quite satisfactory considering the nature of time-history earthquake analysis. In order to improve the accuracy of the analysis results, several approaches can be followed. One is to improve the quality of acceleration functions by making them better fit the target values. Another method is to use the statistical average of the results obtained from several acceleration functions. In this research, the average of the results from three acceleration functions is used. This procedure has shown to considerably improve the accuracy of the results and will be used in the rest of the analyses. However, more research is required in order to recommend the exact number of needed acceleration functions to reach a desired accuracy, based on the problem. Using an appropriate fitting curve can also be considered an effective method to obtain more refined results and better accuracy in extracting ET values from graphs. In the case of linear analysis, a line passing through the origin of the coordinate system can be used for this purpose. As can be expected, this line fits with the target curve quite closely. In practice, a combination of the above mentioned methods should be used to achieve the best possible accuracy. The results of averaging for three acceleration functions are shown in Figure 7. As can be seen in this figure, the average curve follows the target acceleration curve, *SaC*, much more closely. Average displacement response histories for T = 0.5 and T = 1.0 are depicted in Figure 8. It can be seen that displacement responses, in general, follow the target curve with good accuracy.

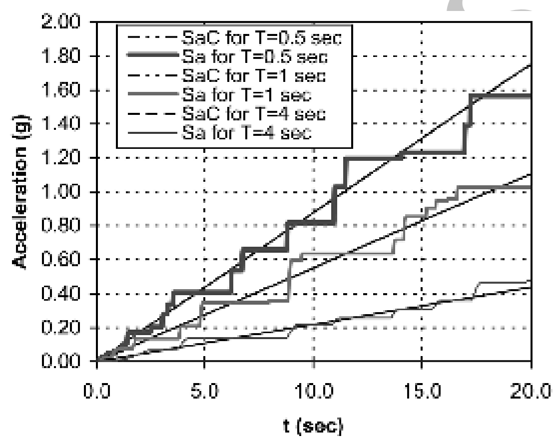

Figure 6. Acceleration response history for ETA20a01

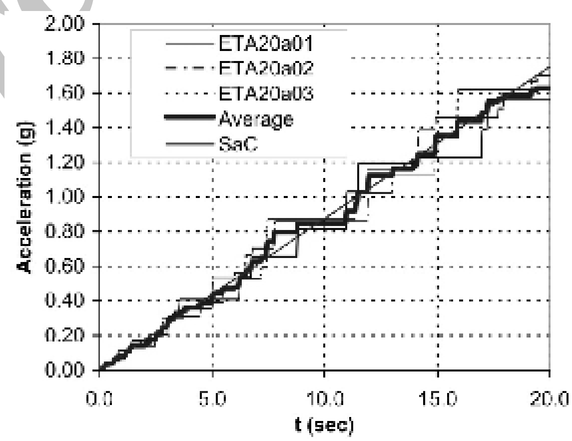

Figure 7. Acceleration response history for T=0.5 sec

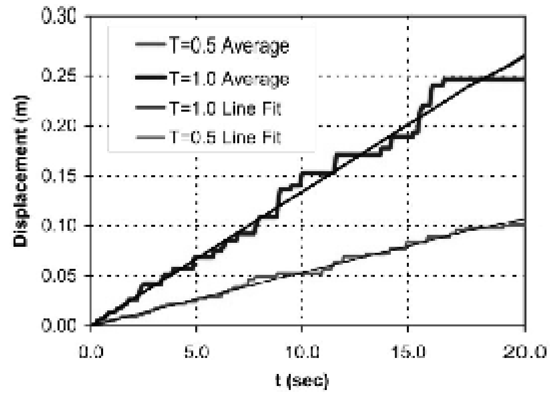

Figure 8. Displacement response history for ETA20a01

5. APPLICATION TO MDOF SYSTEMS

For simplicity, application of the ET method in seismic analysis of MDOF systems will be explained by studying the dynamic response of three-story steel moment frames. Three model frames have been designed according to the AISC-ASD design code. Frame geometry and section properties are depicted in Figure 9. Frame f2D2A3s1b (*T*= 0.8644) has been designed according to the recommendations of the INBC for a high seismicity area. Frame f2D2A3s1b-Weak (*T*= 1.0886) has been designed assuming one half of the codified base shear as the design lateral load. The last frame, f2D2A3s1b-Strong (*T*= 0.59), has been designed for twice the standard lateral load. Total mass of all frames has been assumed to be equal for ease of comparison. All three frames have been subjected to the three ET acceleration functions (ETA20a01-3). In linear analysis, a response modification factor (***R***) of 6.0 has been applied to elastic spectrum, as specified by the code for this type of frame, and ET acceleration functions have been divided by this factor accordingly. In order to reduce the statistical deviations, the average of three acceleration functions has been used in this investigation.

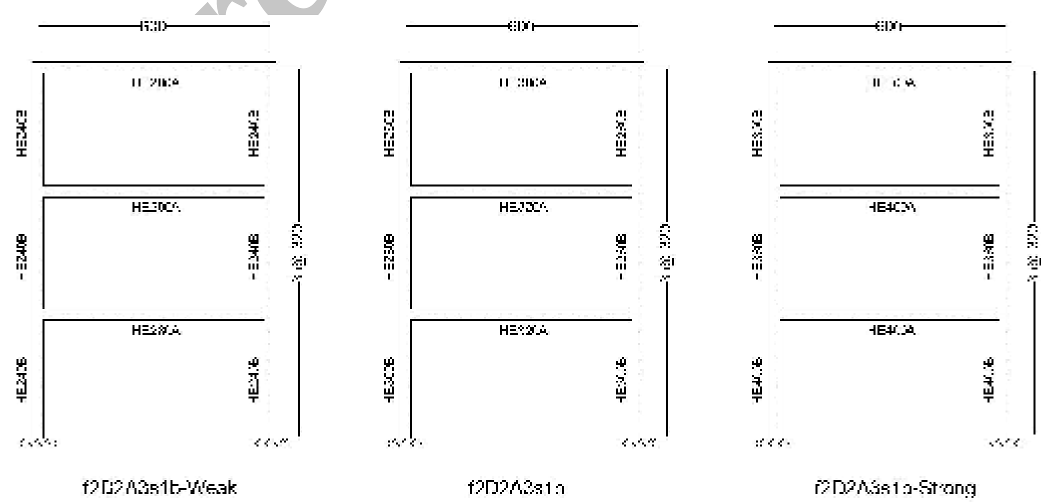

Figure 9. Three story steel moment frames

The average interstory drift for all frames has been depicted in Figure 9. A linear fitting curve has also been used for better visualization of the results. If one were to use maximum drift as the sole design criteria, then, based on the results shown in Figure 10, one would conclude that frame f2D2A3s1b-Strong is the best performer and has achieved the highest endurance time. On the other hand, frame f2D2A3s1b-Weak has the least favorable behavior. Moreover, if one assumes a drift limit at 0.016m (0.005*3.2), as specified by the code for this type of structure, and considering that the acceleration functions have been calibrated to comply with a codified level of excitation at t = 10 sec, one can see that frames f2D2A3s1b and f2D2A3s1b-Strong pass the criteria at t = 10 sec with calculated drifts of 0.015 and 0.007, respectively, but, frame f2D2A3s1b-Weak fails the criteria with a drift of 0.023 at t = 10 sec. Another way to interpret the results shown on this figure is to consider the time when maximum drift criteria of 0.016m is exceeded. In this way, it can be concluded that the endurance time of frames are approximately 4.5, 11.0 and more than 20 for weak, standard and strong frames, respectively.

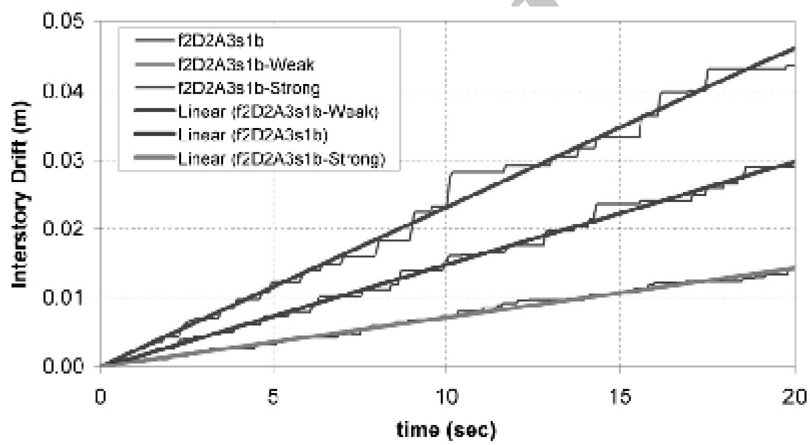

Figure 10. Maximum average inter-story drift in linear analysis

Obviously, interstory drift cannot be indicative of a strong or weak structure on its own. A structure will fail because some earthquake will expose a weak mechanism in the structure. It is important to note that the choice of Engineering Demand Parameters (EDP) depends on the performance target, and that the emphasis should be on the complete structural, nonstructural, and content system and not on a component. For instance, if the issue is global collapse, then the maximum story drift over the height of the structure is an appropriate EDP. For structural damage, local parameters such as shear distortions in joints and rotations at plastic hinges may be most relevant [22]. In most cases, these local parameters can be concluded from story drifts [23]. Because ET analysis is a time-history analysis different EDPs can be calculated through time easily. Therefore the performance of the structures can be judged for combination of different EDPs and effects of different damages can be seen for selecting the better performer and defining the endurance time.

A question may arise now on justification of applying a time-history analysis in the simple case of a linear three story steel moment frame. It should be noted that these simple examples have been provided in order to explain the concept and merit of ET procedure. The main advantage of a time-history based analysis procedure is in its potential for application in cases where static and modal analysis procedures cannot be readily applied. These include cases that involve different sources of nonlinear behavior such as geometric and material nonlinearity and even the linear cases in which the effect of damping distribution is considered to be significant. However, investigation of the potential application of ET method in these cases is beyond the scope of this preliminary work.

6. NONLINEAR ANALYSIS

The concept of ET method can be readily applied in the case of nonlinear analysis without much significant complication [24]. Optimizing ET acceleration functions to be applied in nonlinear range is a formidably complicated problem. However, in order to explain the potential use and merits of this method, the same acceleration functions that were developed in previous section are applied in some nonlinear cases for the matter of explanation. Nonlinear models of the same frames as used in the previous section are prepared by using the OPENSEES beam element with nonlinear distributed plasticity [25]. P-Δ effects have been included in the nonlinear analysis. The R factor has been assumed equal to 1.0 (elastic spectrum) and a viscous damping of 5% as customary for these types of frames has been applied in the analyses. ET analysis can be considered as a type of dynamic pushover procedure, where the applied acceleration function is an increasing function of time. Pushover curves resulted from standard static pushover analysis have been depicted along with the corresponding ET analysis results in Figure 11. A modal pattern proportional to the vertical distribution of the pseudo lateral load is used in static pushover analysis. The maximum base shear of the frames is consistent with their design criteria in both kinds of analysis, i.e. the strong frame, which has been designed for a higher level of lateral load, also shows a higher ultimate lateral strength. It is interesting to note that the results of the ET analysis closely follow the static pushover curves. The ET analysis results are slightly higher than the static pushover results, due to the effect of forces associated with viscosity in dynamic analysis, the effect of lateral force distribution used in the static pushover analysis, etc. It should be noted that data points for ET results in Figure 11 do not necessarily correspond to the value of the maximum base shear and the maximum roof displacement at the same time and are a profile of the results.

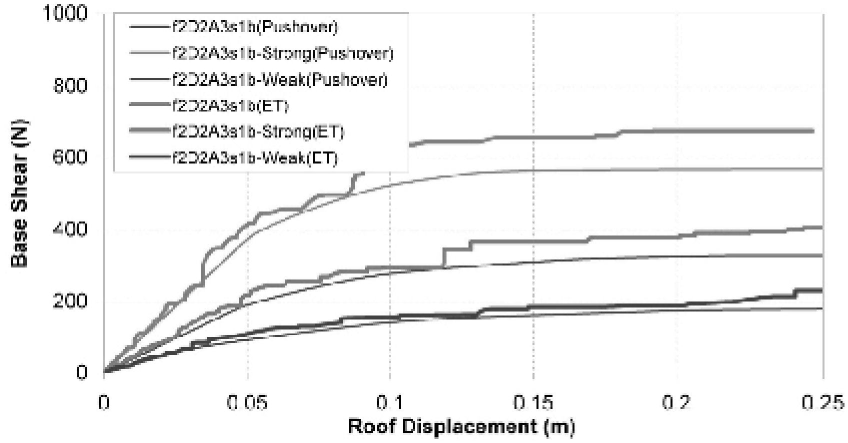

Figure 11. Base shear vs. roof displacement

Maximum average interstory drift of all the stories for nonlinear models has been shown in Figure 12. Since the response modification factor is considered to be 1.0 in these analyses and, considering the traditional safety factor of 5/3 used in ASD design procedure for steel structures, it is roughly estimated that the standard frame will experience nonlinear deformations from about t = 3 sec (t_{target}/R*FS=10/6*(5/3)≈3). The weak frame obviously experiences significantly higher drifts after t=6 seconds and becomes unstable after t = 10 seconds. The analysis did not converge after t = 15 sec for the weak frame. By setting appropriate drift limits in the nonlinear range, the adequacy of each frame to meet design criteria can be checked. It should be noted that any response parameter other than drift, such as various damage indexes or EDPs, can also be plotted against time and used as an evaluation criteria. The endurance time can be defined, based on the analysis of any predefined set of response parameters that the analyst considers significant in each analysis case. Standard sets of design criteria and limiting values can be developed in order to establish codified versions of the ET method.

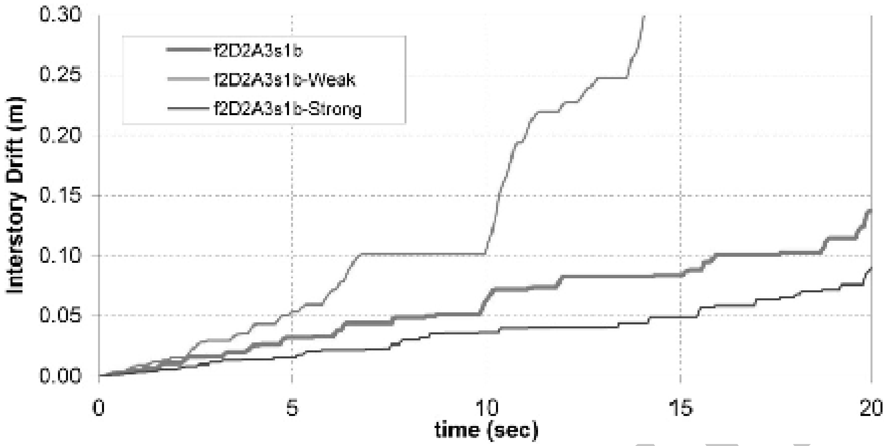

Figure 12. Maximum average inter-story drift in nonlinear analysis

ET method has many common grounds with the IDA method. In IDA, a certain earthquake accelerogram is applied to a structural model at different intensities and the results of multiple time-history analyses are used to evaluate the structure. A major difference between ET and IDA is that in the ET method, intensity is increased in a one step time-history analysis, thus, reducing the number of required analyses. This reduction of the number of analyses is expected to be accompanied by losses of accuracy to some extent. The successful implementation of ET method to be usable in the nonlinear range depends on goodness of fit of ET acceleration functions that can be achieved by calibrating and optimizing them against nonlinear response spectra. In order to conceptually compare the results from ET analysis and IDA analysis, the weak frame, f2D2A3s1b-Weak, that experiences more nonlinearity, has been subjected to two accelerograms from Loma Prieta 1989 earthquake (LPAND270 and LPGIL067) and one from Northridge 1994 earthquake (NRORR360) at various intensities. The response spectra of these accelerograms have been compared with the design spectrum of INBC in Figure 13. The 5% damped first-mode spectral acceleration $S_a(T_1,5\%)$ is plotted against maximum interstory drift ratio in Figure 14. As it could be guessed in low values of intensity measures, when the structure remains linear, the responses of ET method and IDA method are the same. By increasing intensity measure the structure goes through elastic to yielding and nonlinear inelastic, finally leading to global dynamic instability. In these stages the general trend of ET curve is similar to the IDA curve but the onset of each stage and the value of engineering demand parameter (here maximum interstory drift ratio θ_{max}) are different. The extraordinary variability from record to record of the forms and amplitudes of the IDA curves for a single building is reported before and this phenomenon can be found in ET curves too [9,22]. The deterministic of a nonlinear structural system under irregular input present a challenge to researchers to understand, categorize and possibly predict. This variability also leads to the need for statistical treatment of ET curves in order to summarize the results and in order to use them effectively in a predictive mode. As it is clear in Figure 13 the design spectrum of INBC in the long period range is extremely higher than other spectra. As a result, ET analysis becomes unstable at a relatively lower intensity measure. It should be mentioned that by increasing time, the strong motion duration of the ET acceleration function is increased but it is essentially constant for real ground motions in IDA for different levels of IM. Other characteristics of real ground motions that influence the response of structures in nonlinear ranges should be incorporated into ET method to make its results more reliable. Improvement of ET acceleration functions for application in the nonlinear range is under investigation [26]. The aim is to include characteristics of nonlinear response spectra from real earthquakes in the ET acceleration functions optimization procedure. Therefore it is possible to achieve maximum consistency between RHA and ET analysis results in the nonlinear range and provide a better level of approximation.

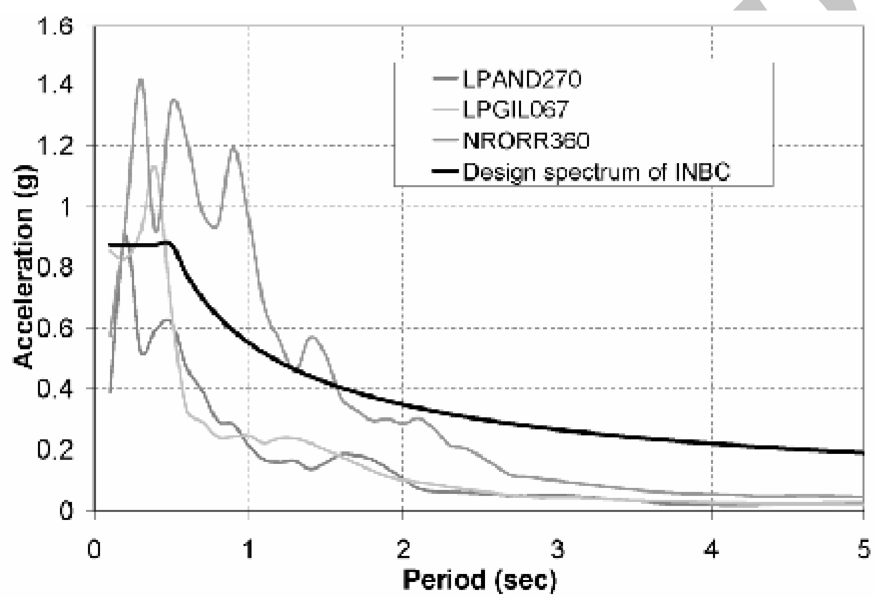

Figure 13. Response spectra of 3 earthquakes and INBC code spectra

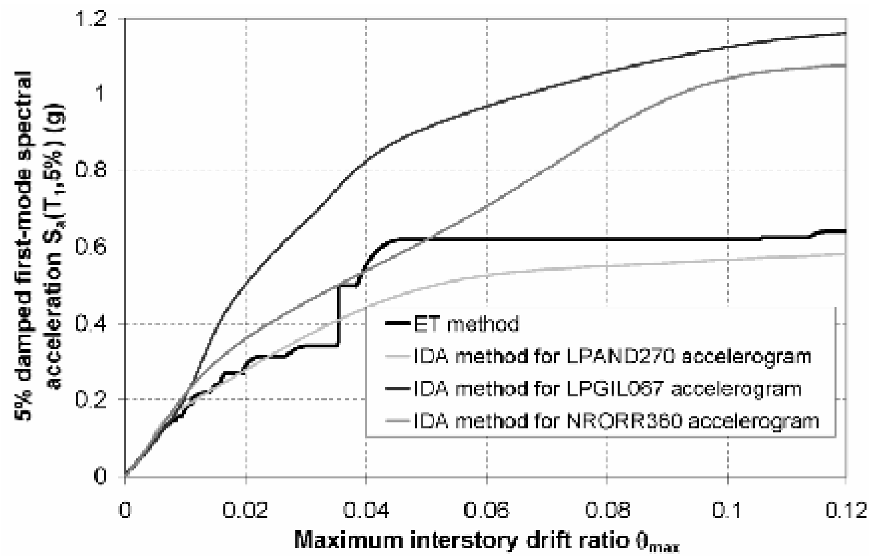

Figure 2. Comparison IDA and ET analysis results

7. CONCLUSIONS

Endurance Time (ET) method is a new dynamic analysis procedure for seismic evaluation of structures. In this method, structures are subjected to a gradually intensifying acceleration function and their performance is judged based on the time interval during which they can endure the imposed intensifying dynamic demand. The basic concept of the ET method is very similar to the exercise test used by cardiologists for assessing the condition of heart patients and athletes. The primary objective of ET method is to provide a comprehensible time-history based dynamic analysis procedure, analogues to the exercise test, which can be used in assessment of the seismic performance of different structures or design alternatives on a comparative basis.

A set of three intensifying acceleration functions has been produced, in which the resulting response spectrum, while complying with a typical codified design spectrum, increases proportionally with time. Numerical optimization techniques have been used, in order to minimize the divergence of the response spectrum from target values, by considering each acceleration function data point as an independent variable. The convergence of the solutions has been quite satisfactory and the resulting response spectra follow the concept of uniform intensification with time within acceptable tolerances.

The application of ET acceleration functions in SDOF systems shows consistent results with the equivalent static procedure. Application of different design criteria in ET analysis has been explained. The ET method has also been applied to MDOF systems in the linear analysis range. The capability of the method in differentiating MDOF systems with different design parameters has been explained using simple three-story shear buildings. Potential application of the ET method in nonlinear analysis has also been investigated by simple examples.

It can be concluded that the ET method has a good potential for consideration as an alternate dynamic pushover analysis procedure in cases where time-history based analysis is the most appropriate method, considering various sources of complicated nonlinear behavior. However, further research is required before this procedure can be reliably applied in practical applications involving highly nonlinear material and large deformations.

Acknowledgement: The authors would like to thank the Sharif University of Technology Research Council, and the Structures and Earthquake Engineering Center of Excellence (SECE) for their support of this research.

REFERENCES

1. Bozorgnia Y, Bertero VV. Earthquake Engineering: from engineering seismology to performance-based engineering, CRC Press, USA, 2004.
2. Fajfar P, Gaspersic P. The N2 method for the seismic damage analysis for RC buildings, *Earthquake Engineering and Structural Dynamics*, No. 1, **25**(1996) 23–67.
3. Krawinkler H. Seneviratna GDPK. Pros and cons of a pushover analysis of seismic performance evaluation, *Engineering Structures*, No. 4, **20**(1998) 452–64.
4. Miranda E. Inelastic displacement ratios for structures on firm sites, *Journal of Structural Engineering*, **126,** 10 (2000), 1150–9.
5. Gupta B, Kunnath SK. Adaptive spectra-based pushover procedure for seismic evaluation of structures, *Earthquake Spectra*, No. 2, 16 (2000) 367–91.
6. Mwafy AM, Elnashai AS. Static pushover versus dynamic collapse analysis of RC buildings, *Engineering Structures*, **23** (2001) 407-24.
7. Chopra AK, Goel RK. A modal pushover analysis procedure to estimate seismic demands for buildings: Summary and Evaluation, *Fifth national conference on Earthquake Engineering*, Istanbul, Turkey, May 2003, pp. 26-30.
8. Aydinoğlu NM. An incremental response spectrum analysis procedure based on inelastic spectral displacements for multi-mode seismic performance evaluation, *Bulletin of Earthquake Engineering*, **1**(2003) 3-36.
9. Vamvatsikos D, Cornell C.A. Direct estimation of the seismic demand and capacity of MDOF systems through Incremental Dynamic Analysis of an SDOF Approximation, *ASCE Journal of Structural Engineering*, No. 4, **131**(2005) 589-99.
10. Bertero RD, Bertero VV. Performance-based seismic engineering: the need for a reliable conceptual comprehensive approach, *Earthquake Engineering and Structural Dynamics*, No. 3, **31**(2002) 627-52.
11. Chandler AM, Lam NTK. Performance based design in earthquake engineering: a multidisciplinary review, *Engineering Structures*, **23**(2001) 1525-43.
12. Estekanchi HE, Vafai A, Sadeghazar M. Endurance time method for seismic analysis and design of structures, *Scientia Iranica*, No. 4, **11**(2004) 361-70.
13. Riahi HT, Estekanchi HE, Vafai A. Estimates of average inelastic deformation demands for regular steel frames by Endurance Time method, *Scientia Irnaica*, No. 5, **16**(2009) 388-402.

14. Ghobarah A. Performance-based design in earthquake engineering: state of development, *Engineering Structures*, **23**(2001) 878–84.
15. Stewart RA, Kittelson J, Kay IP. Statistical methods to improve the precision of the treadmill exercise test, *Journal of American College of Cardiology*. No. 4, **36**(2000) 1274-9.
16. Estekanchi HE, Valamanesh V, Vafai A. Application of Endurance Time Method in Linear Seismic Analysis, *Engineering Structures*, No. 10, **29**(2007) 2551-62.
17. Riahi HT, Estekanchi HE, Vafai A. Endurance Time Method - Application in Nonlinear Seismic Analysis of SDOF Systems, *Journal of Applied Sciences*, No. 10, **9**(2009) 1817-32.
18. Behfar K, Estekanchi HE, Vafai A. An analytical approach for generating design response spectra compatible accelerograms, *The Seventh International Conference on Computational Structures Technology*, Lisbon, Portugal 7-9 September 2004.
19. BHRC. Iranian code of practice for seismic resistant design of buildings, standard No. 2800-05, 3rd edition, Building and Housing Research Center, Tehran, 2005.
20. Clough RW, Penzien J. *Dynamics of structures*, McGraw-Hill Inc, 1993.
21. Estekanchi HE, Riahi HT, Vafai A. Endurance Time Method: Exercise Test for Seismic Assessment of Structures, *14th World Conference on Earthquake Engineering*, Beijing, 2008.
22. Krawinkler H, Medina R, Alavi B. Seismic drift and ductility demands and their dependence on ground motions, *Engineering structures*, **25**(2003) 637-53.
23. Gupta A, Krawinkler H. Relating SMRFs seismic drift demands to element deformation demands, *AISC Eng J*, No. 2, **39**(2002) 100–8.
24. Estekanchi HE, Arjomandi K, Vafai A. Estimating Structural Damage of Steel Moment Frames by Endurance Time Method, *Journal of Constructional Steel Research*, No. 2 **64**(2008) 145-55.
25. OpenSees, *Open System For Earthquake Engineering Simulation*, Pacific Earthquake Engineering Research Center, http://peer.berkeley.edu/, 2002.
26. Riahi HT, Estekanchi HE. Application of endurance time method for estimating maximum deformation demands of structures, *First European Conference on Earthquake Engineering and Seismology*, Geneva, Switzerland, Paper number: 872, 2006.

NOTATIONS

A = Ground base acceleration
$a(t)$ = Acceleration at time t
$a_g(t)$ = Ground acceleration
B = Building response factor
e = Base of natural logarithm
FS = Factor of safety
$F(a_g)$ = Optimization target function
I = Importance factor (assumed 1.0 in this research)
R = Response modification coefficient

S_a = Acceleration response
$S_a(T,t)$ = Acceleration response for period T at time t
$S_a(T_1,5\%)$ = The 5% damped first-mode spectral acceleration
$S_{aC}(T)$ = Code acceleration response for period T
$S_{aC}(T,t)$ = Target acceleration response for period T at time t
$S_u(T,t)$ = Displacement response value for period T at time t
$S_{uC}(T,t)$ = Target displacement response value for period T at time t
t = Time
T = Free vibration period (sec)
t_{cur} = Current time
t_{Target} = Target time
$u(t)$ = Displacement at time t (relative to ground)
α = Weighing factor in target function
ω = Vibration frequency (radians)
ξ = Damping ratio (assumed 5% in this research)
τ = Dummy variable for time
q_{max} = Maximum interstory drift ratio