

Endurance Time Method for Seismic Analysis and Design of Structures

H.E. Estekanchi*, A. Vafai¹ and M. Sadeghazar²

In this paper, a new method for performance based earthquake analysis and design has been introduced. In this method, the structure is subjected to accelerograms that impose increasing dynamic demand on the structure with time. Specified damage indexes are monitored up to the collapse level or other performance limit that defines the endurance limit point for the structure. Also, a method for generating standard intensifying accelerograms has been described. Three accelerograms have been generated using this method. Furthermore, the concept of Endurance Time has been described by applying these accelerograms to single and multi degree of freedom linear systems. The application of this method for analysis of complex nonlinear systems has been explained. Endurance Time method provides a uniform approach to seismic analysis and design of complex structures that can be applied in numerical and experimental investigations.

INTRODUCTION

The basic objective of seismic design is to provide the structure with an appropriate safety margin against failure when subjected to strong earthquakes [1]. The common philosophy of most well-known seismic design codes is to achieve the dual goal of keeping the non-structural damage to a minimum in the case of service level earthquakes and, also, to prevent structural failure in the case of collapse level earthquakes [2,3].

Early observations of structural failure during earthquakes revealed that most structure failures could be attributed to the weakness of structures in sustaining the imposed lateral loads and displacements. This observation formed the basis of well-known earthquake design criteria, based on lateral load, also known as the static seismic design method. In the static lateral load method, the structure is designed to resist a minimum lateral load specified by the code. In this way, minimum lateral strength and stiffness is provided and lateral displacements are limited [2,4].

Extensive research work in the field of earthquake

engineering has revealed many deficiencies and shortcomings in the traditional method of static seismic analysis [5]. According to the static method of analysis, structures with higher lateral strength and stiffness are superior to their less stiff and less strong counterparts. However, experimental and analytical investigation shows that this is not always the case [6-8]. In fact, there are cases in which reducing lateral stiffness results in better seismic performance. The concept of seismic base isolation is one example [9,10].

These apparent shortcomings in traditional seismic design, along with remarkable developments in the field of information technology and the availability of vastly improved analytical tools, have led researchers and engineers to develop more rational and consistent methods for earthquake engineering [11-14]. In this respect, Performance Based Seismic Engineering (PBSE) has gained increased interest among practitioners in earthquake engineering field [3,15,16]. Development of these new methods and criteria should be mainly contributed to the amazing improvement in computational tools that have made possible the solution of sophisticated nonlinear models [17]. The use of static push over analysis is becoming standard practice in structural engineering design offices and the application of nonlinear time history analysis is also gaining popularity. Thanks to these recent developments, it has become possible for the structural analyst to incorporate the most significant nonlinear material and

*. Corresponding Author, Department of Civil Engineering, Sharif University of Technology, Tehran, I.R. Iran.

1. Department of Civil Engineering, Sharif University of Technology, Tehran, I.R. Iran.
2. Department of Civil Engineering, Tehran University, Tehran, I.R. Iran.

geometric behavior into the model and, thus, perform a more realistic analysis of structural behavior during earthquakes.

In recent years the development of new analysis tools has provided the infrastructure for the development of new methods in structural engineering. In this paper, a new method for seismic analysis and evaluation of structures is introduced. The concept of the Endurance Time (ET) criteria, which is quite intuitive, is first explained and prospective methods to implement it are discussed. The criterion is then evaluated by applying the concept to linear single and multi degree of freedom (SDOF and MDOF) systems.

ENDURANCE TIME CONCEPT

The concept of Endurance Time method (ET) can be best explained by considering a hypothetical experiment. Assume that three model buildings with unknown seismic resistance characteristics are to be investigated with reference to their performance against collapse in high intensity earthquakes. Now consider that these models are put on the shaking table and fixed to it. The experiment starts by subjecting the buildings to random vibration with gradually increasing intensity. In the beginning (e.g. at $t = 5$ sec) the amplitude of shaking is quite low so all three buildings vibrate but remain stable, as shown in Figure 1a. As the amplitude of vibration is increased (say, at $t = 10$ sec), a point is reached when one of the buildings collapses. It is assumed that this is model number one, as in Figure 1b. As time passes and the vibration amplitude is further increased (say, at $t = 20$ sec), the second structure fails. Assume this to be building number 3, as in Figure 1c. Further, consider that building number two happens to be the last building to fail in this hypothetical experiment.

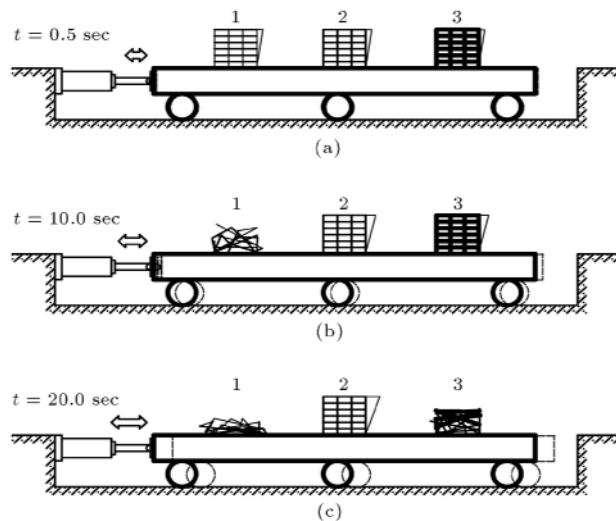


Figure 1. Hypothetical shaking table experiment.

Now, based on this experiment, it is concluded that building number two, which endured longer, has the best performance, while building number one, which failed soonest, performed the worst. Note that the judgment is based on the endurance time, i.e. the time during which each building remained stable, without any reference to the building strength or stiffness or other dynamic characteristics. If the goal were to evaluate the building's resistance to collapse, then, such experiments would seem to show a direct and relevant measure. This is the concept of the Endurance Time criterion in evaluating the performance of buildings subjected to earthquakes.

In the ET method, buildings are rated according to the time that they can endure a standard calibrated intensifying accelerogram. Higher endurance time is to be interpreted as a better performance. A minimum performance index can be set to be used as the design criterion. The idea of the Endurance Time method is somewhat similar to the method used by cardiologists to evaluate the condition of the heart, known as a stress test. In stress tests, the patient is asked to walk on a tread-mill with a variable slope and speed. The test starts with a low slope and low speed condition. During the test, the slope and speed are increased gradually, while the physical and biological condition of the patient, such as blood pressure, heart beat rate, etc. are monitored. The test is commenced until signs of distress or abnormal conditions are observed. The heart condition is then judged on the basis of the speed and slope level that could be tolerated.

In the Endurance Time method, nearly the same concept is applied. The idea behind the ET method is, roughly, to put the structure on a ramp-like accelerogram and see how far it can go. The structure is subjected to a standard accelerogram with intensifying dynamic demand. The specified performance indexes are monitored until they reach a predefined maximum value. The performance of the structure is judged on the basis of the time when the damage limit index is exceeded. This concept has been explained in Figure 2. Consider the damage index curve (e.g. maximum drift, plastic energy, etc.) for a typical structure subjected to an intensifying accelerogram, as shown in this figure.

If the limit value for the specified damage index is specified to be 1.00, then, it can be concluded from Figure 2 that this structure has endured the accelerogram up to about the 12th second. Moreover, consider that the accelerogram has been calibrated and the design criterion is that endurance time should be at least 10 seconds. As can be seen from this figure, the damage value is about 0.82 at $t = 10.0$, seconds i.e. below the limit value; thus, one can conclude that the structure has met the design criteria. The analysis should not necessarily be limited to a single damage criterion. Various different damage indexes and struc-

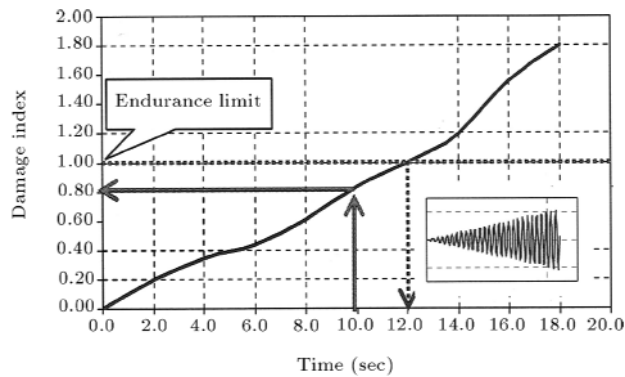


Figure 2. Damage curve against time for a typical structure subjected to intensifying accelerogram.

tural criteria can be monitored simultaneously in order to reach more conclusive results.

In order to achieve quantitative results, one needs to propose a method for implementing the concept. Direct physical testing of actual structures as described above is, of course, not a practical proposition for general application considering time and cost [6]. Analytical software that is capable of modeling and predicting structural behavior up to the complete collapse point are available, but are still considered to be at research level. In this paper, some practical implementations of the Endurance Time method are introduced by making some simplifying assumptions and by making use of already available analytical tools. Modified and improved implementations should be developed based on ongoing research. However, complete numerical implementation of the method, which should, ideally, follow the nonlinear response up to collapse level, should be left to the next generation of structural engineering software and computer hardware capable of performing such analysis.

Basic implementation of the concept is based on three fundamental bases, i.e. dynamic input, structural model and endurance criteria. These will be discussed next.

DYNAMIC EXCITATION

The choice of an appropriate dynamic input is fundamental to the successful implementation of the ET concept. The ideal input function is the one that results in higher consistency and best correlation between the endurance time analysis results and the actual or well-known performance characteristics of the structures subjected to earthquakes. Determination of the most appropriate and optimal dynamic input function should, in itself, be the subject of extensive research work. However, in this research, a function based on engineering judgment and some elementary calculations will be proposed so that the concept and application of the ET method can be explained.

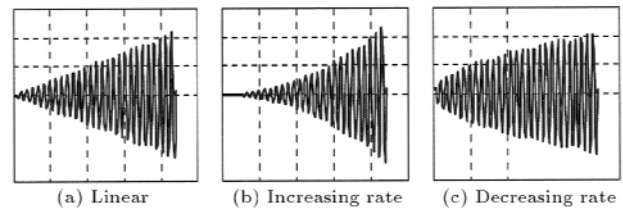


Figure 3. Input profile functions.

An important issue in determining dynamic input is the profile of amplitude increase. As shown in Figure 3, the amplitude increase profile can take various forms. After considering several possible alternatives, the authors came to the conclusion that a linear profile, as shown in Figure 3a, is more suitable for the purpose of initial investigation. In this profile, maximum acceleration is directly proportional to time. Determination of an optimum profile that results in better correlation and consistent results is under investigation.

Another important consideration in ET analysis is the specification of the dynamic input function itself. The most basic form of dynamic input is a simple harmonic. But this kind of input has obvious disadvantages because of poor frequency content consisting of only one harmonic. The structures that have natural vibration periods near the input frequency will experience high dynamic magnification and, thus, will be too much penalized. Considering this issue, a random vibration input with a frequency content resembling that of a white noise has been used as the starting point for generating of intensifying accelerograms. As will be explained later, the frequency content is later modified to better correspond to what is expected in a real earthquake. The accelerograms generated by this procedure seem to be good enough for investigation of the ET concept and will be applied in this study. Further research is required and is under way, in order to propose optimized dynamic input so that the results of Endurance Time analysis are best correlated with the known actual performance of structures in earthquakes and shaking table experiments.

Generation of Intensifying Accelerograms

The first generation of accelerograms to be used as dynamic input in the ET method is produced using random numbers with a Gaussian distribution of zero mean and a variance of unity [18]. A stationary random accelerogram is generated using $\delta t = 0.01$ and $n = 2^{11} = 2048$ with $PGA = 1$ shown in Figure 4. Duration of the accelerogram is equal to $\delta t \times n = 20.48$ sec.

The frequency content of the random accelerogram that is statistically similar to a white noise is then modified, in order to resemble actual earthquake accelerograms. For this purpose, filter functions given

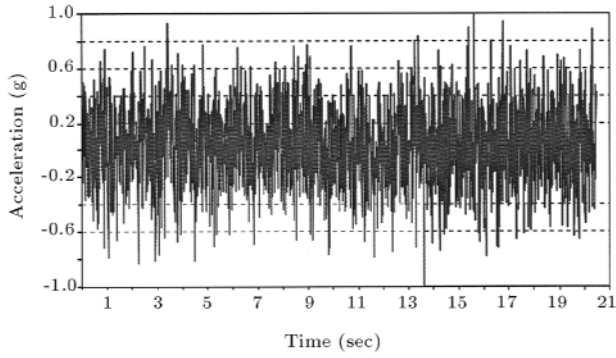


Figure 4. A typical random accelerogram with PGA = 1.

by the following equations are applied to the random accelerograms. Application of these filter functions is explained by Clough and Penzien [18].

$$H1(i\omega) = \frac{1 + 2i\xi_1 \frac{\omega}{\omega_1}}{(1 - \frac{\omega^2}{\omega_1^2}) + 2i\xi_1 \frac{\omega}{\omega_1}}, \quad (1)$$

$$H2(i\omega) = \frac{(\frac{\omega}{\omega_2})^2}{(1 - \frac{\omega^2}{\omega_2^2}) + 2i\xi_2 \frac{\omega}{\omega_2}}, \quad (2)$$

where $\omega_1 = 2\pi/0.5, \xi_1 = 0.2, \omega_2 = 2\pi/0.1, \xi_2 = 0.2$ are used. It should also be noted that the frequencies higher than 25 Hz are filtered out. A sample frequency content resulted from the accelerogram shown in Figure 4 is depicted in Figure 5.

The frequency content is then further modified so as to make the resulting response spectra compatible with typical seismic code response accelerograms. Any given design response spectrum can be used for this purpose. In this paper, the response spectra of the Iranian National Building Code (Standard 2800) [4] has been used as a sample. The resulting frequency content of a response spectra compatible accelerogram, after several cycles of stepwise modification, is shown in Figure 6.

Acceleration response, which is compared with the code response spectrum curve, is depicted in Figure 7. The convergence is assumed to be good enough for the purpose of explanation of the ET

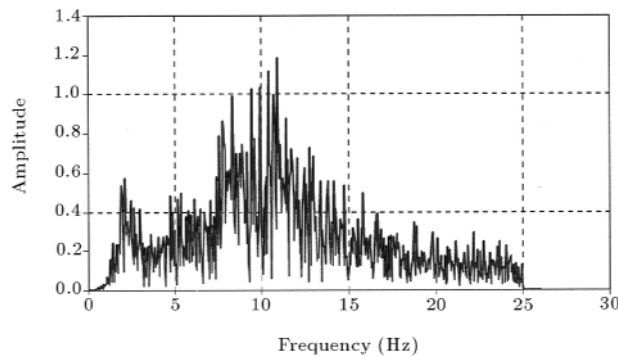


Figure 5. Filtered frequency content.

concept. Further calibration of the accelerogram is under investigation.

In the next step, the acceleration values are adjusted for target values of velocity and acceleration, which, in this case, are set to zero. Finally, the acceleration values are multiplied by a profile function, which, in this study, is a linear one starting from zero and reaching a value of 1.00 at $t = 10$ sec. Three accelerograms have been generated using the procedure explained above. These accelerograms will be referred to hereafter as acc1, acc2 and acc3. Accelerogram acc1 that is obtained from the original accelerogram in Figure 4 is shown in Figure 8.

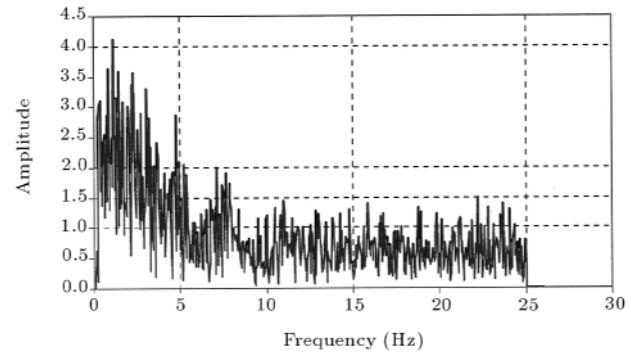


Figure 6. Modified frequency content.

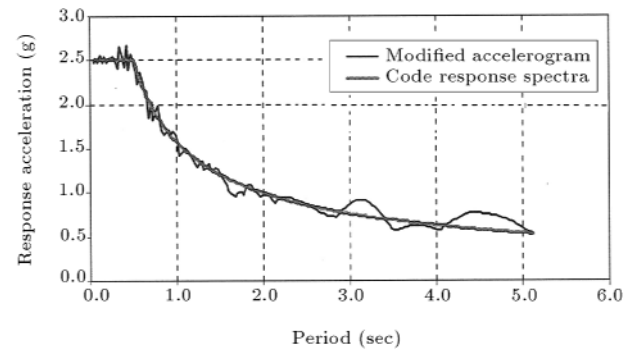


Figure 7. Response of modified accelerogram compared to codified value.

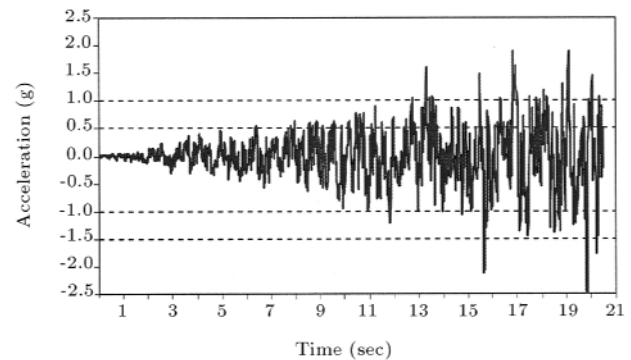


Figure 8. Intensifying accelerogram acc1.

ANALYSIS METHOD

Collapse analysis of the structures is still considered a challenging task for structural engineers involved in the numerical analysis of structures. There are a few programs that are capable of conducting such analysis with reasonable assumptions. However, the experimental evidence to verify the results of such analyses is still quite limited. The minimum modeling requirement for collapse analysis is nonlinear material behavior, including material degradation due to cyclic behavior, large strain and fracture. Analysis should include the effect of large deformations, buckling and collision. Furthermore, for the purpose of illustrating the ET method, the simplest method of dynamic analysis is considered in this paper, i.e. linear systems. The Newmark linear method has been applied for the analysis [19]. It should be evident that, as far as the concept of the ET method is considered, the procedure remains almost the same, even for complex MDOF models, including nonlinear features.

ENDURANCE CRITERIA

The numerical definition of structural collapse and failure is not a straightforward procedure. An intuitive failure criterion can be defined as the displacement of the center of mass of the structure to a lower level from its initial position. However, analytical implementation of such criteria is still considered to be very demanding in terms of computational effort and modeling complexities. The state of the art in this regard is to define the structural damage in terms of damage indexes. Various damage indexes have been defined and proposed by researchers in recent years. Seismic codes that are based on performance based design usually propose a certain damage index and set maximum acceptable values for it. In this way, the structure is assumed to have collapsed when its damage index exceeds specified code limits. The same simplified method shall be used for the purpose of this study, as described in the next sections. Endurance Time is defined as the time it takes for the specified damage criteria to reach its limit value as the structure is subjected to intensifying accelerograms. For example, in the static method of earthquake resistant design, structures are required to be designed according to a specified base shear, which is proportional to certain peak lateral ground acceleration. Also, limits to maximum lateral displacements and drift are usually specified. These can be considered as the simplest forms of damage indexes. In the discussion that follows, basic parameters such as “max response acceleration”, “max displacement response” and “max inter-story drift” have been used for the matter of explanation of the idea. It will be up to the structural designer/analyst to choose an

applicable damage index that is more appropriate for the structure under investigation. It should be evident from the discussion that many different damage indexes can be applied in the procedure. Practical implications of applying different damage indexes for ET analysis are under investigation.

APPLICATION TO LINEAR SDOF SYSTEMS

For linear SDOF systems, the most significant parameter that describes the dynamic characteristics of the structure is its natural frequency of vibration. Consider that three different linear structures, with natural periods of vibration equal to 0.1, 0.5 and 1.0 seconds, are to be studied. These structures will be, hereafter, referred to as ST01, ST05 and ST10. ST01 roughly pertains to a stiff single-story masonry building, while ST05 and ST10 have a period typical of three and nine story steel frame buildings, respectively. Damping ratio will be assumed to be 0.05 of the critical value, as commonly assumed in dynamic analysis. The acceleration response for ST05, subjected to the three generated accelerograms, is shown in Figure 9. As expected, peak acceleration increases with time as the input acceleration is intensified.

In Figure 10, maximum acceleration as a function of time has been plotted, along with average value and a linear fitting curve. As can be seen in this figure,

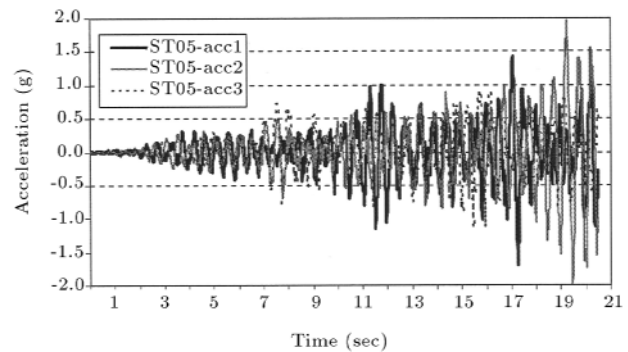


Figure 9. Acceleration time-history for ST05.

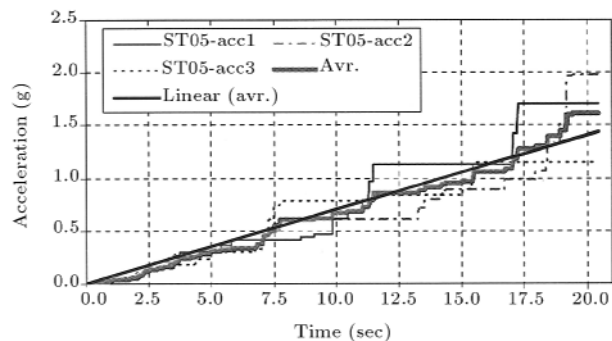


Figure 10. Maximum acceleration and average for ST05.

in spite of the fact that the generated accelerograms are compatible with the same design response spectra and have a similar frequency content, the dynamic response can be significantly different at specific time intervals. For example, consider that one wants to specify the time at which the structure has experienced a maximum acceleration of 1 g. From Figure 10, it can be seen that for acc1, this occurs at t equal to about 11.5 seconds, while, for acc2, this occurs at $t = 16$ seconds.

This could be expected, considering the well-known characteristics of time-history analyses. An averaging method can be used to achieve better approximation. In this example, using the linear fitting curve, the time at $a = 1$ g can be seen to be about 14 seconds.

The displacement response of ST05 subjected to different accelerograms has been depicted in Figure 11. As can be seen in this figure, the displacement response is also an increasing function of time, as expected. Due attention should be paid to the pulsating characteristic of the displacement response. These pulsations can result in the maximum response, to remain constant during a relatively long period of time, making it difficult to interpret the result of analysis regarding the time corresponding to a certain response level. This problem can be avoided by using several accelerograms, along with an appropriate averaging method. Even though, in this paper, the number of accelerograms to be averaged has been set to three as a practical minimum, it should be clear that the desired accuracy and convergence can be achieved by considering a larger number of accelerograms.

Maximum and average accelerations are shown in Figure 12. Maximum displacement is considered to be a simple and effective damage criterion. In multistory buildings, maximum displacement is usually proportional to maximum drift, which is another significant response characteristic that can be related to building damage. Consider that the maximum tolerable displacement for this building has been set to 4cm. Based on Figure 12, one can conclude that the

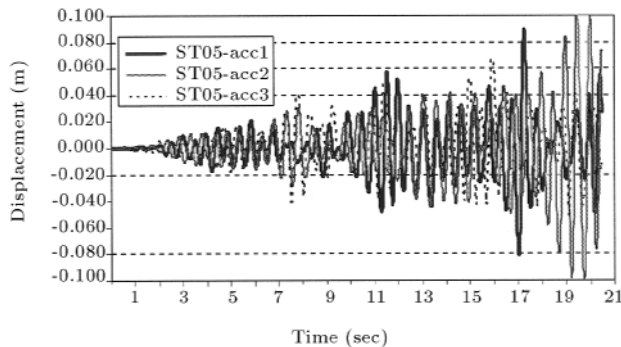


Figure 11. Displacement time-history for ST05.

building can endure the prescribed accelerogram up to $t = 11$ seconds.

A summary of the results for the maximum acceleration of ST01, ST05 and ST10 are depicted in Figure 13. In general, the magnitude of acceleration experienced by all three structures is nearly the same, with that of ST05 being higher for almost the entire time range and that of ST10 being lower in the time interval from 7.5 to 12.5 seconds. These could be expected considering the shape of the codified response spectrum and the frequency content being amplified near $T_0 = 0.5$ sec, according to the assumed soil conditions.

Displacement responses have been depicted in Figure 14. As can be seen in this figure, there is a

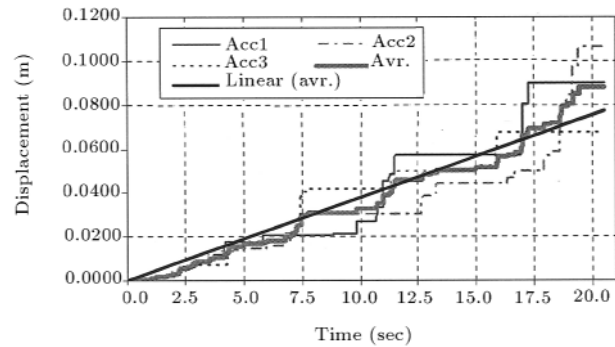


Figure 12. Maximum displacement and average for ST05.

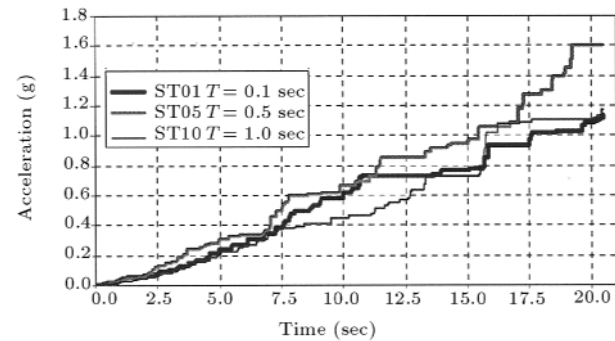


Figure 13. Maximum average accelerations.

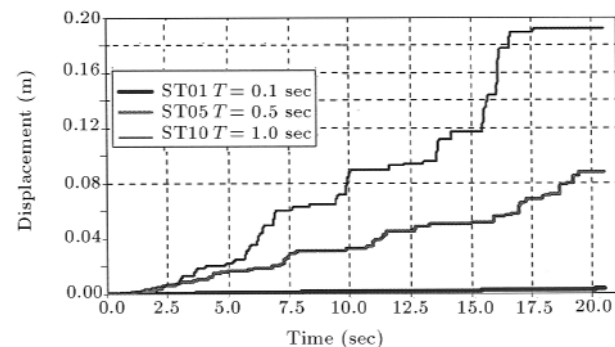


Figure 14. Maximum average displacements.

marked distinction between the displacement responses of sample structures. Significant insight on the structural performance of these structures can be gained through studying the curves of Figure 14. For example, consider that the maximum tolerable displacement for all these structures was to be limited to 4 cm. It can be concluded from Figure 14 that the endurance time for ST10 is about 6 seconds, for ST05 about 11 seconds and for ST01 more that 20 seconds. Now, by specifying the required endurance time of 10 seconds, it can be concluded that ST01 and ST05 are acceptable, i.e. can endure the specified dynamic demand, while ST10 is rejected.

As another example, consider that ST05 and ST10 are three and nine story buildings with heights of 10 m and 30 m, respectively, and the failure criteria is set to be a maximum drift of 0.005. If one, roughly, assumes the vibration mode to be linear, maximum displacements will be $0.005 \times 10 = 0.05$ m and $0.005 \times 30 = 0.15$ m, respectively. Now, by referring to Figure 14, it can be seen that ST05 reaches the value limit at about $t = 15$ seconds, while ST10 reaches its value limit at $t = 15.5$ seconds. Thus, the endurance time for both structures is nearly the same and, if one sets the required endurance time to 10 seconds, then both structures will be considered acceptable.

In the above discussion, the linear analysis of a single degree of freedom is considered for the purpose of describing the basic idea behind the Endurance Time method. It should be clear from this discussion that the analysis can be readily extended to nonlinear and multi degree of freedom systems. The essence of the ET method lies in the definition of standard intensifying accelerograms and appropriate damage criteria. It is interesting to note that the concept of ET can also be readily applied in experimental dynamic investigation of structures. In this case, the most realistic performance criteria, i.e. actual failure of the structure, can be considered the endurance limit as measured against time. Another advantage of the ET method, as compared to other dynamic analysis methods, is in its applicability to experimental shaking table studies. In these cases, the cost and required resources usually eliminate the possibility of conducting the experiment several times with different levels of excitation. Using the concept of endurance time, a single run experiment can be used to obtain the desired results.

APPLICATION TO MDOF SYSTEMS

In this section, the application of the ET method in the dynamic analysis of MDOF systems is explained by considering a three story steel moment frame example. A three story steel frame, named “f3” in Figure 15, has been designed in accordance with conventional loading and analysis procedures, appropriate for ordinary mo-

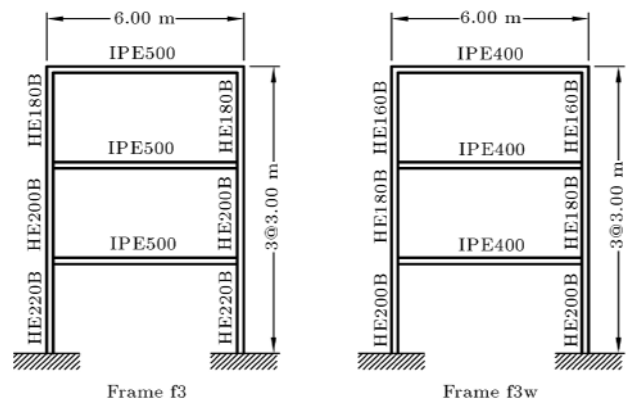


Figure 15. Three story steel moment frames.

ment frames in high seismic zones. For the matter of comparison, another frame with similar properties, but utilizing steel sections with lower stiffness and strength, has also been designed. This weak frame is named “f3w” in Figure 15. These frames are to be analyzed using the ET concept and the results will be studied. For multi story frames, inter-story drift has proved to be a very significant and convenient measure of building damage and performance criteria. This criterion shall be used for the purpose of explanation of the method. It should be clear that other criteria, based on strain energy, nonlinear hysteresis behavior etc., can also be applied with no further complications. The only limitation in the application of the ET method will lie on the computational capability of the analysis program used.

Frame f3 has been subjected to accelerogram acc1 and the time-history of story drifts has been depicted in Figure 16. It can be seen from this figure that the story drifts are nearly the same for most of the time-history and, also, reach peak values at almost the same times. This is due to the fact that in MDOF frame structures, mode number 1 is, usually, the predominating deflection mode. Higher vibration modes have had a much less pronounced effect at peak values for inter-story drifts.

Frame f3 has been subjected to accelerograms

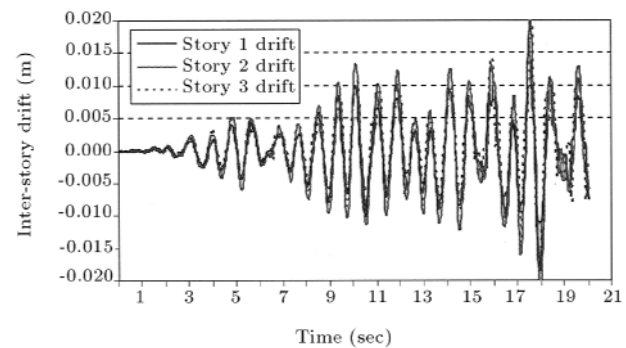


Figure 16. Story drifts time-history for frame f3 subjected to acc1.

acc2 and acc3 as well and the maximum absolute values of drifts along with their average value are depicted in Figure 17. As expected, the maximum drift increases with time and, by considering a limiting value for drift, the endurance time can be evaluated. Due to the randomness of input and resulting response, the peak value can remain constant for a prolonged period of time. For example, maximum drift has remained constant from $t = 10$ to $t = 16$ seconds for f3 subjected to acc1, as shown in Figures 16 and 17. Considering the irregular fluctuation in the drift values for the frame subjected to each accelerogram, the average value should be used, in order to avoid inaccuracy resulted from the random nature of the response.

A curve fitting method can also be used in order to better define the endurance time value. In Figure 17, a linear trend line has been added for this purpose. In linear systems subjected to linearly intensifying accelerograms, it is known that the response is roughly linear. This justifies the use of linear trend line as the fitting curve. However, it should be clear that when applying the ET method to non-linear problems, a more appropriate fitting method, such as spline or polynomial curves, should be used.

Now, considering a limiting value of 0.005 for inter-story drift, as a typically recommended maximum value, the limiting drift for frame f3 with a story height of 3.0 m will be $0.005 \times 3.0 = 0.015$ m. Thus, one can conclude from Figure 17 that frame f3 has endured an acc1-3 set of accelerograms up to time $t = 13.5$ seconds. If one had standardized these accelerograms for the specified site and had specified a minimum endurance time of, say, 10 seconds, one would then conclude that frame f3 passes the seismic design criteria. Generating a standard set of intensifying accelerograms to be used as the design criteria in the ET method is under investigation and is the subject of another study.

The result of the same analysis as applied to frame f3w, i.e. the weak version of f3, is summarized in Figure 18. The general trend for f3w is the same as f3. It should be noted that the resulting drift

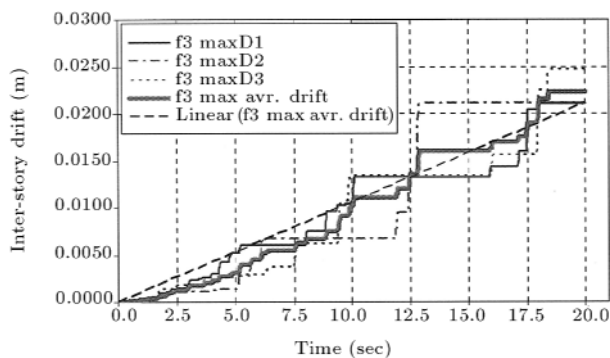


Figure 17. Maximum story drifts for frame f3 subjected to acc1-3 accelerograms.

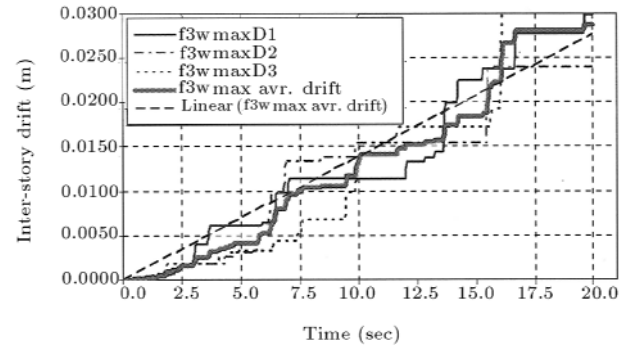


Figure 18. Maximum story drifts for frame f3w subjected to acc1-3 accelerograms.

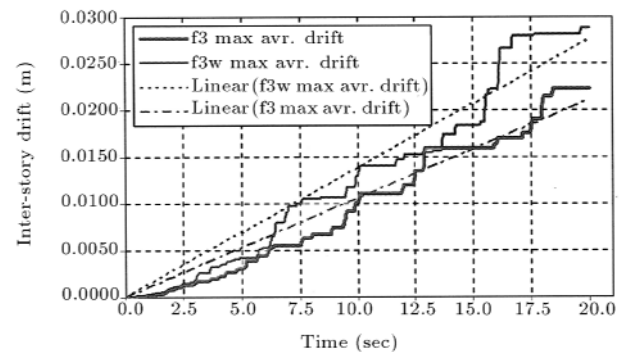


Figure 19. Maximum average story drifts for frames f3 and f3w.

values are, generally, higher for f3w. This could be expected, considering the lower stiffness of frame f3w. Considering the limiting value of 0.005 for inter-story drifts, as above, it can be concluded that the endurance time for f3w is about 10.5 second.

Analysis results for frames f3 and f3w are compared in Figure 19. The significant point in this figure is the endurance time for f3w being clearly lower than that of f3. On the other hand, the weakness of f3w as compared to f3 is observed in its response to the intensifying accelerograms. Again, if one considers that the accelerograms have been somehow standardized and a minimum endurance time of 10.0 seconds has been specified for the building site, it can be concluded that f3w passes the seismic design criteria. If, however, the minimum endurance time was specified to be 12.0 seconds, f3 would pass the design criteria while f3w would fail.

In case of the simple example considered above, one could already predict the analysis result by considering the fact that f3w is less stiff than f3. However, in cases of more complex models, particularly models involving several sources of nonlinearity, it is not possible to predict the endurance time and the advantage of one design over the other. For example, consider that the effectiveness of various energy absorbing devices in reducing the maximum drift of a structure is to be studied. These types of analysis are usually encountered

when considering seismic retrofit studies. It should be clear from the above discussion that by applying the ET method, a conclusive analysis result can be readily achieved.

SUMMARY AND CONCLUSIONS

In this paper, a new approach to performance based earthquake analysis and design has been introduced. In this method, the structure is subjected to standard accelerograms that impose increasing dynamic demands on structures with time. Specified damage indexes are monitored up to the collapse level or other performance limits that define the endurance limit point for the structure. Endurance time is defined as the length of the time interval from start to failure point (or limit point), i.e. the time at which the damage index reaches its maximum tolerable value. Longer endurance time implies better or improved performance. Standard accelerograms are calibrated and a minimum endurance time is specified to be used as the design criteria. In this way, the structure is considered acceptable or unacceptable by its endurance time being higher or lower than the benchmark value.

The Endurance Time method provides a unified and objective approach to seismic analysis and design of structures that can be applied in numerical and experimental investigations, regardless of model complexity. Also, the method is readily applicable to complex nonlinear systems. The definition and calibration of standard intensifying accelerograms is a key issue in this method. Furthermore, a simple approach to generating required accelerograms has been discussed. Three such accelerograms were applied in the simple case of linear elastic single and multi degree of freedom systems. Methods for generating improved accelerograms to achieve better convergence are under investigation. Also, practical implications in cases of nonlinear MDOF systems are being studied.

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