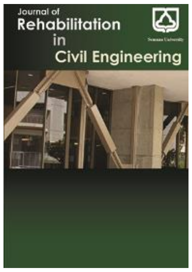
Journal of Rehabilitation in Civil Engineering 2-2 (2014) 46-62



journal homepage: http://civiljournal.semnan.ac.ir/

The Comparison of the IDA and ET on Steel Structures Rehabilitated by Isolators

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ARTICLE INFO

Article history:
Received: 02 April 2014
Accepted: 22 October 2014

Keywords:
Nonlinear analyses,
Incremental dynamic analysis
Endurance time method,
Seismic rehabilitation,
Base isolation.

ABSTRACT

Endurance time (ET) method is a relatively new performance-based earthquake method for structural evaluation and design. In this method, the structure is subjected to a gradually intensifying accelorogram(s) and its performance is assessed based on the maximum time duration that it can meet the specified performance objectives. Using moment-resisting and braced frames which are rehabilitated by base isolators; it is shown that ET can estimate results of the Incremental dynamic analysis (IDA) with acceptable accuracy. Spectral acceleration has been considered as the intensity measure. While the IDA would require several nonlinear dynamic analyses under multiple suitably scaled ground motion records, the ET can practically estimate the seismic behavior of simple to complex systems only by few nonlinear time history analyses. Moreover, it is shown that even in intense ground motions, super structures do not show considerable nonlinear behaviors and interstory drifts, because of energy dissipating by base isolation. The results also show that the ET method has the benefits of both time history analysis and the simplicity of the nonlinear static methods and can potentially be a useful tool for evaluation and designing seismic-resistant structures.

1. Introduction

Due to recent advances in the field of earthquake engineering, More efficient seismic-resistant design and rehabilitation methods have been innovated, which facilitate the way towards construction of much safer structures. Performance-based earthquake engineering (PBEE) is a relatively novel approach, in which structural elements are designed based on achievement of a levy of performance objectives. Different performance objectives can make the structural rehabilitation or design process more flexible, based on some factors like: frugal situation, importance and functions of structures, risks levels, and so on. This phenomenon is a conspicuous development in the domain of civil engineering, because it enables engineers to suggest more economical design and rehabilitation plans. Performance objectives relate performance levels (as an analysis output) and risk levels (as an analysis input) to each other. As for performance levels, they show levels of structural damage which can be displacements, rotation of hinges or drift ratios. To monitor and control lateral displacements of structures, an accurate estimation of maximum lateral displacement of structures subjected to various ground motions is called for. Structures under such ground motions may step into inelastic phase. But the evaluation of seismic performance of structures with nonlinear dynamic analysis has some complications out of haphazard characteristic of ground shakings and being very time-consuming. Accordingly codes recommend using other simple techniques, and by passing the time, new techniques have been being innovated to solve these mentioned problems.

Incremental dynamic analysis (IDA) is a kind of time history analysis in which structures are exposed to records, each of which are scaled for different intensifying levels of excitations. Therefore at different intensity measures (IMs), engineering demand parameters (EDPs) are achieved. This method can give a complete description of structural behavior from the weakest to strongest ground motions. Accordingly this method is the most accurate and complete method, but it needs a huge number of nonlinear dynamic analyses-because the results change from record-to-record significantly. Moreover, the procedure of processing its results is complicated and time consuming. Then researchers try to solve these drawbacks by either decreasing the number of ground acceleration records or finding other equivalent methods like the endurance time method (ET).

The endurance time method (ET) is a time-history-based analysis which has been introduced as a potential alternative to the incremental dynamic analysis and the nonlinear static analysis. The concept of ET is originated from the IDA method. [1] This method has accuracy of the IDA method and simplicity of the pushover procedure. In this method the EDPs of structures at different IMs are predicted by subjecting those structures to some predesigned intensifying records. IMs in ET acceleration functions are time, in other words the intensity of records increases with time constantly. Because of the intensifying essence of these records, they can provide a fair evaluation of the structures through elastic to yielding and nonlinear inelastic phases [2], (and finally global dynamic instability).

There are two strategies in order to rehabilitate structures (to make balance between demand and capacity): increasing the capacity of structures or decreasing their seismic demands. The seismic-isolation method is a simplified method to reduce or eliminate the earthquake damage potential. Seismic isolation, a seismic resistant design method, which instead of increasing capacity of structures, focuses on mitigating seismic demands and excitability of structures by means of increasing their period and damping ratio.

In this paper the accountability and applicability of the ET method in estimation of seismic performance of steel structures which are rehabilitated by base isolators are assessed in nonlinear seismic analyses by comparing their results with those of incremental dynamic analysis. Firstly, the concept of endurance time method and its methodology are discussed. Later, an existing steel structure is rehabilitated by base isolation and lastly, as will be demonstrated, EDPs calculated by the ET method are compared with those from incremental dynamic analysis. Ultimately, this comparison show fair consistency between results of these two methods [4].

2. The concept of ET method

The ET method is a kind of pushover technique, in which structures are subjected to preplanned intensifying accelerograms in order to percept the seismic performance of structures. The general concept of this method is to some extent originated from the concept of exercise test to evaluate the cardiovascular system of heart patients [5]. In this test the patient has to be put on a treadmill and made walk on it, while its slope and speed are gradually increased until the patient shows signs of physical idiosyncrasy; like: gasping or inability to continue the test. Then the cardiovascular system situation is assessed based on the time in which the patient endured the test. Tangible description of the concept of ET method can be more contended by mentioning a shaking table experiment, in which three different structures are put and fixed on a shaking table to be evaluated in terms of their seismic performance. As demonstrated in fig. 1(a), at the beginning of the test, low-amplitude vibration is applied on all three structures by the shaking table and then the intensity of vibrations increases constantly and gradually. While the vibration amplitude increases, in one moment of the test, one of the structures fails whereas the two other structures are able to continue the test (as shown in fig.1 (b)). By increasing the amplitude of vibration, the third and the second structures fail respectively as shown in Figs. 1(c)-(d). Due to the facts that the first structure (A) failed at t=5(s), the third structure (C) failed at t=15(s) and the second structure (B) failed at t = 20 (s), and also the increasing vibration is analogous to the real ground shaking, it can be inferred that the structure 'A' which failed firstly is the weakest and the structure 'C' which endured longest has the best performance. The structure 'C' which failed after the structure 'A' and before the structure 'B' has a performance in between the performance of mentioned structures. If the former dynamic applied excitations was rectified so that they accord to earthquake-induced excitations, it can be expected that the structure 'B' is capable of enduring more intensive excitations than the structure 'A'. Then this experiment can give a comprehensive assessment about the seismic performance of different structures.

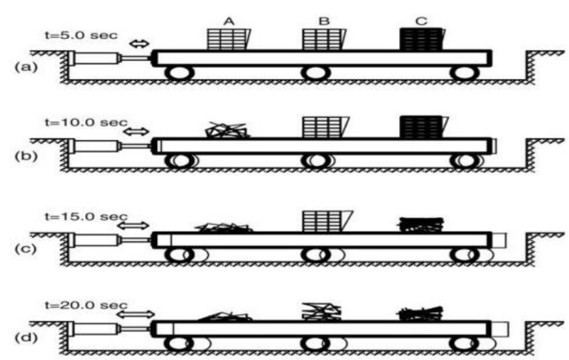

Fig. 1. The shaking table test

In the endurance time method a structure is subjected to intensifying acclerograms and then the maximum quantity of EDPs as of the beginning of the test or analysis till the target time should be calculated. These

EDPs can be one or a set of performance criteria which are used in evaluation or design of structures, from simple parameters like: displacements, tensions… or more complicated failure indices, like plastic rotations, can all be used in this method (based on the nature of study) [5]. In the Fig. 2, the application of the ET method is manifested, if we regard '1' as an ultimate level of damage index, maximum time in which structures endured and didn't reach the ultimate level (of damage index) of their response (or in other word their endurance time), can be extracted. It can be concluded from Fig. 2 that the frame '1' endured 10 seconds; the frame '2' endured 15 seconds and the frame '3' (endured) more than 20 seconds. If the applied accelerogram is calibrated so that this excitation intensity measure accords to the excitation intensity measure of design earthquake (based on the seismic codes of lateral load), in the 12th second of accelerogram, the frame 1 which its endurance time is less than 12 seconds has not acceptable performance. However, both frame 2 and 3 pass the design criteria. It is worth noting that in the assessment above, more than one damage index can be used simultaneously [5].

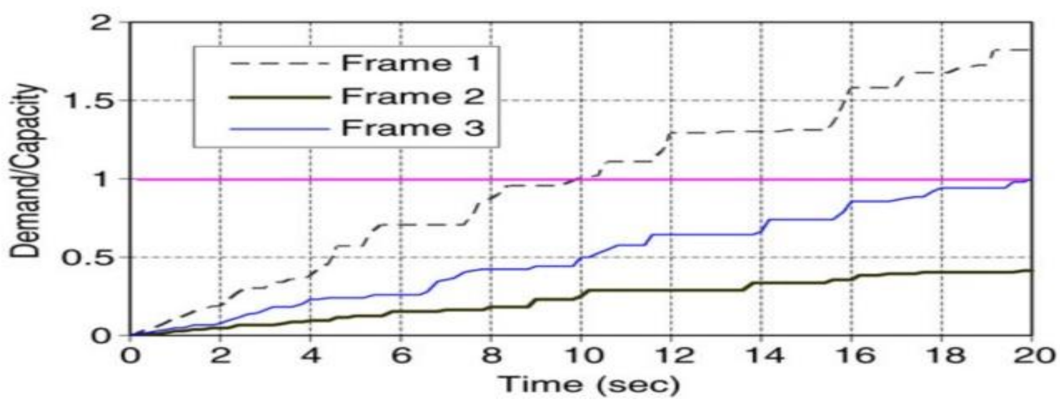

Fig. 2. Demand/capacity of frames under ET records

3. Design of ET functions

The practical application of the ET method depends on generating intensifying excitation functions which their results comply with the results of real earthquake. To reach this objective, the concept of response spectra is used in the ET functions. These functions are designed based on numerical optimization methods in a way that their acceleration spectrum till any specific time (according to Eq. (1)) gets harmonious with the design acceleration spectrum:

$$S_{uc}\,(T,\,t) = \frac{t}{t_{target}}\;Sac(T)\,\frac{T^2}{4\pi^2} \tag{1}$$

$$S_{ac}\,(T,\,t) = \frac{t}{t_{target}}\;Sac(T)$$

Where Sac (T, t) is the target acceleration response at time t, T is the period of free vibration and Sac (T) is the codified design acceleration spectrum. Suc (T, t) is the target displacement response at time t.

The merit of the ET method is its ability to evaluate the seismic response of structures in various intensities based on the design spectrum. This method can reach the goal with much less time than time–history analyses. In comparison to linear and nonlinear static and spectral dynamic analyses, due to the fact that the ET's nature is time-history analysis, the ET method can consider all the complications of structural behaviors like: buckling failure, crack, large displacements, time effects, dampers, cyclic behavior and so forth. Also, because of simple conceptual principles and not having complications of choosing records which are compatible with a given situation, its application is easier. The demerit of this method in comparison to the static and spectral method is its own difficulties and in comparison to other complete dynamic analyses is its low accuracy [4].

The ET functions are 4 generations which have been developed and modified constantly. Each generation includes several functions which their main difference is their target representative spectrum. The other discrepancy is period amplitudes which are covered in their design process. This issue in nonlinear applications becomes important. Other differences are time intervals, record duration… which are specified in files of records. In the first generation of ET records, the compatibility with spectrum was being disturbed while they became incremental. In the second generation of records, spectral increments in accordance with representative spectrum are obtained with numerical optimization in the domain of linear spectrum. These records with some modifications for long periods can be used also in nonlinear analyses. The third generation of ET records has been optimized in the nonlinear domain. In the fourth generation, the consistency (in terms of movement cycles and strong-ground-motion duration) is considered in their production process.

In this study two sets of records which are designed for nonlinear analyses are used: ETA20e and ETA20en, the former belongs to the second generation and the latter belongs to the third generation. As will be discussed later, because of the nonlinear nature of analyses, in order to smooth diagrams of results, moving averaging is used. In comparison to simple averaging, in nonlinear analyses, moving averaging is more comprehensive and useful [4].

4. Steel frame modeless used in this study

As mentioned in the paragraphs above, rehabilitation of steel structures with seismic base isolators is taken into consideration in this study. For this reason, one old existing building, which is a symmetrical four-storey And three-span building, (besides in one direction has moment-resisting frames and in the other direction has braced frames) is used. This structure got more weakened by changing its elements with weaker elements to make it so frail that it needs rehabilitation. The report of tensile strength tests of this structure are shown in Table. 1, also the average and expected tensile strength tests of elements are shown in Table. 2.

Table 1. The report of tensile strength tests of elements

Element	Position	F_y(kg/cm^2)	F_u(kg/cm^2)
Column	Storey 4	3260	4380
Column	Storey 3	2950	4123
Column	Storey 2	3250	4265
Column	Storey 1	3170	4250
Beam	Storey 4	3389	4530
Beam	Storey 3	3291	4480
Beam	Storey 2	3380	4500
Element	Position	F_y(kg/cm^2)	F_u(kg/cm^2)
Beam	Storey 1	3271	4356
Brace	Storey 4	2840	4095
Brace	Storey 3	2980	4130
Brace	Storey 2	2850	4100
Brace	Storey 1	3120	4240

Table 2. The average and expected tensile strength tests of elements

Element	F_y(kg/cm^2)	F_{ye}(kg/cm^2)	F_u(kg/cm^2)	F_{ue}(kg/cm^2)
Column	3013	3157.5	4160.1	4279.5
Beam	3272.3	3332.75	4390	4466.5
Brace	2816	2947.5	4073.63	4141.25

In Table. 2:

Fy: Yield strength

Fye: Expected yield strength

Fu: Ultimate strength

Fue: Expected ultimate strength

One of the moment-resisting frames along with one of the braced frames are extracted to be studied in this paper. The geometry and section properties of frames are depicted in Figs. 3-4.

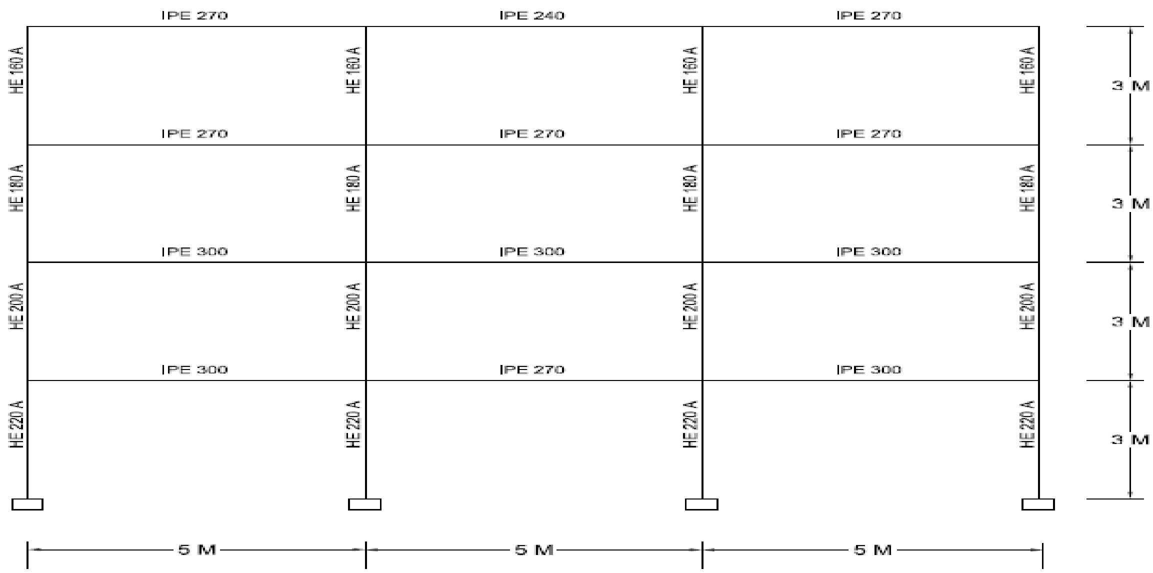

Fig. 3. Geometry of the moment-resisting frame

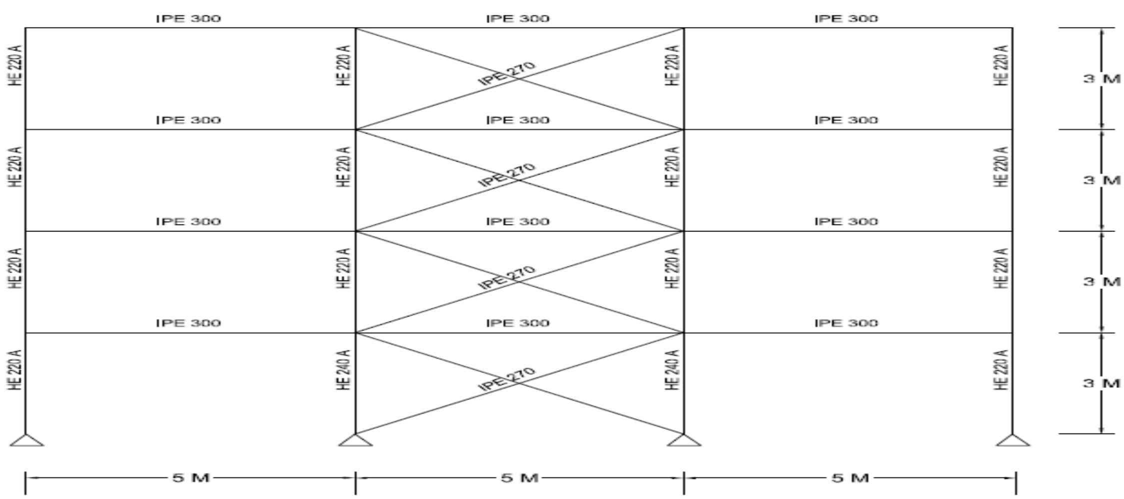

Fig. 4. Geometry of the braced frame

A bilinear material model with post-yield stiffness equal to zero is used to study the nonlinear behavior of the frames before and after rehabilitation. Because excitability of superstructures decreases significantly after rehabilitation of structures by the base isolation method, disregarding of strength losses in modeling can be reasonable. This material model along with those ones which their post-yield stiffness is equal to 3% of the initial elastic stiffness has been used widely by researchers. This material model is applied in the analyses using FEMA beam-column elements in Perform with nonlinear distributed plasticity.

The FEMA beam-column element is used for both beams and columns of frames. In order to model the nonlinear behavior of braces, inelastic-bar elements are used in Perform. All material modelings are applied based on what detailed in Federal emergency management agency [6].

5. Design of base isolators

Seismic isolation means separating a part or whole of structures from other parts or the whole structure from the earth to mitigate seismic response of that part during earthquakes. Traditional methods of seismic design are based on increasing the capacity of structures. In these approaches lateral resistance capacity of structures are achieved by ductility or increase of resistance. Then dimension of elements and joints increase or braces and shear walls are used [7]. This phenomenon leads to increasing expenses. Contrary to above approaches, seismic-isolation lessens damages of structural and nonstructural elements by decreasing the seismic demand of structures. In this method the major period durations of structures are increased by isolators put under superstructure. Increasing the natural period of structures can often end up decreasing accelerations and forces on the superstructure with respect to the diagram of the acceleration spectrum. On the other hand the high time period leads to a system with low horizontal stiffness which can cause large displacements. This large displacement should be limited by increasing damping ratios as well. The high damping ratio also causes more energy dissipation. Finally seismic isolators should have enough vertical stiffness in order to bear and pass on weight of structures to the earth. As of the 19th century, there have been being continuous attempts to make instruments that can make all these goals befall. Different kinds of seismic isolators are outcomes of these attempts, like: rubber isolators, frictional isolators, lead rubber isolators (LRB), etc [8]. In this research LRBs are studied. LRBs-which are being used widely-consist of lead, rubber and steel plates that each of which causes high damping ratio, low horizontal stiffness and high vertical stiffness respectively. LRBs in this research are designed based on Guideline for design and practice of base-isolation systems in buildings No. 523 [9]. It is worth mentioning that the formulization of this guideline is very similar to the formulization of International Building Code IBC 2000 (International code council). Firstly, natural period of the system and its damping ratio should be assumed based on engineering judgment, catalog of producers and recommendations of codes. Natural period of seismic isolator systems shouldn’t be less than three times the period of fixed-base structure tantamount with the superstructure. The effective damping ratios of both base-isolated systems are assumed 10%. The target natural period of the moment-resisting frame is considered TD= 3 (s) and the one of braced frame is TD= 2 (s).

The effective horizontal stiffness (Keff) is:

$$K_{eff} = \frac{W}{g}\left(\frac{2\pi}{T_D}\right)^2 \quad (2)$$

Where W is general weight of the superstructure, TD is target natural period of the system and g is the Gravity acceleration.

Design displacement of the system is calculated from below equation:

$$D_D = \frac{g}{4\pi^2}\frac{A\times(S+1)Ts\times T_D}{B_D} \quad (3)$$

Where BD is the damping coefficient calculated in proportionate to βeff (effective damping).

The initial yield force of system Qd is:

$$Q_d = \frac{\pi}{2} k_{eff} \times \beta_{eff} \times D_D \quad (4)$$

The secondary stiffness of the system k_2 is:

$$K_2 = k_{eff} - \frac{Q_d}{D_D} \quad (5)$$

Table 3. Specifications of Base isolators

Isolator	T_D(s)	β_{eff} (%)	D_D(m)	K_{eff}(KN/m)	Q_d(KN/m)	K_2(KN/m)
Moment-resisting frame	3	10	0.33	12.372	0.64	10.43
Braced frame	2	10	0.23	28.04	0.969	23.63

It is worth saying that in this study the rehabilitation objective is the Basic Safety objective and the information level is the minimum level. In this rehabilitation objective, a structure under the design base earthquake should have Life safety (LS) as its performance level and under the Maximum credible earthquake should have Collapse prevention (CP) as its performance level either [6].

6. Study Methodology

6.1. Choosing appropriate IMs and EDPs

Earthquakes can be measured by various kinds of criteria, like: strong-ground-motion duration, magnitude, power spectral density, intensity and so on. Each of these quantities has some advantages and disadvantages. Then each of them cannot be a complete representative of earthquakes and their effects on structures, unless they are considered in relation with other criteria. For example earthquakes with the same PGA (Peak ground acceleration) may have different influences on a specific structure. Then other premises like: frequency content or magnitude should be considered as well to evaluate responses.

Moreover, the mentioned parameters just consider the excitation power of earthquakes with regardless of structures traits like: damping ratio, natural period of structure and the effects of mode shapes. One parameter which can show the ability of exciting of earthquakes and the excitability of structures is the spectral acceleration of the first mode for a given damping ratio. This intensity measure can be scaled with different scale factors as well as other parameters like: peak ground acceleration or peak ground velocity, which is essential factor in the IDA method. These characteristics lead to using the spectral acceleration of the first mode as an intensity measure in this Paper.

The damage measure (DM) or the engineering demand parameter (EDP) is a positive quantity which shows a structural response under seismic loads. There are a lot of options for this criterion like: maximum base shear, maximum interstory drifts, rotation of joints, maximum displacement ratio of roof, different damage indices (e.g.

Park–Ang index, cumulative hysteresis energy and Mehanny–Dierelein index (2000)), etc. the appropriate EDP can be chosen based on the kind of structures or the objective of analyses. Two or More EDPS should be considered simultaneously usually. For example, when the behavior of non Structural elements is important, some parameters like interstory drifts or absolute accelerations of stories become important. It is worth reminding that one of the primary objectives of seismic-isolators is to control the behavior of non structural elements by decreasing the two–mentioned parameters simultaneously. Generally, one way to check the design of base–isolators is to find whether the amount of interstory drifts or storey acceleration is decreased significantly by the base-isolation or not. For this reason, one of the studied EDPS in this Paper is the maximum interstory drift. Moreover, in case of not having any architectural and space limitations, the damage of base–isolated structures can befall because of the rupture of base–isolator. This phenomenon can be out of either high horizontal or vertical displacement of base isolators (the latter happens in case of having near fault records).Then, a damage measure for this kind of structures is the ratio between the maximum displacement of the base isolation and their allowable amount.

6.2. Incremental dynamic analyses

The IDA method is a rather new approach in the performance–based earthquake engineering, which can predict the seismic demand of structures in different levels of excitations comprehensively. Although the IDA method can be done with one record scaled by a set of scale factors, this approach cannot show the Structural behavior completely for future possible earthquakes. Because seismic responses vary record-to-record significantly. FEMA 440 [10] (Federal emergency management agency) recommends a set of records to be used in the IDA method. In this paper these records are discarded and other sets of records which are compatible with the seismic situation of the district are selected. The reason of this issue is that this set of records is the basic records to generate ET functions. Then comparing the results of ET functions with the results of this set is not a valid approach to verify the ET method.

Firstly twenty records were selected based on the seismic situation of the area and then they were refined in terms of having various frequency or energy contents. The reason is to prevent from having similar records with similar effects on models. In that, if selective records had similar energy or frequency contents, either high or low energies, the results of the IDA method would be less or further than those of the ET method unnaturally and unreasonably. The selected records and their attributions are shown in Table. 4. Due to the fact that planner frames are analyzed, from each pairs of records, only a record which has higher PGA is considered.

Table 4. Description of the set of ground motions used in this study

Date	Earthquake name	Magnitude (Ms)	Significant duration(s)	PGA(g)	Abbreviation
1999/09/20	Chi-Chi, Taiwan	7.6	13.11	0.413	Chi-Chi P1425
1992/04/25	Cape Mendocino	7.1	15.34	0.385	Cape Mendicino P0810
1989/10/18	Loma Prieta	7.1	5	0.357	Loma Prieta P0764
1966/06/28	Parkfield	6.1	5.1	0.357	Parkfeild P0034
1971/02/09	San Fernando	6.6	14.5	0.324	San Fernando P0056
Date	Earthquake name	Magnitude (Ms)	Significant duration(s)	PGA(g)	Abbreviation
1980/06/09	Victoria, Mexico	6.4	8.6	0.621	Victoria, Mexico P0266
1987/10/01	Whittier Narrows	5.7	6.58	0.396	Whittier Narrows P0630

IDA curves are curves which show EDPs of structures in relation with IMs. IDA curves usually follow a function like EDP=F (IM) - for a unique and incremental IM. Accordingly, IDA curves can be averaged and smoothed with moving averaging or the other smoothers like Hastie and Tibshirani (1990)…. As mentioned in paragraphs above, the first–mode spectral acceleration (Sa (T, 10%)) is the IM used in this paper). To obtain this measure, firstly, PGAs of all records should be scaled to 0/1 g, 0/2 g … 3g respectively, secondly the first–mode spectral accelerations of each record are obtained and in each PGA they are averaged. At the end of this process each PGAs becomes equivalent to a first–mode spectral acceleration. As for EDPs, in each PGA, maximum interstory drifts and maximum displacement of base–isolators are reached and then averaged. This procedure should be continued till either the average of the maximum displacement of base–isolators passes their allowable amount or specific acceptance criteria are passed. Lastly, IDA curves are depicted as shown in Fig. 5 (This figure is time-history response and average curve of it).

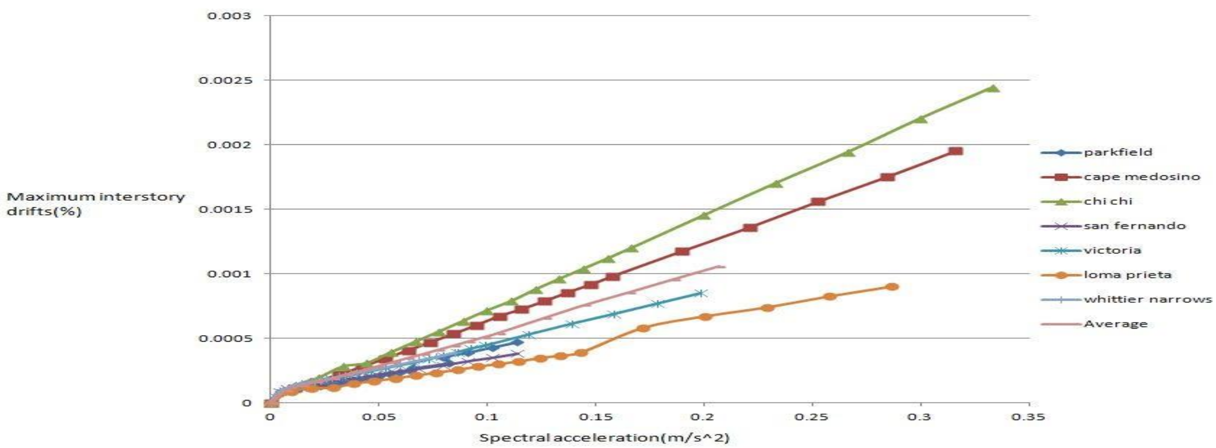

Fig. 5. Maximum interstory drifts of the moment-resisting frame with seven records and the average of them

6.3. The ET methodology

In the ET Method, Time is a potential intensity measure. In that, excitation Levels increase with Time increments absolutely. As mentioned previously, the Main objective of this study is a comparison between ET and IDA method, moreover in the IDA method first–mode spectral accelerations are considered as the intensity measure. Then in ET analyses, first–mode spectral accelerations should be regarded as the IM as well. A common characteristic of ET functions is that the averages of acceleration spectra of three records in each series vary linearly. With regard of this characteristic, firstly, acceleration spectra and their average should be drawn for t=0 till t=10 (s) [5].

Secondly, first–mode spectral acceleration with regard of natural periods and damping coefficients of models should be extracted from the average curve. This spectral acceleration is tantamount to the tenth second of spectral acceleration. In other seconds, equivalent spectral accelerations are calculated by linear interpolation.

The average of first–mode spectral acceleration of ETA20e (for the first 10 seconds):

Sa (T1, 10%) = 0/2113

The moment-resisting frame

Sa (T1, 10%) = 0/ 8227

The hinged frame

The average of first–mode spectral acceleration of ETA20en (for the first ten seconds):

Sa (T1, 10%) = 0/1984

The moment-resisting frame

Sa (T1, 10%) =0/7827

The hinged frame

As for engineering demand parameters (EDPs), after analyzing models with tree records of each mentioned series, their time–history responses should be averaged and absolute amount of those averages should be obtained. Then in each time, the maximum amount of absolute average responses should be regarded as the response of that time. In this stage horizontal lines appear in

ET curves which are precursor to restrengthening of models. As shown in Figs. 6, 7, 8 and 9. It is worth saying that in these diagrams the IM is still the time. Ultimately these diagrams should be smoothed by moving averaging with averaging radius equal to one second, in each time the EDP will be the average of responses in that time (node) and responses of hundred nodes (Times) before and after that time (in ET records Time steps are 0.01 second).

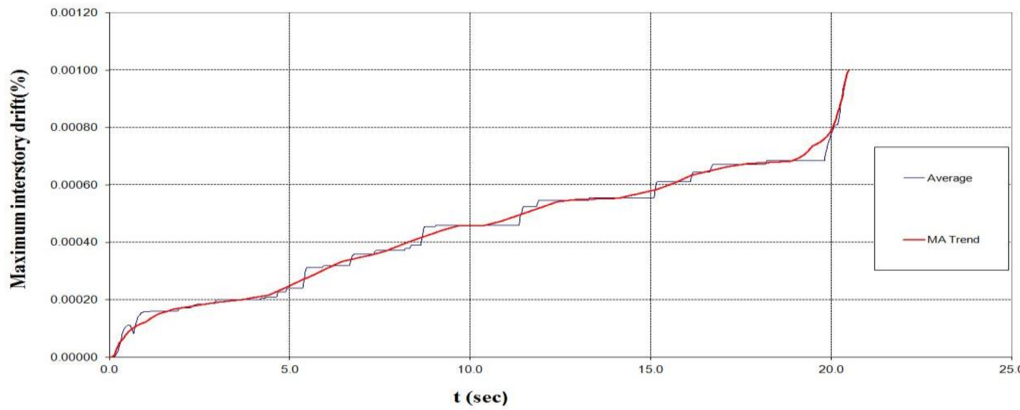

Fig. 6. The average and moving average of maximum interstory drifts of the moment-resisting frame with the ETA20e

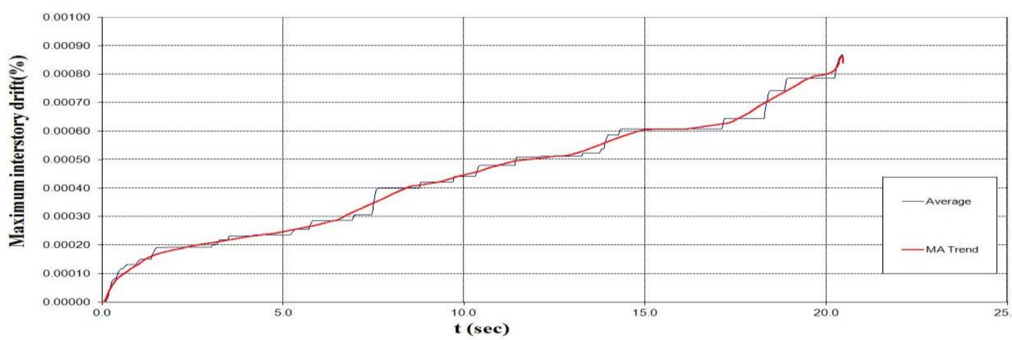

Fig. 7. The average and moving average of maximum interstory drifts of the moment-resisting frame with the ETA20en

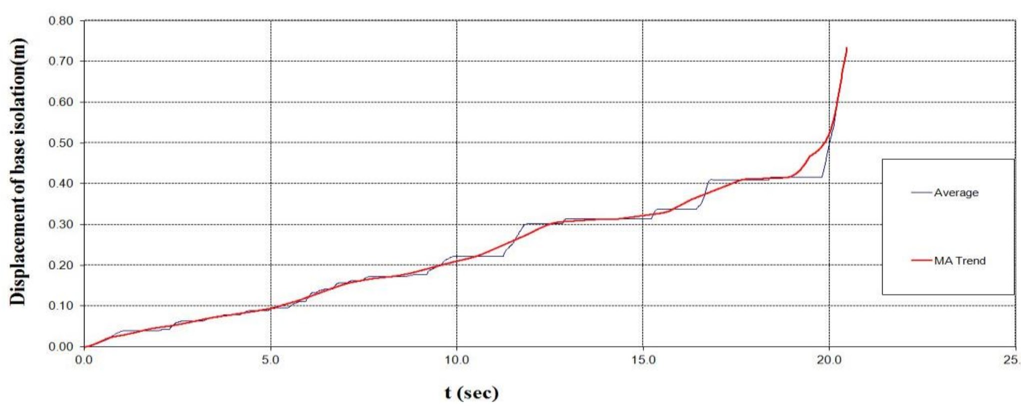

Fig. 8. The average and moving average of maximum displacement of the base isolators of moment-resisting frame with the ETA20e

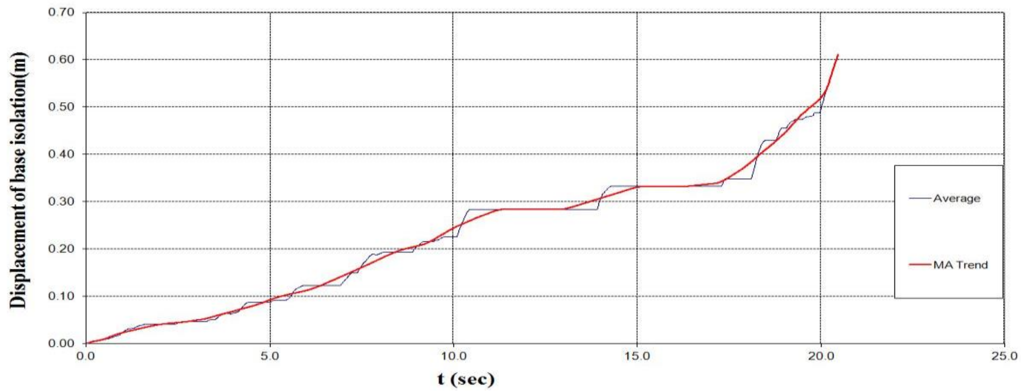

Fig. 9. The average and moving average of maximum displacement of the base isolators of moment-resisting frame with the ETA20en

6. Comparative study

It is not expected that ET curves coincide with IDA ones exactly, because, in the IDA method, responses change from one record to another significantly. It is hoped that by using enough appropriate records this problem gets solved. However, it is expected their results to have similar trends.

Figs. 10 and 11 show maximum interstory drifts for braced and moment resisting frames reached by ET functions and the IDA method. These results of ET functions are considerably close to those of the IDA method. The ETA20en series has closer responses to the IDA method in comparison to the ETA20e series. This phenomenon is seen in the both braced and moment resisting frames. Moreover, after T =19 (s) curves of ETA20e get vertical as shown in Fig. 10 (For braced frame) and in Fig. 11 (For moment-resisting frame). This vertical part shows instability of super structures which is unreasonable. Because superstructures did not have considerable nonlinearity which low quantities of maximum interstory drifts validate this point. Maximum interstory drifts in high intensity measures are about one percent, which is far from drift quantities which cover the collapse stage. Besides, low quantities of maximum interstory drifts prove the preconception that assumes base–isolated structures as SDOF systems. It also shows that super structures do not face significant nonlinearity.

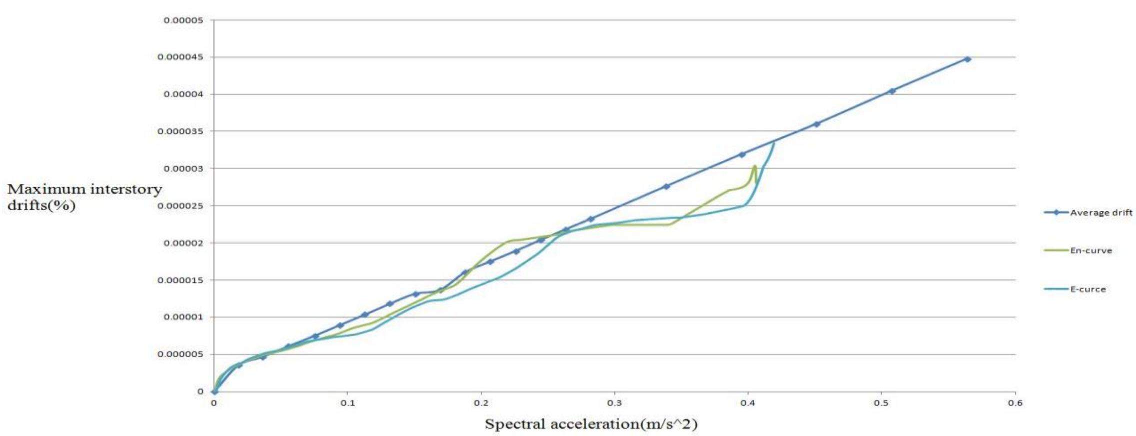

Fig. 10. Comparing maximum interstory drifts of the braced frame results of the ET method and IDA

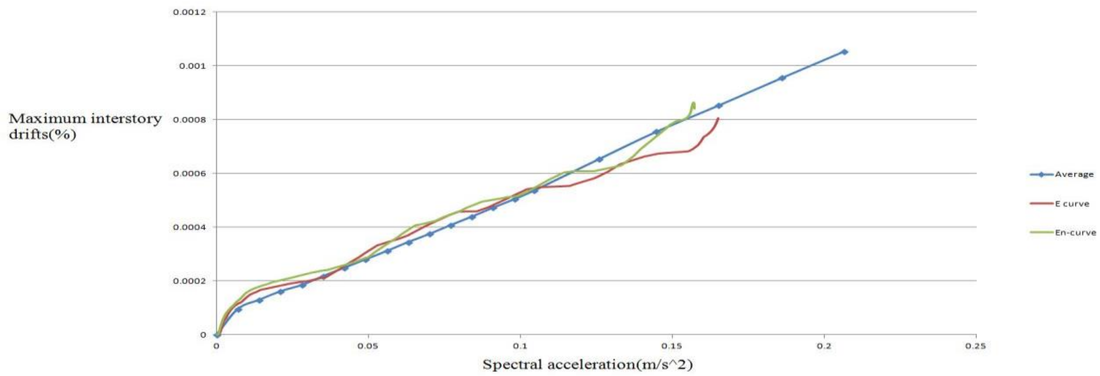

Fig. 11. Comparing maximum interstory drifts of the moment-resisting frame results of the ET method and IDA

Figs. 12and 13 show maximum displacement of base isolators calculated by ET functions and the IDA method. Generally, ET functions have different trends in comparison to the IDA method. It seems that ET curves are more reasonable than IDA curves, because they show linear behavior, yielding and nonlinear behavior of base isolators constantly. The reason of this phenomenon is that ET functions have a continuum nature because their (time) steps are 0/01 (s) which is very low. This enables them to cover all parts of structural behavior. But in this study PGA steps are 0/1g which is not low enough and it cannot consider structural behavior within PGA steps. A solution for this problem is to decrease PGA steps, which increases the number of analyses significantly [4].

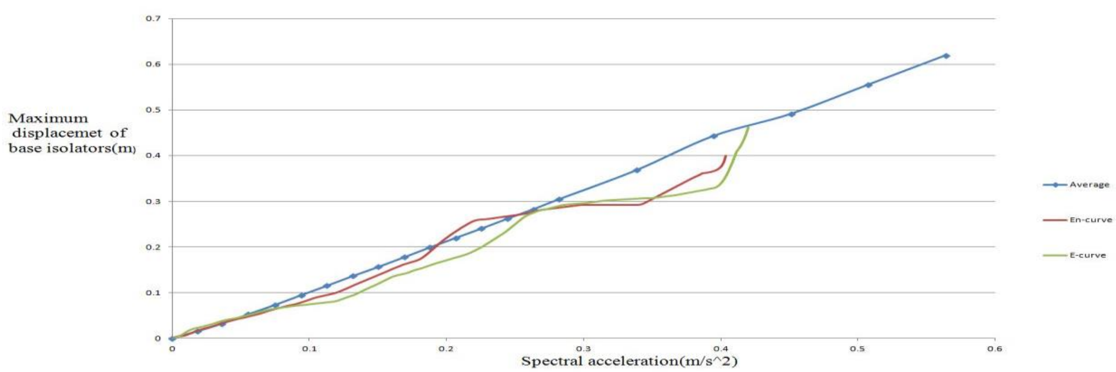

Fig. 12. Comparing maximum displacement of isolators of the braced frame results of the ET method and IDA

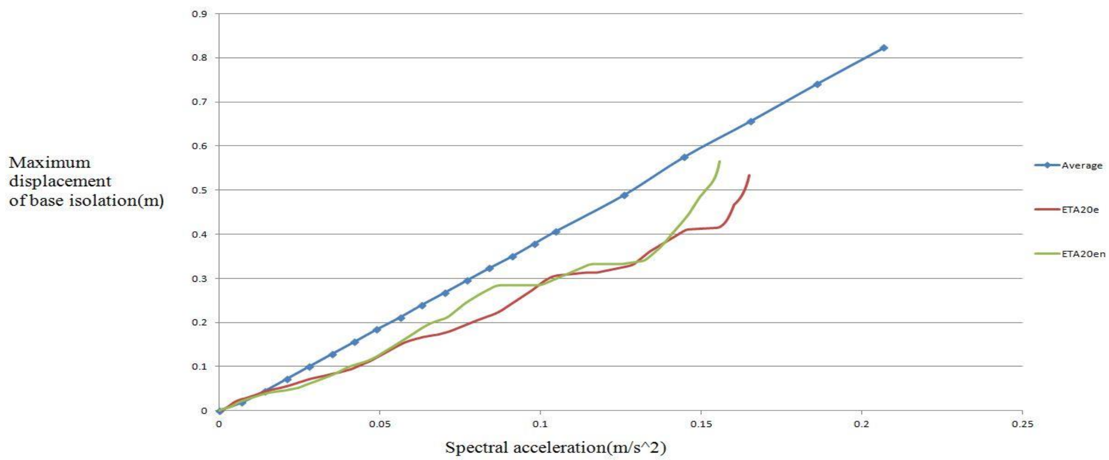

Fig. 13. Comparing maximum displacement of isolators of the moment-resisting frame results of the ET method and IDA

7. Conclusion

- In this study the ET method is compared with the IDA method on existing-steel structures which are rehabilitated by base isolators. Interstory drifts and maximum displacement of isolators are considered as engineering demand parameters (EDPs) and spectral acceleration of the first mode is considered as the intensity measure (IM). Using moment and braced frames which are rehabilitated by base isolators; it is shown that:
- ET can estimate results of the Incremental dynamic analysis (IDA) with acceptable accuracy. While the IDA would require several nonlinear dynamic analyses under multiple suitably scaled ground motion records, the ET can practically estimate the seismic behavior of simple to complex systems only by few nonlinear time history analyses. This method has simplicity and speed of static analyses along with the accuracy of dynamic analyses and can potentially be a useful tool for evaluating and designing seismic resistant structures.
- In intense ground motions, super structures do not show considerable nonlinear behaviors and interstory drifts because of energy dissipating by base isolation.
- ETA20en records have more accurate and convergent results in comparison to ETA20e records to estimate the behavior of either superstructures or base isolators.
- The more the period of structures, the more errors and divergences are seen in the results of the ET method -in comparison to the IDA method.

REFERENCES

[1] Estekanchi, H. E. (2012), The Manual of The Endurance Time Method, (The 3rd edition), Faculty of civil engineering, Sharif University of Technology, Tehran, Iran.

[2] Riahi, H. T. and Esekanchi, H. E. (2010), "Seismic assessment of steel frames with the endurance time method", Journal of Constructional Steel Research, 66, 780-792.

[3] Estekanchi, H. E., Valamanesh, v. and Vafai, A. (2007), "Application of endurance time

method in linear seismic analysis", Engineering Structures, 29, 2551-2562.

[4] Izadifard, R. A., Arshadi, H. and Estekanchi, H. E., (2013) "The comparison between the incremental dynamic analysis and endurance time method on steel structures which are rehabilitated by base isolators" (M.Sc. Dissertation), International Imam Khomeini University of Qazvin, Qazvin.

[5] Estekanchi, H. E., Arjomandi, k. and Vafai, A. (2008), "Estimating structural damage of steel moment frames by endurance time method". Journal of Constructional steel Research, 64, 145-155.

[6] FEMA-356(2000), Prestandard and commentary for the seismic rehabilitation of buildings, California, Redwood City.

[7] Sayani, P. J. and Ryan K. L. (2009), "Comparative evaluation of fixed-base and base-isolated buildings using a comprehensive response index", J. Struct. Eng. – ASCE, 135(6), 698-707.

[8] Sayani, P. J. and Ryan K. L. (2009), "Evaluation of approaches to characterize seismic isolation for design", J. Earthq. Eng.,13(6), 835-851.

[9] Guideline for design and practice of base-isolation systems in buildings No. 523 (Office of deputy for strategic supervision bureau of technical execution system (2007), Tehran, Iran

[10] FEMA 440(Federal emergency management agency) IMPROVEMENT OF NONLINEAR STATIC SEISMIC ANALYSIS PROCEDURES, Applied Technology Council.