An overview on the concepts and methodologies of incremental dynamic analysis IDA (with a single record and multiple records)

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Abstract:

Incremental dynamic analysis is a seismic analysis of structures based on the structural performance which states the behavior of the structures in a range of different intensities of earthquake. Due to the dynamic and non-linear nature of the earthquake, certainly the results of this method in comparison to the other types of analyses are closer to the reality of structural behavior. However, this method is a time consuming method and like other kind of time history methods, is too dependent on the records. Moreover, selection of intensity measures and engineering demand parameters are important issues that should be discussed. In this paper, a review on the history and concepts and how to perform incremental dynamic analysis (IDA) are discussed with a single record and multiple records.

Keywords: Incremental dynamic analysis; Intensity measure; Engineering demand parameter; Seismic Vulnerability; levels of performance

1. Introduction

The increasing growth of computer processing power may enhance the accuracy of analysis results by using more complex methods. As a result, analyses are shifted from linear static to linear dynamic, nonlinear static and nonlinear dynamic. For the latter, nonlinear dynamic analysis, is usually designed to control structures, structural analysis with one or more records to obtain one or more single point. On the other hand, in the methods like the Push-over method (ATC, 1996) (SPO) or the capacity spectrum method (ATC, 1996), With scaling the representative of static load, a continuous picture of the behavior of structures in all domains of the elastic mode to the yielding mode and eventually collapse of the structure, are given to us.

As we pass through a single static analysis we reach to the Pushover analysis, similarly by the development of time-history analysis, we only reach a few history-time analysis by which the seismic loads are scaled. Firstly, the concept of this method is expressed by Bertero(1977) and is used by several researchers later.

This method has accepted by the guidelines of Federal Emergency Management Office. #1) Ph.D. Candidate.
Agency (FEMA) as The Incremental Dynamic Analysis (IDA) and can be used as a method in order to determine the potential collapse capacity of the entire structure. IDA is already a widely used and versatile method which only some of its objectives are:

1. Full understanding of the response or demand of the structure in a vast range of different levels of ground motion records.
2. Better understanding of the structural effects at different levels of the ground motion with less or more power.
3. Better understanding of changes of the nature of structure response with increase of the intensity of ground motion (e.g.: changes in maximum displacements in height, beginning of reducing the hardness, strength and their models).
4. Evaluation of the dynamic capacity of the entire structural system.
5. Finally, surveying IDA curves with multiple records and how to remain constant …

2. IDA concepts with a record:

As a first step, it is necessary to briefly explain the different terms of this method:

We suppose that we have an accelerogram which is chosen from the accelerogram database of earthquakes as the first step. (Seismologists may reform the baselines, filter and rotate them before). Unscaled accelerogram \( a_1 \) is a vector with members: \( a_1(t_i), t_i = 0, t_1, ..., t_{n-1} \). To determine the effect of stronger and weaker earthquake, a simple transfer with a unified scale is introduced that scales the amplitude of accelerograms by \( \lambda \) as a scale factor to high and low. It can also be done by scaling the linear acceleration spectrum with \( \lambda \) as a coefficient or scaling the amplitudes in all frequencies with \( \lambda \) in the frequency domain.

![Figure.1 The IDA curve with six converged points in nine-story steel structure (the Vamatsikos et al. 2002) [1]](image-url)
Scale factor (SF) of an unscaled accelerogram $a_1$, is a non-negative integer $\lambda \in [0, +\infty)$, when multiplies to unscaled accelerogram $a_1$, makes $a_\lambda$. It is worth saying that SF makes a one by one projection from real accelerogram to all scaled projection of itself. The amount $\lambda = 1$ represents a natural accelerogram and $\lambda < 1$ representing the accelerogram with smaller scale and $\lambda > 1$ is indicative of a larger scale.

Although SF is the simplest way to show the scaled images of accelerogram, but it is not an appropriate device for engineering purposes and does not show enough information about the actual power of scaled records and its effect on the structure. For more applied usage, a criterion can be defined which shows the SF in a one by one form and shows a better correlation with the destructive potential of that scaled record. However, this idea can too optimistic.

Intensity measure (IM) of a scaled accelerogram, $a_\lambda$, is a non-negative integer IM $\in [0, +\infty)$ that follows $IM = f_{a_\lambda}(\lambda)$ relation and depends on the unscaled accelerogram, $a_1$, which increases monotonously with scale factor $\lambda$. While numerous quantities have been proposed to determine the intensity of the earthquake record, it is not always obvious how to scale them. For example, we can regard the moment magnitude, duration of earthquake and modified intensity measure of Merkaly. These quantities can be recognized as unscaled quantities. Typical examples for the scalable IMs are: peak ground acceleration (PGA), peak ground velocity (PGV), spectral acceleration of the first mode of the structures $Sa(T_1, 5\%)$ with 5% damping ratio and normalized factor $R = \frac{\lambda}{\lambda_y}$ (which is the smallest scale factor that causes the structure yield under a specific record). Which is numerically equivalent to the yielding reduction factor; $R$. On the other hand these IMs also have the feature of proportionate to SF, in the terms of $IM_{prop} = \lambda \cdot f_{a_1}$. Besides, the $S_{ax}(T_1, x, b, c, d) = [S_{a}(T_1, x)]^b \cdot [S_{a}(cT_1, x)]^d$ quantity which has offered by Mehanny-Dierlein and Shome-Cornell, is proposed and monotonous. However, it is not proportionate unless $b + d = 1$ numbers of non-monotonous IMs such as SDOF non-linear displacement of nonlinear systems is presented by Loco and Cornell, but they are not focused on in this paper and moreover, our purpose of IM, will be non-scale and monotonous from now on.

Destruction Measure (DM) or the Engineering Demand Parameter EDP is a non-negative integer $EDP \in [0, +\infty)$ that shows response of structure and seismic load. In other words, EDP is an observable quantity which is relevant to a part of the output results of nonlinear dynamic analysis or it can be concluded from them. Different choices such as the maximum base shear, rotation of the nodes, Maximum ductility of the stories, different damage indicators (such as the general or partial total cumulative stored energy, Park-Ang index general, stability index proposed by Mehanny-Deierlein, maximum relative displacement of the roof, the maximum relative angle of displacement $\theta_1, \theta_2, ..., \theta_n$ for a n-story building, or maximum of these values $\theta_{max} = \max(\theta_1, ..., \theta_n)$ can be regarded as EDP. Choosing appropriate EDP depends on usage and chosen structure itself, two or more EDPs (which are resulted from the same nonlinear analyses) are perhaps used to state different characteristics of answers, limited states or the modes of failure in PBEE. If it is necessary to evaluate the failure
of non-structural members in a multi-story frame, certainly, the right choice will be the maximum acceleration of stories. On the other hand, as for the structural failures of the frame structures - \( \theta_{\text{max}} \) - Expresses well the rotation of the joints and local and general collapse of the structure. So it is a good option for EDP. In the latter case, instead of using the effective relative displacement which is obtained based on the rotation of the structure, the relative displacement expressed on the basis of the ultimate relative displacement is used for EDP in various cases.

Structural response is often symptomatic number, and yet usually either absolute values or the magnitude of the positive and negative segments separately are used to check the structure. IDA with a record is a dynamic analysis which checks the desired structural model with the scaled time-history of the earthquake. In simple words, the incremental dynamic analysis or the Dynamic Push-over (DPO) is a set of non-linear dynamic analyses which are done based on scaled images of an accelerogram. IMs are provided in such a way that eventually cover all elastic to non-linear and finally collapse areas of the structure. The purpose of IDA is acquiring EDPs of structural model at each level of IMs for the scaled earthquake and usually the amounts of response of structures in accordance with the intensity are drawn as continuous curves.

The IDA curve represents the behavior of an EDP variable in a study of IDA for one or more IMs which is determining the used scaled accelerogram. IDA curve can be drawn in one or more dimensions, depending on the number of IMs which obviously one of them must be scalability. Like any standard engineering methods, such a figure often is drawn up and down; the IM is the independent variable, which acts like force. And at the vertical axis like the stress-strain diagrams, variation of force and load is drawn (Figure 1). Categorically, the results of IDA can be expressed for different IDA curves depending on the chosen IM and EDP.

2.1 The general characteristics consideration of IDA curve

In fact, the investigation on the IDA is the study of the characteristics of structural-model and accelerograms; when a model is analyzed under different earthquakes, it often gives non-similar responses, which is hard to foresee them from before, For example, according to Figure (2), which are the results of a braced five-story frame, A various responses from gradually deprecation of hardness to failure and fast behavior, non-monotonous, front and rear progressive torsion can be seen. Each figures represents the applying demand of each earthquake records to the structure at different intensities and their similarities and disparities are quite deceptive.

All curves show the linear elastic region as a separate figure that almost finishes at \( \theta_{\text{max}} \approx 0.2\% \) & \( S_a^{\text{yield}}(T_1,5\%) \approx 0.2g \), namely when the first brace buckles. In fact, each structural model with linear elastic members shows up such a behavior as far as the first member arrives into the nonlinear region. EDP / IM slope in this region of the curve IDA, the specified elastic stiffness of the EDP and IM is read. Usually this amount varies somewhat from a record to another record, but in the single degree of freedom systems, with the various records it is as the same and even for a multi-degree of freedom structure, if the effect of higher modes on the IM is considered, the same result will be seen again.
Due to the other end of the curves in figure (2), it is observed that how the curves come to an end in different levels of IM. The curve (a) suddenly becomes straight after its first buckling and it indicates huge relative displacements and sudden collapses. On the other hands, the curves (c) and (d) are ragged around the elastic slope; similarly all these curves behave based on the equal displacement rule. It means in structures with the mean period proportionate to total nonlinear displacements, usually are about elastic model displacements. Rotating pattern which the curves (c) and (d) indicate, consist of progressive parts from softening in regions in which local slope or hardness with IM increases more. In technical words, it means sometimes the rate of accumulation of EDP accelerates in structures and sometimes this acceleration decreases which this decrease can be powerful enough to avoid accumulation of EDP instantaneously or even move it back. Thus IDA curve locally goes to relatively lower EDPs and give us a non-monotonous function of IMs (fig.2 (d)). Sometimes with assuming that the structure is allowed to have a little collapse mechanism and the used EDP can consider it, when EDP is accumulated with a higher rate in the structure, the final straightening region happens which indicates the beginning of dynamic instability. Similarly, this phenomenon is defined in the static instability, in which with a few increase in IM, the deformations change to infinity. Then the curve becomes straight at the maximum value of IM where IM goes to straight line and EDP to infinity (fig.2 (a, b)).
Although the indicated examples are based on $\theta_{\text{max}}$ and $S_a (T_1, 5\%)$ values, such behaviors are visible for vast choices from IMs and EDPs.

The hardening of IDA curves are not only an anecdotal observation but also have been reported even for elastic and simple fully-plastic two linear systems by Chopra previously. As a result it is interpreted that if a system responds more strongly in a specific intensity measure, it maybe can show even a similar or less response and huge hardening if a structure is put under sever earthquakes. But the pattern and the length of time are the ones produce more differences than the intensity. As the accelerograms are scaled to top, the cycles of poor responses in the earlier parts of the time-history response get strong enough to inflict damage or strong yielding and thus change the characteristics of structure for subsequent stronger cycles. For multi-story buildings, a stronger earthquake can cause sooner yielding which can work as a fuse that eases topper story as shown in Fig. (3). Even simple oscillators when yield earlier in a cycle, their responses may be less in the next cycles, while their previously EDP values have been higher (Figure 4), and perhaps this is due to the elongation of period of the system.

![Figure.3 IDA curves for maximum relative displacements of different levels of a five-storey building (the Vamatsikos et al. 2002) [1]](image-url)
Figure 4 The ductile response of a structure with one degree of freedom in different levels of intensity (from Vamatsikos et al., 2002) [2]

Similar phenomena for structural restrengthening (a severe hardening), in which a system in some IMs has driven to a total collapse (it means the analysis cannot be
converging and EDPs are infinite). But in some higher intensity levels, a higher response and non-destructive reappears (Figure 5).

![Diagram](image)

**Figure 7** Restrengthening of the structure in the IDA curves for a three-story building (the Vamatsikos et al. 2002) [1]

As the complexity of two-dimensional IDA curve has been seen, it is natural to investigate the characteristics in an analytical way. By assuming a monotonous IM, the IDA curve is a function with this form \((0, +\infty) \rightarrow (0, +\infty)\). It means every IM gives an EDP value while for a specific EDP at least there is one or more IMs (in non-monotonous IDA curves) and therefore the figure is not necessarily one-by-one. Moreover, since the EDP is often defined as the maximum value or consist of fixed value responses, IDA is not necessarily a smooth curve and makes it indistinguishable from the definition and may include a limited number of points of discontinuity caused by some collapse separations and be the next restrengthening point.

### 2.2 The capacity and limit states in any one of the IDA curves

Function levels or limited states are major components of performance-based earthquake engineering and the IDA curve contains necessary information to obtain
them. But it is necessary to explain each of them briefly that how IDA curve gives them, namely when with a statement or a rule that it is satisfied shows reaching regarded limit stated. For example, immediate occupancy (FEMA, 2000a and b) is a performance level of the structures which is fulfilled by reaching to a certain amount of EDP (usually based on the maximum relative displacement of stories), while the overall collapse (at least in FEMA350 and FEMA 2000a) is mentioned based on a IM or EDP that dynamic instability is observed. The issue is occurred when several points satisfy such conditions, then the question is what should be done (Fig. 6)? Or which point should be selected?

The reason that several points can satisfy the terms of limited states is because they are related to re-hardening of structures in the severe cases. Usually, in conservative case the lowest value of IM which shows the limited state is selected. Extending this concept to all of IDA curves means that we disregarded the upper part of the first straight line and we have studied just the points above the first indication of dynamical instability.

It should also be noted that all these discussions are in order to acquire the dynamic instability to the numerical instability to predict the collapse. Obviously not being convergent of time integral sometimes is the safest and perhaps the only numerical equivalent to the true dynamical collapse phenomenon, but like all models, Depending on the quality of numerical program, integration steps and even rounding the error can be annoying. Therefore, we assume that such events are considered as well as possible and suitable predictions had made. This allows us to advance the many of the basic rules used to define the limit states. The first rule is based on the EDP that can be expressed as follows: if \( EDP \geq C_{DM} \) is the limited state satisfied (Figure 6). Usually the basic concept is that EDP indicates collapse and when it is lower than a specific value, it is assumed that the structure is at the limit state.
These $C_{DM}$ values can be acquired by theoretical or engineering experiments, and perhaps they have no particular value and just have a probability distribution. For example, the limit $\theta_{\text{max}} = 2\%$ reflects the function level of immediate occupancy structures for Steel Moment Resistant Frame (SMRF) with the first type of connection in FEMA guidelines. Also a similar method like this which is used by Dieirlien and Mehanny (2000), is the other case that a specific damage measure is used as a EDP and when the amount is larger than the unit, it is assumed that the collapse occurred. Such limits may include randomness, for example, FEMA350 states collapse limit states by $\theta_{\text{max}}$ which causes that the rotation of joints for destroying the capacity of gravity load carryings gets enough. This effect as the random variable of experiments, analysis and judgment for each connection type is defined. Even though a same amount of $C_{DM}$ may show several limit states points on IDA curves (Fig. 6). This ambiguity can with a certain way (for example the definition of limit state point as the littlest IM in a conservative way), or with an appropriate understanding of the various areas of satisfying or not satisfying the limit states can be solved. The rules e based on the EDP had advantage of simplicity and understanding, especially for non-collapse levels. In the collapse level, they may actually be a sign of weakness of the model. If the model is realistic enough, we need to explicitly contain such information, ie, with a little non-convergence instead of a limited EDP show the collapse level. They also have the advantage of being consistent with other less serious cases which are somewhat recognized by EDP.

Another option that is a rule based on IM, as initio because of the need of obtaining
better collapse capacity by a point on the curve made by IDA, the curve separates into two distinct areas, one without collapse (smaller IM) and one in collapse (bigger IM). As for monotonic IMs this rule can be expressed as follows: if \( IM \geq C_{IM} \), the limit state is passed (Figure 6). The main difference with the previous case is the difficulty of definition of \( C_{IM} \) that shows the collapse for all the IDA curves, and thus it has to be done for each curves separately. However, it has the advantage that it specifies the collapse region and the difficulty of definition of this point for each curve is its disadvantage. In general, such a rule resulted in the capacity based on both IM and EDP. A special case is regarding the final point as capacity (i.e.: with usage of the smallest straight line to determine capacity based IM). All IDA curves till the first sign of dynamic instability are regarded as non-collapsed.

The tangent slope 20 percent method of FEMA (2000a) is based on IM and in this method the last point on the curve with tangent slope equal to 20% is considered as the capacity point. The idea is based on the notion that the straightening of smooth curves is an indication of dynamic instability (i.e.: EDP increases more quickly and goes to infinity). Because EDP cannot be possible, the place in which \( \theta_{\text{max}} \) increases 5 times as much as the initial velocity is regarded as the capacity point. Because of the wave shape of IDA curves, several points with this attitude can be observed, which shows that the structure moves towards collapse and then it should be noted that if this behavior is seen for higher IMs or not (as in Figure 6). In fact, the lower points should be disregarded from the choices of capacity points. As mentioned above, IDA curves are partially smooth, but the approximation of tangent slopes can be fined by uniform interpolation. With Pessimistic vision it can be interpreted as a part of curve points which can be an appropriate approximation of changing velocity.

Simple rules listed above are the main framework to build combination rules, namely the logical expression with a combination of the above types that many of them are linked with a logical OR operator. For example, when a structure has different failure modes that are not detectable by an EDP, to find the total collapse state using the OR statement is useful. A simple example is offshore platforms which have yielding modes for pile displacements or soil in a specific displacement of deck, while the yielding of the braces at the maximum interstory displacement is more observable. That gives more specific. The first phenomenon (based IM) which occurred is determination of the collapse capacity.

The other case is total collapse capacity, which based on FEMA (2000a, b) is defined with the OR combination of a 20% slope of the IM -based method and the rule of \( C_{DM} = 10\% \) based on EDP that \( S_a(T1, 5\%) \) and \( \theta_{\text{max}} \) are IM and EDP options. If either of these two rules is reached, the capacity is defined based on the both. This means that the 20% percent stiffness seeks imminent collapse, however, the amount of 10% prevents from the high amount of \( \theta_{\text{max}} \) in areas that the model loses its accountability. As for what is stated about the definition of EDP, this definition may suffer from a lack of precision, because near to the area of the straight line, a vast range of EDPs can be related to a small range of IMs, then reaching the real EDP is sensitive to the trend of the IDA curve and the amount of 20%. On the other hand, if the definition is used the bases will be stronger. This phenomenon is a usual observation f of collapse capacity and specifies that the collapse capacity can be addressed based on IM better.
3. IDAs with a record and summarizes

As it has to be specified, The IDA curve cannot fully express the structural behavior for future events with a record. Since IDA would be greatly dependent on the selected records, therefore, sufficient number of records is required to cover all areas of responses. As a result, we have to analyze a structural model for a set of seismic records.

Studying of multiple records is a series of IDA studies with a record for a structural model under different accelerograms. Such a study will produce a series of IDA curves which can be plotted on a sheet by choosing the same IM and EDP. IDA curves, a set of IDA curves for a structural models under different accelerograms that all of them are parameterized for the same IMs and EDPs.

With regard of that each curves (which provides the characteristics of the record and structural model), has been defined in a complete algebraic form, if we want to consider the randomness according to the record that the structure can experiences, we have to consider the probabilistic characteristics. The IDA curve of specific structural models and various numbers of them is not algebraic, it is a line or a random function in the form of EDP = f (IM) (for a single monotonous IM). In conclusion, by providing mean, median, and 16% and 84% of range of response, as a sample, we can recap a set of records and thus we can define 16% and 84% of mean and median of IDA curves. According to above, for estimating two-dimensional random lines (with assuming the same IM), for example, we need methods which divided into two major categories.

The first category is parametric methods. In this case, a parametric model of the known EDP is assumed in the opposite of IM for providing a sample of parameter values (each line adjusts separately) and then statistical parameters will be reached. As an example, the two-parameter model of power law \( \theta_{\text{max}} = a[S_a(T_{1,5\%})]^2 \), presented by Shome and Cornell, which is provided based on documentary assumptions of conditional logarithmic distribution of known \( \theta_{\text{max}} \) for \( S_a(T_{1, 5\%}) \). This model often provides the most powerful explanatory curves which allow to obtain important analytical results (Jillyr and Cornell). This is the general feature of parametric methods; while these methods have little flexibility and precision to obtain each curve, they lead us to simple explanations.

On the other hand, there are non-parametric methods which basically use scattered figure smoothers like the moving average, moving median or smoothing curve of Hasti and Tibshirani. Perhaps the simplest of all, is moving median with a zero length window (or with sectional median) which simply contains the calculated values of EDP at each level of IM and then finding the mean and standard deviation of known EDP for a level of IM. This method acts well until where the first IDA curve reaches its capacity and when EDP moves to infinity. Unfortunately, most of these smoothers suffer from this kind of problem but with sectional median or with sectional percentage act stronger generally. Instead of calculating the means at each level of IM we calculate a sample mean and 16% and 84% percentages and they shift to infinity only when the collapse occurs at 50% and 84% and 16% of the records respectively. Another advantage is that, with appropriate assumptions (e.g.: continuity and uniformity of curves), linking line of the ratios of x% of known EDP for the similar IM linking line ratios \((100-x\%)\) percent of known IM for EDP. In addition, this method is well accordant with strong assumption of
logarithmic distribution of known $\theta_{\text{max}}$ for $S_a(T_1, 5\%)$ which is the median value and 16% and 84% ratios of the median are multiplied by $e^{\pm \text{dispersion}}$ (Jelyr and Cornell).

Finally, a variable has been suggested to show the collapse by Shome and Cornell that the conventional moments are used to identify the non-failures. Hence the infinities are deleted while collapse probability for known IM would be summarized individually with a logistic regression.

A simple but important problem is sum up the capacity of N-sample curves which are expressed based on either EDP or IM. Because there are neither random nor infinite lines, the problem can be reduced to a conventional statistical matter and we can measure means, standard deviation and ratios, as usual. Still logarithmic being of observed capacity data of using the median (either in the form of 50% ratio estimate or a non-Logarithmic average of logarithms) and offers standard deviation of the logarithms for the distribution. Indeed, when probabilistic calculations of limit states are done, there is necessity to address the potential dependence (or correlation) between demand and capacity.

4. IDA in PBEE

As an analytical method, the power of IDA is in the proper use within the framework of possibilities, where we focus on estimation of similar annual events which the demand passes limit state or capacitance C. This process is somewhat similar to passing a known limit state or performance level (e.g.: immediate occupancy or collapse prevention in FEMA) in a certain alternation period of time. Such calculations can be summarized within the framework of an accepted equation by the Pacific Earthquake Engineering Center, Cornell and Krawinkler (2000).

$$\lambda(DV) = \int \int \frac{G(DV|EDP) dG(EDP|IM)}{dG(IM)}$$  \hspace{1cm} (1-1)

Where IM, EDP and DV are the Intensity measure, the Engineering demand parameter and Decision variables respectively. Here we usually use numerical IMs and EDP the expected limit state. The decision variable is simply defined as a numerical variable:

If the limit state passes $DV = 1$ (and equals zero otherwise), $\lambda$ is the common risk curve, it means that annual frequency is an average of IM, for example, $x$ is passed.

$$|d\lambda(x)| = |d\lambda(x)/dx|dx$$ is its differential (it means that $|d\lambda(x)/dx|$ is density of average speed.) $|dG(EDP|IM)|$ is Differential supplementary cumulative distribution function (conditional) EDP for known IM or $f_{DM|IM}(y|x)dy$. In the previous sections, the statistical properties of random IDA curves were investigated. These distributions have exactly the same features of $|dG(EDP|IM)|$. Finally, in limit state, when at the left side of equation 1.1 we are looking for $\lambda(DV = 1) = \lambda(0)$, then if $F_c(y)$ the cumulative distribution function ‘C’ (namely statistical feature of capacity which had discussed at the end of the
previous section) is $G(0|EDP) = Fc(y)$, in total collapse state, the estimation of capacity is obtained from IDA analysis. Briefly, to store the feature of earthquake $\dot{\lambda}(IM)$, with a smart choosing IM, DM and structural model, IDA gives the necessary information for PBEE demand characteristics and the total collapse capacity feature accurately.

5. Scaling and IM choices

As discussed above, we see that a useful engineering point of view can be resulted from study of a levy of IDAs or with a single IDA. Although it is often expressed that concerns about the accuracy of obtained EDP results from records that have been scaled to up and down (something that is unusual in practice and in research). While it is not always expressed well that the concern is usually about analysis with weaker records rather than representative of a stronger records. Thus it can be said more carefully in the last two sections that the median (or any statistical index) EDP obtained from scaled records with different levels of IM are correctly the median of the population of non-scaled records with the same IM, because of the limitations of the data record (that a few records with known level of IM can be found). Since we are interested in a range of IM levels, it’s more complete and practical to ask: If the median function (like regression) of EDP against IM obtained from scaled records can be estimated from a similar obtained function of non-scaled records? In the papers, there is a lot of expressions for such questions. For example, a comparison of the results obtained in the investigation of Bazrov et al. (1998) in the figure (7) in which the two given regressions are very close together. But this is just one of the results is given there. For sufficiency it is necessary to say that the usual answer to this question depends on the structure, EDP, IM and desired population of samples. For example, the figure(7) for steel structures with the median period (one second), in which EDP is the maximum relative displacement of stories and the IM is the first mode spectral acceleration for typical groups of records, The answer is positive. On the other hand the answer to this case in which IM is only different from the previous cases and the PGA is the IM, the answer is no. Why? Because a structure which its first mode is dominant is extremely sensitive to the frequency content of first mode, which $S_a(T1, 5\%)$ considers the notion while PGA cannot. With changing the magnitude, the shape of the spectrum changes. Which also shows that the average ratio of $S_a(T1, 5\%)$ to the PGA changes along with the magnitude. Thus, the relative displacement of the scaled record median against the PGA depends on the ratio of different values of magnitude in the sample and maybe indicates such a curve for a population of specific magnitudes or not. On the other hand, the first mode spectral acceleration cannot work for long-period structures with high periods which are much more sensitive than short-period ones, because the spectral shape is dependent on the magnitude again.
There are various questions of accuracy, efficiency and practicality of choosing an appropriate IM for specific applications. But in general, it can be said that if IM is selected so that the regression of EDP on M, IM, and R is effectively independent on M and R (in the given area). Scaling the records is a good estimation of the distribution of EDP for the specified IM.
With further inquiries on the IDA, new ideas about the further questions to select the effective IM can be reached. For example, less dispersion in the EDP against IM shows that smaller record collection and less nonlinear-analysis are needed in order to estimate the median of EDP against IM. Then, the desired choice for IMs is less dispersion. Figure (8) shows IDA curves for flexural 6-story steel frame in which the EDP is the maximum relative displacement stories and the IM in the case (a) is PGA, in the case (b) is $Sa (T_1, 5\%)$. As the results based on IDA clearly show, less dispersion are along all EDP regions. In addition, the IDA for studying how specific IMs can speculate the collapse capacity is used. It is shown that the $Sa (T_1, 5\%)$ has more priorities than PGA, because the amount of IMs related to the straight part are less than the first part.

6. Conclusion

Incremental dynamic analysis is a seismic analysis of structures based on performance which states the behavior of the structures in a range of different intensities of earthquakes. Due to the dynamic and non-linear nature of the earthquake, certainly the results of this method in comparison to the other types of analyses are closer to the reality of structural behavior and earthquakes. In this paper, a review on the history and concepts and techniques of performing incremental dynamic analysis (IDA) are discussed with a record and multiple records. As it has been specified, The IDA curve with one record cannot fully express the structural behavior for future events. Since IDA would be greatly dependent on the selected records. Then the Studying of multiple records - series of IDA studies with a record for a structural model under different accelerograms is necessary. Such a study will produce a series of IDA curves which can be plotted on a sheet by choosing the same IM and EDP. IDA curves, a set of IDA curves for structural models under different accelerograms that all of them are parameterized for the same IMs and EDPs.

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