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# COMPARISON THE STRUCTURAL NONLINEAR RESPONSE UNDER TRADITIONAL IDA AND THE CONDITIONAL SPECTRUM-BASED SELECTED EARTHQUAKES

A. Nicknam[*], M. J. Asgari and E. Yousefi Dadras
Department of Civil Engineering, Iran University of Science and Technology, Tehran, Iran

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## ABSTRACT

Selecting sufficient number of appropriate ground motions compatible with regional tectonic condition is a major step in Incremental Dynamic Analysis method (IDA). This article is intended to compare and assess the nonlinear structural response of the selected four frames (two steel moment-resisting frames and two RC Special Moment Frames) through performing IDA method by use of the traditional and Conditional Spectrum methods. First, the CMS and CS approaches are briefly explained and discussed. The selected frames are dynamically nonlinearly analyzed using the traditional IDA and CS methods at four hazard levels (50%, 10%, 2%, and 1%). The plots of results obtained from the two approaches, in the form of mean curves, are compared and assessed. It is concluded that the CS method ends up with more reasonable results except in case where the frame poses structural instability as the result of using CS method. The necessity of performing more research work in this area is highlighted.

**Keywords:** IDA; spectral shape; ground motion selection; Non-linear dynamic analysis; CS.

## 1. INTRODUCTION

Incremental Dynamic Analysis (IDA) is a promicing analyzing approach that has recently emerged in several different forms to estimate more thoroughly structural nonlinear response under dynamic loads. The Peak Ground Acceleration (PGA) of a record has been a commonly used parameter as Intensity Measure (IM) in long years ago. The commonly used Uniform Hazard Spectrum (UHS) has been shown to be an unsuitable tool for this purpose, as it conservatively implies that large-amplitude spectral values will occur at all

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[*] E-mail address of the corresponding author: A_nicknam@iust.ac.ir (A. Nicknam)

periods within a single ground motion [1]. More recently, spectral response values (e.g. spectral acceleration at the first-mode period of vibration, Sa (T1)) have been used as the most appropriate form of IMs.

It is quite postulated that the dynamic response of structures is highly sensitive against ground-motion record used.. Notably, it is not possible to accuratly conclude the structural response by using only a single time history as the future probable earthquake ground motion at a particular site and logically deferent ground-motions having diferent earthquake characteristics significantly influences the response of structures. It is recognized that, the earthquake time histories used in dynamic analysis procedure should be compatible with the regional hazard curve otherwise the structural responses would be associated with a large uncertainties ending up with unreliable responses. For this reason, the standard code of practice (e.g., ASCE-7-2010) states that the dynamic nonlinear analysis of structures should be performed by selecting earthquake time histories that match the target response spectrum within the periods 0.2T to 1.5T [2]. Lots of questions arise in the process of selecting ground-motion records. It is important to note that the selected set of ground-motion records should reflect the seismic hazard at particular site. There are a variety of methods for selecting the suite of ground motions to be used in dynamic analysis and the selection method strongly influence the structural response. Traditionally, IDA is performed by arbitrarily selecting the suites of earthquake each of which is scaled to multiple levels of intensity being applied upon the selected structure until a targeted hazard level(e.g., 20% or 2% or performance level) is reached [3, 4].

Although the spectral acceleration at T1 has long been found to be an effective IM [5], records with the same Sa(T1) is still associated with significant variability in the structural nonlinear response of multi-degree-of-freedom structural model. This problem so called; "spectral shape effect", comes up with considerable bias in structural response as discussed by Baker et al. [6].A method to account for this spectral shape effect is through selection of a set of ground motions that is specific at the building's fundamental period on the site hazard characteristics. Recently an approach has been developed to select ground motions that match both a conditional mean and conditional variability [7]. The CMS was initially proposed with an emphasis on the mean spectrum and less attention was paid to the variability in the spectrum. This was to some extent acceptable because there was no simple way to select ground motions being matched with a specified variability [8].The quantitative investigations of ground motion scaling proposed by Baker et al. [7] indicated that, a suite of ground motions may be safely scaled to the suite's median spectral acceleration value, at a period T, without biasing the median response of a structure having the same first-mode period T [5]. However, recent work suggests that in some other situations record scaling may induce some bias in structural response [9, 10]. This bias appears to be as a result of scaled ground motions having inappropriate values of spectral shape or epsilon (ε), which is an indirect measure of spectral shape [6].

## 2. SCOPE AND OBJECTIVE

It is claimed that, the Conditional Spectra approach (CS) for selecting sets of ground motion proposed by Baker et al. [7] comes up with small earthquake intensities [8]. The objective of this article is to compare and evaluate the results of CS method with those of the traditional arbitrary selection approach by implementing the two methods upon the selected structures. The article starts with presenting an introduction on the problem followed by a brief description on the Conditional Mean Spectra (CMS) and Conditional Spectrum methods. In the next section, the problem is discussed by implementing the traditional and CS methods on the selected structures. The results of modeling and calculating the nonlinear responses are explained in the subsequent section associated with a comparison and discussion. Finally the conclusion conducted in this study are presented followed by the relevant references.

## 3. GROUND MOTION SELECTION

A common finding of record selection research is that, structural responses in dynamic analysis are dependent upon spectral shape, and that, if scaled ground motions have the same spectral shape, as the target ground motions, the resulting structural responses from the scaled ground motions are statistically similar to the responses from unscaled ground motions [9].

Although the two parameters, magnitude (M) and distance (R) can majorly affect the spectral shape of the records, the ground motion hazard parameter ε has also been found to be a useful predictor of the spectral shape [9]. A record's epsilon value (ε), given period T, represents the distance (the number of standard deviations) between the record's spectral acceleration to the mean value predicted by an attenuation relationship expressed as:

$$\varepsilon(T) = \frac{\ln Sa - \mu_{\ln sa}(M,R,T)}{\sigma_{\ln sa}(T)} \tag{1}$$

Where $\mu_{\ln sa}(M,R,T)$ and $\sigma_{\ln sa}(T)$ are the predicted mean and standard deviation of lnSa given period T respectively and lnSa(T) is the log of the spectral acceleration at the period of interest. The first two parameters are computed using ground motion models (also sometimes is called the attenuation models). A positive epsilon value corresponds to a record with larger spectral acceleration than the predicted for an event with the given magnitude M and distance R.

Figure 1 shows the acceleration spectrum of Loma Prieta ground motion illustrating the distinctive spectral shape of such a rare ground motion. This spectrum has a rare spectral intensity at 1.0 second of 0.9g, with the probability of exceedance 2% in 50 years. The predicted expected spectrum attenuation consistent with the event magnitude, distance, and site characteristics associated with this ground motion is also shown [11]. The figure shows that this extreme ground motion has a much different shape than the predicted mean spectrum. The spectrum has a "peak" from approximately 0.6 to 1.8 seconds associated with lower intensities at the other periods confirming the important role of the spectral shape of the ground motion record [12].

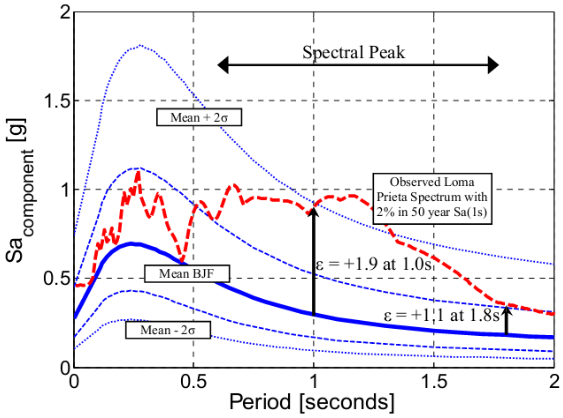


Figure 1. Comparison of an observed spectrum from a Loma Prieta motion with spectra predicted[11]

It has been recognized that records with large positive epsilon cause a systematically smaller response in structures than those records with small negative epsilon, under the same spectral acceleration. In Figure 2, the geometric means of the three record sets are displayed. As seen, the average shapes of the three records in Figure 2 differ even though each set has a wide range of magnitudes and distances [6]. It can be shown that, at long return periods site's ground motion hazard is dominated by large-epsilon events. These events cause a smaller response than the (on average) zero-epsilon records which is typically used to estimate the demand meaning that the estimation of structural response at long return periods is conservative when the traditional scalar IMs is used [9].

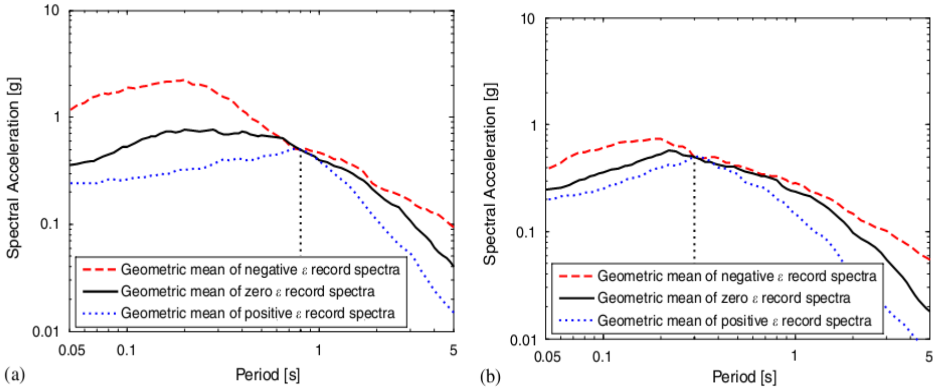


Figure 2. (a) The geometric mean of response spectra for negative-ε, zero-ε and positive-ε record sets, after each record's spectrum has been scaled to Sa(0.8s)=0.5g; and (b) the geometric mean of response spectra for negative-ε, zero-ε and positive-ε record sets, after each record's spectrum has been scaled to Sa(0.3s)=0.5g

## 4. A BRIEF EXPLANATION ON THE CONDITIONAL SPECTRA (CS)

In order to explain the CS approach proposed by Baker et al. [7], we have to first briefly explain the Conditional Mean spectrum method (CMS). The calculations involved in obtaining the CMS are simple. To summarize the approach in an easily reproducible format, a step-by-step calculation procedure is briefly presented hear.

**Step 1: Determine Sa at the first period of the structure and associated M, R, and ε**

Having the first period (T*) of the selected structure, Sa(T1) is simply calculated by means of the regional hazard curve. There after the corresponding ε(T*) is simply obtained. For example: If the target Sa(T*) is obtained from PSHA, then the M, R, and ε(T*) values can be taken as the mean M, R, and ε(T*) from disaggregation[6].

**Step 2: Compute the mean and standard deviation of the response spectra given M and R.**

Next, we compute the mean and standard deviation of log spectral acceleration values, $\mu_{\ln sa}(M,R,T)$ and $\sigma_{\ln sa}(T)$ at all periods, for the $M, R$, etc.

**Step 3: Compute ε at the other periods given ε ($T^*$)**

In this step we compute the "conditional mean ε" for all the other periods other than $T*$.The conditional mean ε at the other periods can be shown to be equal $\varepsilon(T*)$ multiplied by the correlation coefficient between the ε values at the two periods [6].

$$\mu_{\varepsilon}(T_i)\big|\varepsilon(T*) = p(T_i, T*)\varepsilon(T*) \tag{2}$$

Where $\mu_{\varepsilon}(T_i)\big|\varepsilon(T*)$ denotes the mean value of $\varepsilon(T_i)$, given $\varepsilon(T*)$. The user is recommended to used Equation (3) for calculating the correlation coefficient $\rho(T_i,T)$ between 0.05 and 5 seconds [15, 16].

$$p(T_{\min}, T_{\max}) = 1 - \cos\left[\frac{\pi}{2} - \left[0.359 + 0163 I_{(T_{\min}\langle 0.189)} \ln\frac{T_{\min}}{0.189}\right]\ln\frac{T_{\max}}{T_{\min}}\right] \tag{3}$$

Where $I_{(T_{\min}\langle 0.189)}$ is an indicator function equal to 1 if $T_{\min}\langle 0.189$ s and equal to 0 otherwise, and $T_{\min}$ and $T_{\max}$ denote the smaller and larger values of the two periods of interest respectively [6].

A more refined (but more complicated) correlation model, valid over the wider period range of 0.01 to 10 seconds, is also available [15], but equation 3 is nearly reliable equivalent value provided that only periods between 0.05 and 5 seconds are of interest.

**Step 4: compute the Conditional Mean Spectrum**

The CMS can now be computed using the mean and standard deviation from Step 2 and the conditional mean ε values from Step 3. Substituting the mean value of $\varepsilon(T_i)$ from Equation 2 into Equation 1 and solving for lnSa($T$) produces the corresponding conditional mean value of lnSa($T_i$) given lnSa($T*$)

$$\mu_{\ln} sa(T_i)/\ln sa(T*) = mLnsa(M,R,T_i) + p(T_i,T*)\varepsilon(T*)\sigma_{Lnsa} \tag{4}$$

$$\sigma_{\varepsilon}(T_i)\varepsilon(T^*) = \sqrt{1 - p^2(T_i, T^*)} \tag{5}$$

Where $\mu_{\ln} sa(M, R, T_i)$ and $\sigma_{\ln sa}(T_i), p(T_i, T^*)$ are obtained using Equation 3, and $M, R$ and $\varepsilon(T^*)$ were identified in Step 1. The exponential of these $\mu_{\ln} sa(T_i) / \ln sa(T^*)$ values gives the CMS and $\sigma_{\ln sa}(T_i) \vert \ln sa(T^*)$ values gives the variance of the CMS.

## 5. GROUND MOTION SELECTION BASED ON THE CONDITIONAL SPECTRA APPROACH

The distribution means and co-variances are set equal to the desired target values (Equation 4 and 5). The only difference between CMS and CS is the use of Monte Carlo simulation to probabilistically generate Conditional spectra from the above mentioned multivariate normal distribution. For each simulated Conditional Spectrum, a ground motion with a similar Conditional spectrum is selected. For this purpose, a large number of ground motions are collected (3500 signals from NGA database). Equation (6) is used for selecting appropriate earthquakes through the calculation of the differences between a ground-motion response spectrum and that of the simulated earthquakes as the result of using Monte Carlo method. The sum of the squared errors (SSE) between $0.2T^*$-$2T^*$ is used expressed as:

$$SSE = \sum_{i=1}^{p} (\ln sa(T_j) - \ln sa(T_j^{(s)}))^2 \tag{6}$$

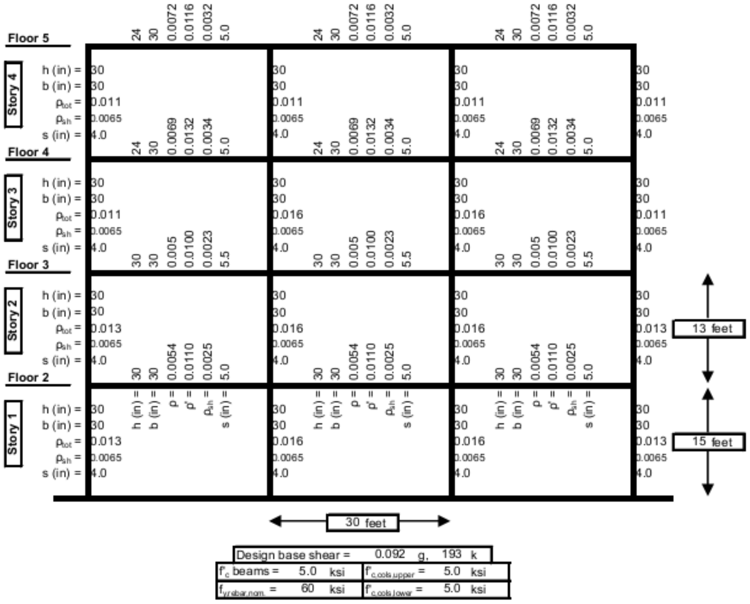
where $\ln sa(T_j)$ is the logarithmic spectral acceleration of the (optionally scaled) ground motion in at period $T_j$, $\ln sa(T_j^{(s)})$ is the target lnSa at period $T_j$ from the simulated Conditional spectrum, p is the number of periods considered and SSE is the sum of squared errors a measure of dissimilarity. Notably, the measure of similarity defined by Equation 1 is not unique and discussion of the other measures of similarity can be found [16,17]. The selection is done by computing SSE for each ground motion in the database thereafter the ground motion having the smallest SSE is chosen.

## 6. STRUCTURES CONSIDERED

The comparison procedure is done through performing IDA on just those structures studied as part of a larger research effort [18, 19] aimed at better evaluating the robustness of CS method. Table 1.1 and figure 3 summarizes the structural models used in this paper. These specific four buildings were chosen to study the variation effects of height (3–9 stories), type of the structures (Steel moment frame– RC Special Moment Frame), and the first-mode period (0.86–2.23 sec). Furthermore, the OpenSees (2006) analysis platform [20] is used for this study accounting for P-Delta effects under a combination of the gravity loads on the lateral resisting frame and on the leaning columns.

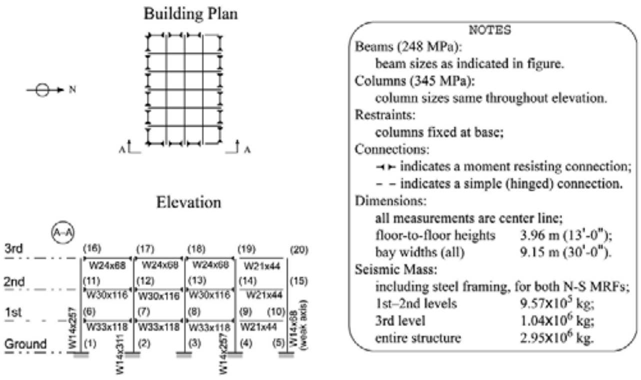
Table 1: structural model characteristics

| Building | stories | Type | $T_1$(s) |
|---|---|---|---|
| A [19] | 4 | RC Special Moment Frame | ٠.٨٦ |
| B [18] | 3 | Steel moment-resisting frame | ١ |
| C [19] | 8 | RC Special Moment Frame | ١.٧ |
| D [18] | ٩ | Steel moment-resisting frame | ٢.٢٣ |



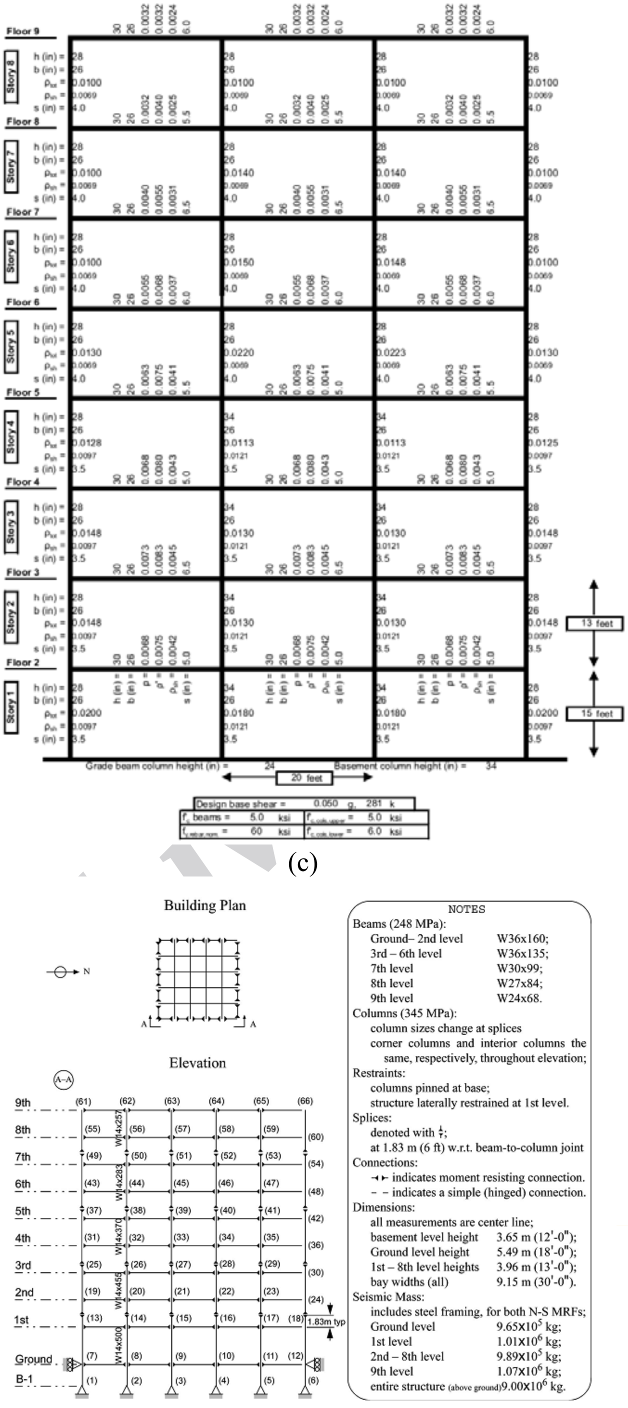
| Design base shear = | 0.092 g, | 193 k |
|---|---|---|
| f'c beams = 5.0 ksi | f'c,cols,upper = | 5.0 ksi |
| fy,rebar,nom = 60 ksi | f'c,cols,lower = | 5.0 ksi |

(a)



NOTES
Beams (248 MPa):
beam sizes as indicated in figure.
Columns (345 MPa):
column sizes same throughout elevation.
Restraints:
columns fixed at base;
Connections:
→← indicates a moment resisting connection;
– – indicates a simple (hinged) connection.
Dimensions:
all measurements are center line;
floor-to-floor heights 3.96 m (13'-0");
bay widths (all) 9.15 m (30'-0").
Seismic Mass:
including steel framing, for both N-S MRFs;
1st–2nd levels $9.57\times10^5$ kg;
3rd level $1.04\times10^6$ kg;
entire structure $2.95\times10^6$ kg.

(b)



| | | | | | |
|---|---|---|---|---|---|
| Design base shear = | 0.050 | g, | 281 | k | |
| $f'_c$ beams = | 5.0 | ksi | $f'_{c,cols,upper}$ = | 5.0 | ksi |
| $f_{y,rebar,nom}$ = | 60 | ksi | $f'_{c,cols,lower}$ = | 6.0 | ksi |

(c)

NOTES

Beams (248 MPa):

| | |
|---|---|
| Ground– 2nd level | W36x160; |
| 3rd – 6th level | W36x135; |
| 7th level | W30x99; |
| 8th level | W27x84; |
| 9th level | W24x68. |

Columns (345 MPa):
column sizes change at splices
corner columns and interior columns the same, respectively, throughout elevation;

Restraints:
columns pinned at base;
structure laterally restrained at 1st level.

Splices:
denoted with ⸸;
at 1.83 m (6 ft) w.r.t. beam-to-column joint

Connections:
→← indicates moment resisting connection.
– – indicates a simple (hinged) connection.

Dimensions:
all measurements are center line;

| | |
|---|---|
| basement level height | 3.65 m (12'-0"); |
| Ground level height | 5.49 m (18'-0"); |
| 1st – 8th level heights | 3.96 m (13'-0"); |
| bay widths (all) | 9.15 m (30'-0"). |

Seismic Mass:
includes steel framing, for both N-S MRFs;

| | |
|---|---|
| Ground level | $9.65\times10^5$ kg; |
| 1st level | $1.01\times10^6$ kg; |
| 2nd – 8th level | $9.89\times10^5$ kg; |
| 9th level | $1.07\times10^6$ kg; |
| entire structure (above ground) | $9.00\times10^6$ kg. |

(d)

Figure 3. Structural modeling documentation (a) structure A (b) structure B (c) structure C (d) structure D

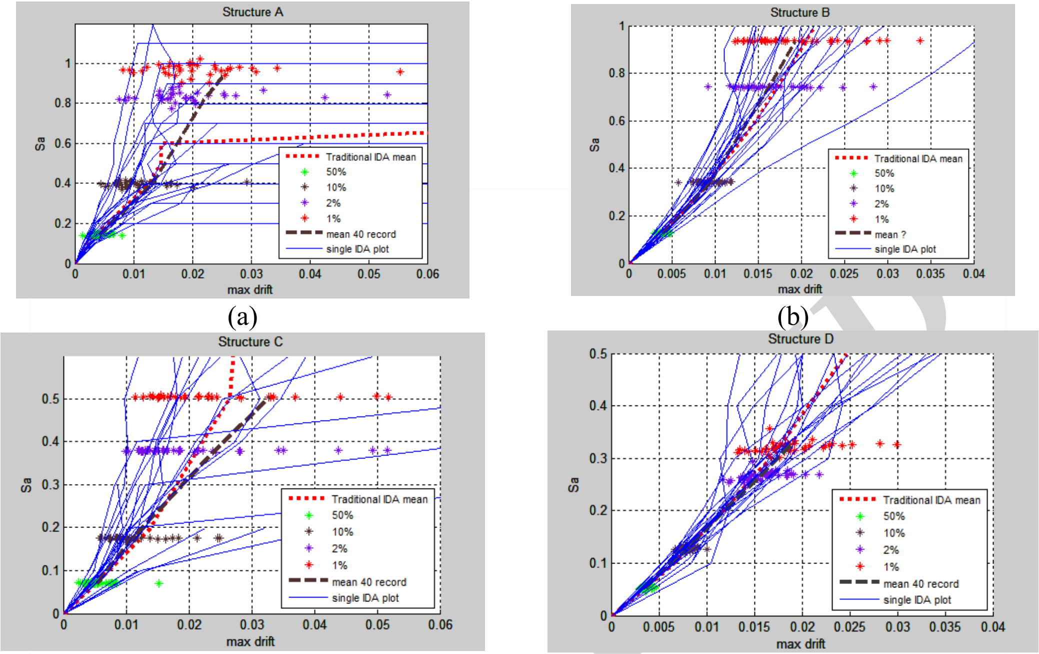
## 7. RESULTS, COMPARISON, AND DISCUSSION

In this study, we first performed the traditional IDA algorithm using 20 single arbitrary accelerations record [3, 4]. The analysis was run and the Damage measures (DMs) in the form of maximum inter-story drift was extracted against all Sa (T1) (IMs). The IMs were calculated for four different hazard levels i.e., the probability of exceedances (PEs); 50%, 10%, 2% and 1%. The result of this step is the plots of 20 single IDA curve for each structure. The procedure was followed for the four selected structures (see Figure 4).The mean curves of these plots were calculated for the purpose of comparing the results with those of CSs. Next step was following the calculation process using CS approach.40 records compatible with the site and the first periods of the selected structures were selected using CS method. The selected earthquakes were applied to the selected four structures and the IMs corresponding to the four PEs (50%, 10%, 2%m and 1%) were calculated. For example, when computing the Maximum Inter-story corresponding to 2%, the 40 earthquakes obtained from Sa (T1) were used which had been chosen by means of the correlation coefficient and Sa (T1) corresponding to 2% hazard level and so on. The Sa (T1) for the selected structures at different hazard levels including 50%, 10%, 2%, and 1% are summarized in Table 2.

Table 2: Target spectral acceleration, Sa($T_1$), at different hazard levels

| structure | 50% | 10% | 2% | 1% |
|---|---|---|---|---|
| A | 0.15 | 0.40 | 0.83 | 0.97 |
| B | 0.13 | 0.34 | 0.74 | 0.94 |
| C | 0.07 | 0.18 | 0.38 | 0.51 |
| D | 0.05 | 0.13 | 0.27 | 0.32 |

Figures 4a-4d demonstrate the comparisons of results obtained from the traditional individuals IDA curves associated with the mean cures and those of Baker at four hazard levels;50%, 10%, 2%, and 1% [7]. Let us put a comment on the results of dynamic nonlinear analysis of the selected four structures; two steel moment-resisting frames and two RC Special Moment Frames. In the B and D labeled structures the IMs have grown quite higher than those corresponding to PEs of 1% which is due to the high strength of these two structures. This result means that, the traditional method strongly suffers from detecting the possible upper hazard levels e.g. 2% or 1%. The scaling factor in CS method is restrained to 4 due to the fact that since scaling process doesn't consider the variation of frequency content of earthquake, it should be limited to an predefined value otherwise might end up with mathematic signal rather than physical ground motion. In traditional IDA the scaling factors are quite high in comparison to that of CS which is limited to 4. Table 3 shows the maximum scale factor of records in each hazard level of IDA process. We recommend that, more strong earthquakes be used in traditional IDA since higher hazard levels are supposed to be experienced. Conversely, frames A and C ended up with different results, in that firstly the inter-story obtained from the traditional method is considerably smaller than those of CS method. Secondly, the frame labeled A came up with collapse problem while that of CS shows the frame to be still stable. We believe that, the procedure still needs much work to better understand the problems associated with collapse conditions.



(a) (b)

(c) (d)

Figure 4. Comparison of the traditional IDA curves (20 ground motions), corresponding mean curves with those of CS method (40 selected records) for structures: (a) structure A, (b) structure B, (c)structure C, and (d) structure D

Table 3: maximum scale factor of records in IDA process

| | structure A | Structure B | structure C | structure D |
|---|---|---|---|---|
| 1% | 31 | 44 | 21 | 19 |
| 2% | 27 | 35 | 16 | 17 |
| 10% | 13 | 16 | 8 | 8 |
| 50% | 5 | 6 | 3 | 3 |

## 8. CONCLUSIONS

In this study, the nonlinear response of the selected four frames: two steel moment-resisting frames and two RC Special Moment Frames were calculated using the traditional and CS methods aimed at better understanding seismic strength capacity of structures.

The following conclusions are conducted with respect to the limited number of dynamic analysis performed;

- The traditional method suffers from detecting the state limits of structures corresponding to the hazard levels e.g. 2% or 1% while CS does.

- The scaling factors of the traditional method may grow up to values more than 10 while that of CS is limited to 4.

- Surprisingly, the deformation stability of the two methods may be quite different which physically this result doesn't make sense. In other word the question is; weather such diferent results is as a result of mathematic or physic problems. The traditional approach may end up with unstable process of frame deformation state while that of CS shows the frame to be stable. For this reason, the authors recommend more research work is required to be done in this area.

## REFERENCES

1. Baker JW. The conditional mean spectrum: A tool for ground motion selection, *Journal of Structural Engineering, ASCE*, **137**(2009) 322–31.
2. American Society of Civil Engineers. *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-05, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA, 2005.
3. Vamvatsikos D, Cornell CA. Incremental dynamic analysis, *Earthquake Engineering & Structural Dynamics*, **31**(2002), 491–514.
4. Vamvatsikos D, Cornell CA. Applied incremental dynamic analysis, *Earthquake Engineering & Structural Dynamics*, **20**(2004): 523–53.
5. Shome N, Cornell CA, Bazzurro P, Carballo JE. Earthquakes, records, and nonlinear responses, *Earthquake Spectra*, **14**(1998) 469–500.
6. Baker JW, Cornell CA. Spectral shape, epsilon and record selection, *Earthquake Engineering and Structural Dynamics*, **35**(2006) 1077–95.
7. Jayaram N, Lin T, Baker JW. A computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance, *Earthquake Spectra*, No. 3, **27**(2011) 797-815.
8. Abrahamson NA, Al Atik L. Scenario spectra for design ground motions and risk calculation, *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada, 2010.
9. Baker JW, Cornell CA. *Vector-Valued Ground Motion Intensity Measures for Probabilistic Seismic Demand Analysis*, Blume Center Technical Report #150, Stanford University, Stanford, CA, 2005.
10. Luco N, Bazzurro P. Does amplitude scaling of ground motion records result in biased nonlinear structural drift responses, *Earthquake Engineering and Structural Dynamics*, **36**(2007) 1813-35.
11. Boore DM, WB Joyner, TE Fumal. Equations for estimating horizontal response spectra and peak accelerations from we stern North America earthquakes: A summary of recent work, *Seismological Research Letters*, **68**(1997) 128-53.
12. Haselton CB, Deierlein GG. *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings*, Technical report, Report 2007-08, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, 2007.
13. U.S. Geological Survey (USGS), 2008. 2008 Interactive Deaggregations (Beta)

http:// eqint.cr.usgs.gov/deaggint/2008/.

14. Baker JW, Jayaram N. Correlation of spectral acceleration values from NGA ground motion models, *Earthquake Spectra*, No. 1, **24**(2008) 299–317.
15. Baker JW, Cornell CA. Correlation of response spectral values for multicomponent ground motions, *Bulletin of the Seismological Society of America*, No. 1, **96**(2006) 215–27.
16. Beyer K, Bommer JJ. Selection and scaling of real accelerograms for bi-directional loading: A review of current practice and code provisions, *Journal of Earthquake Engineering,* **11**(2007) 13–45.
17. Buratti N, Stafford PJ, Bommer JJ. Earthquake accelerogram selection and scaling procedures for estimating the distribution of drift response, J*ournal of Structural Engineering, ASCE* **137**(2011) 345–57.
18. *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*. Report No. FEMA-350, SAC Joint Venture, Federal Emergency Management Agency, Washington DC, 2000.
19. Haselton CB. *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Moment Frame Buildings*, Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, 2006.
20. *Open System for Earthquake Engineering Simulation (Opensees)*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, 2007. http://opensees.berkeley.edu/. (Last verified 03=29=2010).