This is a peer-reviewed, accepted author manuscript of the following article:
Fadaei, E., Shakib, H., & Azarbakht, A. (2020). Structural global performance assessment versus individual element-oriented performance-based assessment. Iranian Journal of Science and Technology - Transactions of Civil Engineering. https://doi.org/10.1007/s40996-020-00379-9

Structural Global Performance Design versus Individual Element-Oriented Performance-Based-Design

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Abstract In this paper three new performance indices have been proposed which could be used in order to determine the global performance of a given structure. The ASCE41-13 standard and the FEMA350 guidelines were used as representatives of, respectively, an element-oriented and a system-oriented performance based design algorithm. Two ten-storeyed special steel moment frames, consisting of a regular and an irregular structure, were designed and assessed using these two algorithms. The results show that the element-oriented design algorithm significantly underestimates the seismic demand and capacity, especially in the case of the immediate occupancy and collapse prevention limit states. This underestimation can cause a significant drop in the estimated confidence levels.

Keywords: Performance Based Design; FEMA350; ASCE41-13; Confidence Level.

Introduction

Performance Based Design (PBD) is a major improvement which has become increasingly used in recent decades. It aims to provide stakeholders with an interpretation of structural performance corresponding to a given hazard level (Bozorgnia and Bertero

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2004). Vision2000, which is an attempt by practicing engineers to improve current codes in a significant manner, was a pioneering concept in this field. Its aim was to provide PBD regulations in which performance, hazard levels and acceptance criteria would be rigorously defined.

FEMA273 (FEMA1997) and FEMA356 were developed for the purpose of the seismic rehabilitation of existing buildings. Four limit states were introduced in FEMA356 (FEMA 2000a) which could be evaluated at different hazard levels by employing linear or non-linear procedures. These limit states are: Immediate Occupancy (IO), Operational (O), Life Safety (LS), and Collapse Prevention (CP). FEMA356 later turned into ASCE41-06 (ASCE 2007) and ASCE41-13, which are mandatory regulations in the US. There is a shortcoming, however, which is that these regulations do not explicitly take into account either different uncertainties or acceptance criteria which are based on system-oriented behaviour. In other words, exceeding a limit state in just one structural element is interpreted as meaning that the whole structure has exceeded the prescribed global performance level.

The various structural limit states have mainly the same definitions in different regulations. For example, in ASCE41-13, IO corresponds to very minor damage in which the structural system keeps its stiffness and strength without any residual drift. At the LS level, moderate damage occurs, but stiffness and strength change slightly. A residual drift will occur and the structure needs to be repaired. At the CP level, the stiffness and strength change significantly, although the columns and walls of the building are still able to carry their gravity loads. Significant residual drift occurs, and the structure will be near collapse. No further occupancy of the structure is possible.

Incremental Dynamic Analysis (IDA) is a method involving comprehensive nonlinear response history analysis, which has been widely used in the last decade

(Vamvatsikos and Cornell 2002). It can be used to define a given structural behaviour from elastic to extensively inelastic behaviour, by implementing an Engineering Demand Parameter (EDP) at different Intensity Measure (IM) values. The aleatory uncertainty is explicitly taken into account in the IDA procedure by including a relatively large set of ground motion records. Despite the good progress which has already been made, the IDA calculation method (Vamvatsikos and Cornell 2004 and 2005, Han and Chopra 2006, Vamvatsikos and Fragiadakis 2010, Dolsek 2009, Asgarian et al 2010, Azarbakht and Dolšek 2010), the definitions of different limit states on an individual IDA curve still remain a practical challenge. This issue has different aspects in the case of different seismic lateral bearing systems. For example, the FEMA350 criteria are based on experimental data for steel moment frames i.e. Liu and Astaneh-Asl (2000), Lee and Foutch (2000), Venti and Engelhardt (2000), and Gilton et al (2000). The IO limit state which corresponds to the Maximum Inter-storey Drift Ratio (MIDR) reaches up to 2 %. The CP limit state is reached if the MIDR reaches up to 10 %, or if the IDA curve slope drops to a value which is less than 20 % of the initial (elastic) stiffness. In fact, structural global behaviour is used to define the limit states in the FEMA350 guideline (FEMA2000b). On the other hand, on the basis of the ASCE41-13 standard (ASCE/SEI 2014), a set of appropriate plastic hinges needs to be assigned to each element. The maximum deformation during the response history analysis is then compared to the acceptable thresholds (provided in the standard). Consequently, when the first element demand is exceeded, this is taken as the limit state of the structure. In other words, the ASCE41-13 standard implements an individual-element-oriented criterion in order to define this limit state.

When an element goes beyond the performance threshold, this does not necessarily mean that the whole structural limit state has been reached. This important issue is

addressed in the US Army Corp of Engineers' Manual: Earthquake design and evaluation of concrete hydraulic structures (USACE 2007), as well as in the FEMAP695 guideline (FEMA2009). This issue is the main focus of the current study, where it has been assumed that the seismic design (or assessment) of a specific structure based on the two regulations should produce nearly the same result. For this reason, two ten-storeyed special steel moment frame structures were assessed by means of FEMA350, as being representative of the global behaviour point of view, as well as by ASCE41-13, as being representative of the element-oriented behaviour point of view. The number of elements, the cumulative time intervals of demand excess in hinges, the ratio of maximum hinge rotation to capacity rotation, and the global confidence levels are calculated and discussed in detail in order to propose a new view-point in this area of research.

Methodology

In order to assess the ASCE41-13 definitions for limit states, two ten-storeyed special steel moment frame structures were designed based on the ASCE7-10 (ASCE 2010) and AISC (2010) regulations. The first structure was regular in all its aspects, whereas the second structure was irregular in height. IDA was performed for the given structures for a set of appropriate ground motion records. The EDP and IM corresponding to the IO and CP limit states were defined on each IDA curve, based on the FEMA350 guideline. Three new indices were then proposed including: a Cumulative Time Demand Excess Ratio (CTDER), a Maximum Hinge Demand Excess Ratio (MHDER), and a Ratio of Demand Excess Elements (RDEE). The CTDER index is expressed mathematically in Equation (1). This index is calculated for each hinge of a given element and for a given ground motion. The total damage in each element will clearly increase as the CTDER index increases.

$$CTDER_{i,k}^{j} = \left(\frac{t_{ce,i}}{t_{d,k}} \right)_j \quad (1)$$

where i is the element number, j is the number of the nonlinear behaviour, k is the record number, $t_{ce,i}$ is the cumulative time for which the demand exceeds the acceptance level (based on ASCE41-13), and $t_{d,k}$ is the total ground motion significant duration (Trifunac and Brady 1975).

The MHDER index is defined in Equation (2). It shows the magnitude of the demand excess in each hinge under the effect of a certain ground motion record. This index, too, clearly has a positive correlation with the total damage in each element.

$$MHDER_{i,k}^{j} = \left(\frac{\theta_{\max,i,k}}{\theta_{\lim}} \right)_j \quad (2)$$

where $\theta_{max,i,k}$ is the maximum hinge rotation in the j^{th} nonlinear behaviour in the i^{th} element in the case of the k^{th} ground motion record. θ_{lim} is the acceptable hinge rotation based on the ASCE41-13 standard.

The RDEE index, defined in Equation (3), takes into account the normalized number of elements that are beyond the acceptance criteria based on the ASCE41-13 standard.

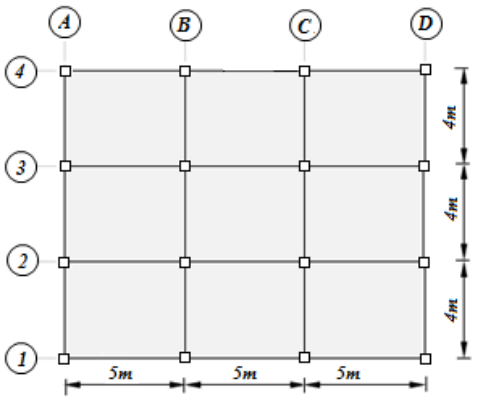
$$RDEE_{k}^{j} = \frac{N_{ee,k}^{j}}{N_{el}} \quad (3)$$

It should be mentioned that in this study these indices were calculated for the columns and beams separately. The idea was that various combinations of these three proposed indices would shed light on the detailed behaviour of the structural system as a whole. This should make possible decisions about whether or not the design of a given structure as a whole is acceptable. This issue is discussed in detail in the following sections.

Structural Models and Analyses

The two investigated ten-storeyed special steel moment frames were designed based on the provisions of AISC 2010, and are shown in Figure 1. The first structure has no irregularities, whereas the second structure has a severe lateral stiffness irregularity in height. Both structures were designed for a highly seismic region, i.e. the Tehran metropolis, and soil type C (based on the ASCE7-10 standard). The dead and live loads on the typical floors are, respectively, equal to 600 kg/m2 and 200 kg/m2. Rectangular box shaped and IPE profiles were used for the column and beam sections, respectively. The effective mass and lateral strength were identical for both structures, and their fundamental period was equal to 1.37s.

(a)



(b)

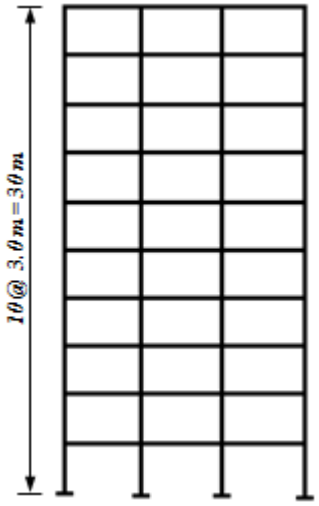


Figure 1. Properties of the designed structures in this study, (a) plan view and (b) elevation view.

The OpenSees platform was utilized in order to perform the nonlinear analyses (*Opensees*). In order to simplify the analysis procedure, a 2D model was created. Elastic beam-column elements with two nonlinear zero-length elements, at each of the two ends, were used to model the beams and columns. The nonlinear backbone curve is defined based on the ASCE41-13 standard (ASCE/SEI 2014), and is shown schematically in Figure 2. The strain hardening part, marked as the line B-C in Figure 2, has a slope which is equal to 3% of the initial (elastic) slope. The Ibarra and Krawinkler model (Ibarra et al. 2005) was implemented within the OpenSees platform; it consists of five modelling parameters, as can be seen in Figure 3. The five modelling parameters include: (1) the pre-capping plastic rotation, θ_p (2) the post-capping (i.e. from maximum moment to fracture) plastic rotation, θ_{pc} (3) the cumulative rotation capacity that determines the reference energy dissipation capacity of a structural component, Λ (4), the effective-to-predicted component yield strength, M_y /M_{yp} and (5) the capping-strength-to-effective-yield-strength ratio, M_c /M_y.

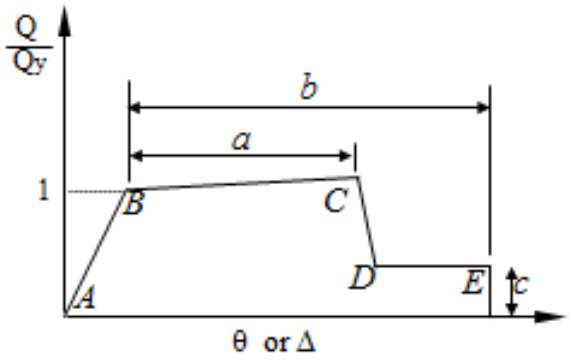


Figure 2. The schematic nonlinear backbone for beam and column members (ASCE/SEI 2014).

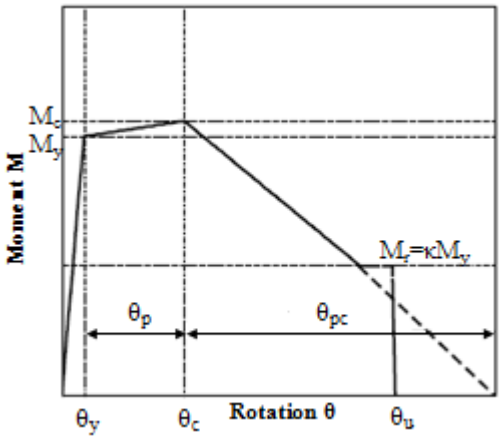


Figure 3. The Ibarra-Krawinkler backbone deterioration model (Ibarra et al. 2005).

Rayleigh damping proportional to mass and stiffness (Chopra 1995) was assumed by considering 5% damping for the first and third modes. The floors were assumed to be rigid, and the P-Delta effect was taken into consideration (Mazzoni et al. 2007).

The Hunt and Fill algorithm (Vamvatsikos and Cornell 2004) is utilized within the IDA algorithm in order to quantify the structural limit states in terms of IM and EDP. The spectral acceleration at the fundamental period of structure and five percent damping, Sa(T1, 5%) was taken as the IM measure, and the MIDR was assumed as the EDP. It is worth noting that MIDR has a positive correlation with the structure's global instability, as well as with the limit states (FEMA2000b). A total of 25 far-field ground motion records were used as input for the IDA (ATC-63 2008). The moment magnitude of the given

records was between 6.5 and 7.6, and no directivity effect can be observed among them. The characteristics of the records are summarized in Table 1.

Table 1. Characteristics of the 25 ground motion records for the IDA analysis (ATC-63 2008).

No.	Event	Station	Magnitude	Distance (km)	PGA (g)	Time duration(sec)
1	Kobe, Japan, 1995	Nishi-Akashi, 090	6.9	25.2	0.5	11.25
2	Kobe, Japan, 1995	Shin-Osaka, 000	6.9	28.5	0.24	10.35
3	Kocaeli, Turkey, 1999	Arcelik, 000	7.5	13.5	0.22	11.06
4	San Fernando, 1971	LA-Hollywood Stor FF, 090	6.6	25.9	0.21	13.17
5	Landers, 1992	Coolwater, LN	7.3	20	0.28	10.58
6	Landers, 1992	Coolwater, TR	7.3	20	0.42	8.24
7	Superstition Hills, 1987	Poe Road (temp), 270	6.5	11.7	0.45	13.71
8	Superstition Hills, 1987	Poe Road (temp), 360	6.5	11.7	0.3	13.66
9	Hector Mine, 1999	Hector, 000	7.1	12	0.27	11.68
10	Manjil, Iran, 1990	Abbar, T	7.4	13	0.5	29.12
11	Chi-Chi, Taiwan, 1999	TCU045, E	7.6	26.8	0.47	11.34
12	Chi-Chi, Taiwan, 1999	TCU045, N	7.6	26.8	0.51	10.82
13	Friuli, Italy, 1976	Tolmezzo, 000	6.5	15.8	0.35	4.25
14	Landers, 1992	Yermo Fire Station, 270	7.3	23.8	0.24	17.6
15	Landers, 1992	Yermo Fire Station, 360	7.3	23.8	0.15	18.88
16	Loma Prieta, 1989	Gilroy Array #3, 000	6.9	12.8	0.56	6.37
17	Northridge, 1994	W Lost Cany, 000	6.7	12.4	0.41	6.29
18	Northridge, 1994	W Lost Cany, 270	6.7	12.4	0.48	5.58
19	San Fernando, 1971	LA—Hollywood Stor FF, 180	6.6	25.9	0.17	13.43
20	Kocaeli, Turkey, 1999	Duzce, 180	7.5	15.4	0.31	11.80
21	Kocaeli, Turkey, 1999	Duzce, 270	7.5	15.4	0.36	10.86
22	Loma Prieta, 1989	Capitola, 000	6.9	35.5	0.53	12.15
23	Loma Prieta, 1989	Capitola, 090	6.9	35.5	0.44	13.16
24	Imperial Valley, 1979	Delta, 262	6.5	22.5	0.24	51.43
25	Imperial Valley, 1979	Delta, 352	6.5	22.5	0.35	50.52

Limit states Definitions

In this section, the seismic performance is assessed by three algorithms, as follows:

(1) The IO and CP limit states are controlled by implementing the FEMA350 and ASCE41-13 algorithms, respectively, as being representative of system-oriented and element-oriented behaviours.

(2) CTDER, MHDER and RDEE indices are employed in order to combine the element-oriented and the system-oriented point of views. This should help to define the structure's behaviour as a whole.

(3) The confidence levels, at the IO and CP limit states, are calculated based on the element-oriented and system-oriented point of views.

Element-oriented and system-oriented limit states

As all the parameters were identical for the structural analysis, it was anticipated that the limit states would be relatively close, based on the FEMA350 and ASCE41-13 algorithms. The IO and CP limit states on the median IDA curve are shown in Figure 4 in the case of the FEMA350 and ASCE41-13 algorithms. As can be clearly seen from this figure, the two above-mentioned algorithms do not result in nearly the same points for the IO and CP limit states. To further investigate this issue, the differences between the IMs, in the cases of the IO and CP limit states, respectively, versus the ground motion records are shown in Figures 5 and 6, from which it can be seen that these differences are more significant in the case of the IO than in the case of the CP limit state. Although this study was not focussed on irregularity effects, it can also be clearly seen from Figures 5 and 6 that the differences are more significant in the case of the irregular structure when compared with the regular structure. On average, the difference between the IMs corresponding to the element-oriented and system-oriented algorithms, were, respectively, 69 % and 42 %, in the case of the IO and CP limit states. These results primarily show that the element-oriented design algorithm (ASCE41-13) results in a significantly more conservative output when compared to the system-oriented design algorithm (FEMA350). This issue is further elaborated in the following sections.

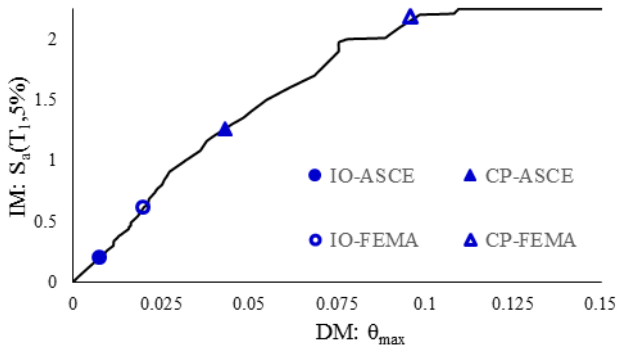


Figure 4. The IO and LS limit states on the median IDA curve of the regular structure obtained on the basis of the FEMA350 and ASCE41-13 regulations.

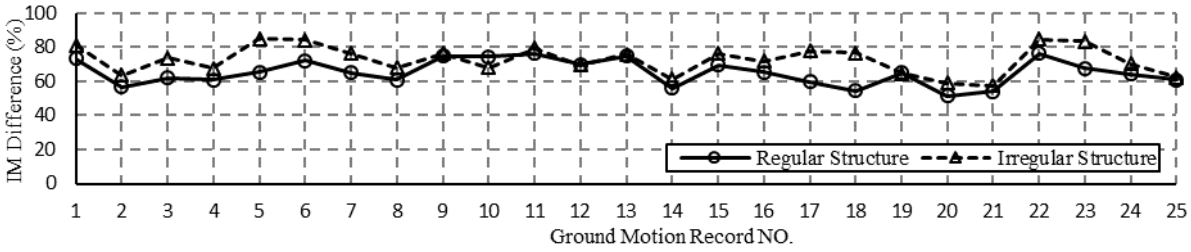


Figure 5. The difference between the IMs corresponding to the IO limit state versus the ground motion records based on the FEMA350 and ASCE41-13 algorithms.

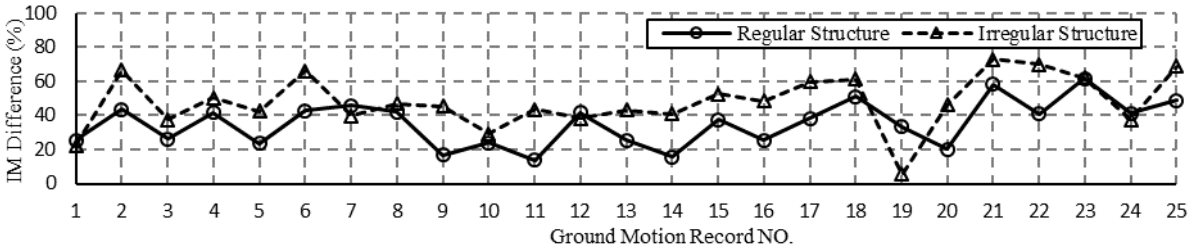


Figure 6. The difference between the IMs corresponding to the CP limit state versus the ground motion records based on the FEMA350 and ASCE41-13 algorithms.

Limit states based on the new proposed indices

In the element-oriented design algorithm, any hinge demand excess corresponds to a structural limit state. However, no information about the size of this demand excess is available. In other words, the cumulative amount of demand excess in individual hinges has to reach to a certain level in order to be able to degrade the total stiffness and strength of the structure sufficiently. The IMs corresponding to the IO and CP limit states, based on the FEMA350 guidelines, are taken here as benchmarks since the FEMA350 recommendations are based on extensive experimental data, e.g. see Liu and Astaneh-Asl (2000), Lee and Foutch (2000), Venti and Engelhardt (2000) and Gilton et al (2000). The CTDER, MHDER and RDEE indices are then calculated independently for the beams and columns. For example, the CTDER index shown in Figure 7 corresponds to an IO limit state based on the ASCE41-13 standard in the case of the regular structure under the effect

of record No. 1. As can be seen in Figure 7, the majority of the beam elements are beyond the IO limit state, for at least more than 20 % of the ground motion significant duration. The CTDER index is averaged over the whole beam hinges, which presents a 37.5 % data point in Figure 8. In fact, Figure 8 shows the average CTDER index versus the different ground motion records. As can be seen from this figure, most of the beams show nonlinear behaviour beyond the limit state, although this varies between 8 % and 66 %, depending on the input ground motion. Figure 9 shows the average MHDER index, only over the demand excess elements. This index is up to three and four, respectively, in the cases of both the regular and the irregular structure. By averaging over the records, the CTDER index is 37.5 % and the MHDER index is 2.87 %.

The RDEE index is shown in Figure 10, which confirms that almost 90 % of the beams have gone beyond their limit states. On the other hand, no column had any demand excess at the IO limit state. A summary of the behaviour of the two investigated structures, referring to the IO limit state, is provided in Table 2.

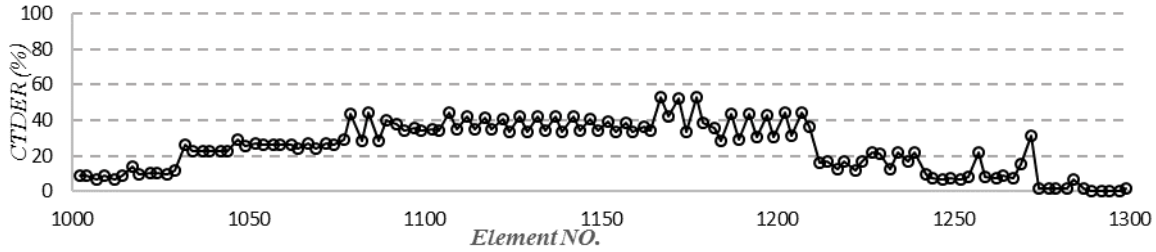


Figure 7. The CTDER index versus different beam elements at an IM corresponding to the IO limit state based on ASCE41-13, and the regular structure, under record No.1.

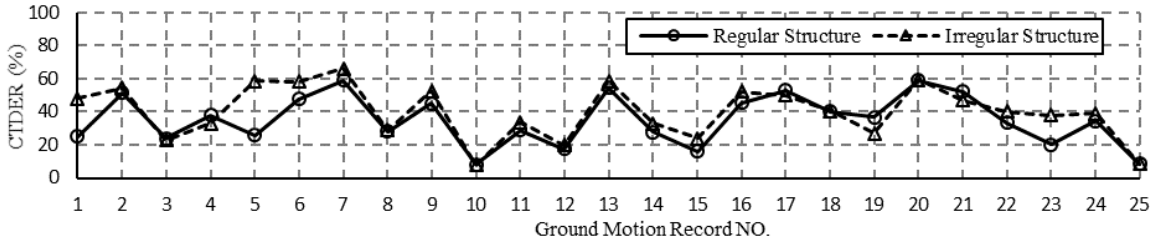


Figure 8. The beams' average CTDER index versus the different ground motion records in the case of the IO limit state based on ASCE41-13.

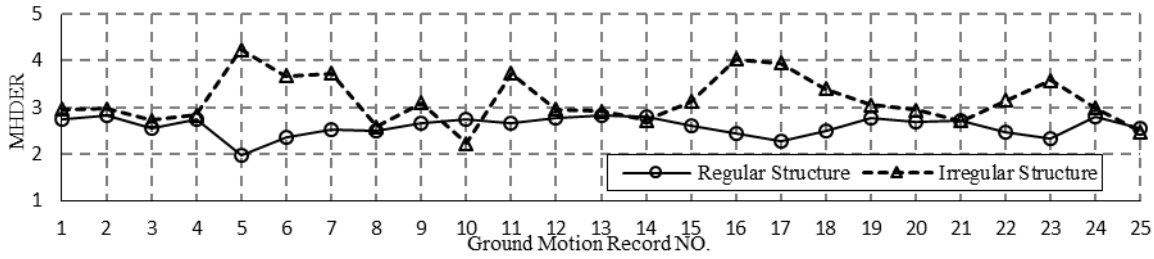


Figure 9. The beams' average MHDER index versus the different ground motion records in the case of the IO limit state based on ASCE41-13.

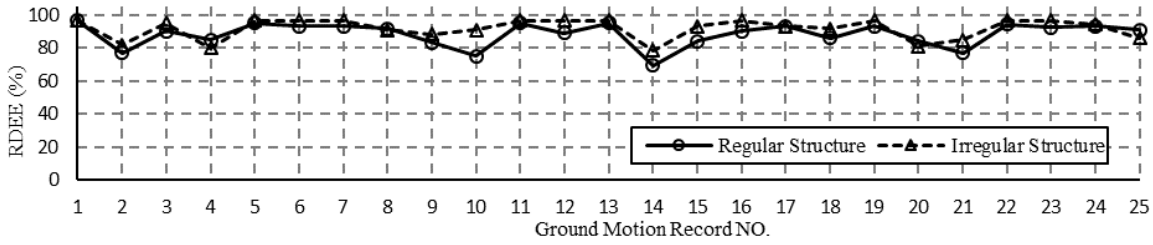


Figure 10. The beams' RDEE index versus the different ground motion records in the case of the IO limit state based on ASCE41-13.

Table 2. Summary of the proposed indices for the given structures at the IO limit state.

Structural Members	Beams		Columns	
Structure type	Regular	Irregular	Regular	Irregular
CTDER	35%	40%	0	0
RDEE	88%	92%	0	0
MHDER	2.6	3.15	0	0

The average values of the CTDER and MHDER indices, and of the RDEE indices for the beams, are presented, for the CP limit state, in Figures 11, 12 and 13, respectively. By averaging over the records, the CTDER index is 11 %, and the MHDER index is 1.62 %. 57 % of the beams experience a certain level of demand excess.

In the case of the IM corresponding to the CP limit state (based on FEMA350), only the demand at the bottom of the first storey columns exceeds the limit state (based on ASCE41-13). This behaviour was anticipated, since columns are usually designed more

conservatively than beams, especially in special moment frames. The average values of the CTDER and MHDER indices are shown, respectively, in Figures 14 and 15, in the case of the first storey columns. By averaging over the records, the CTDER index is 13.5 %, and the MHDER index is 1.66 %. A summary referring to the CP limit state is provided in Table 3.

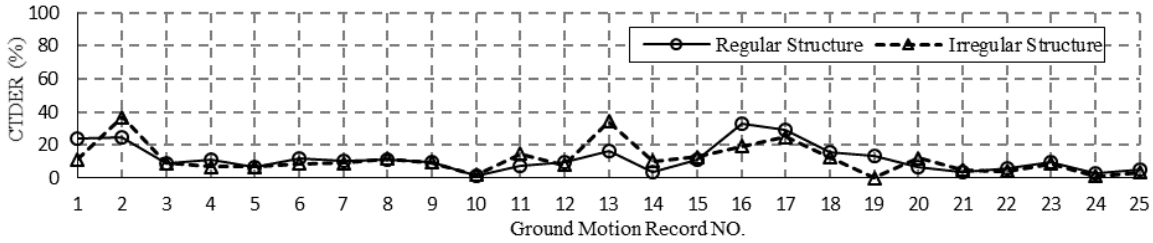


Figure 11. The average CTDER index for the beams versus the different ground motion records, in the case of the CP limit state based on ASCE41-13.

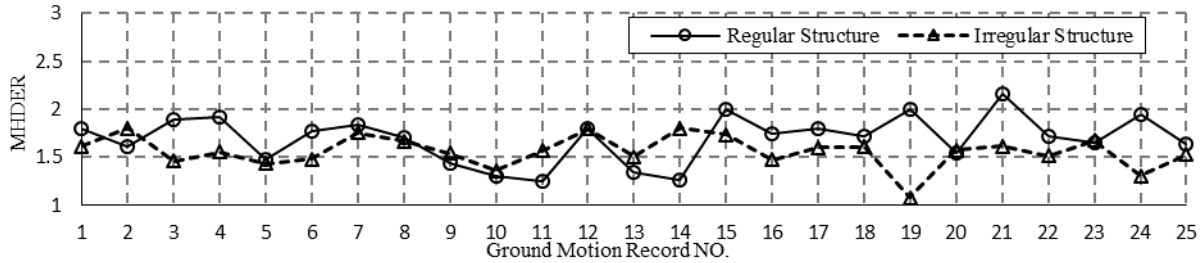


Figure 12. The average MHDER index for the beams, versus the different ground motion records, in the case of the CP limit state based on ASCE41-13.

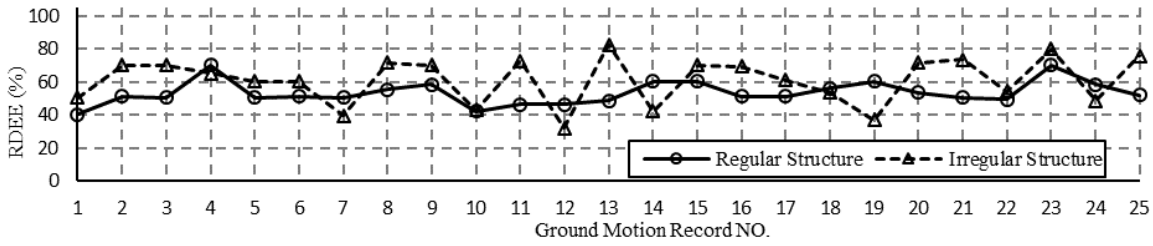


Figure 13. The RDEE index for the beams, versus the different ground motion records, in the case of the CP limit state based on ASCE41-13.

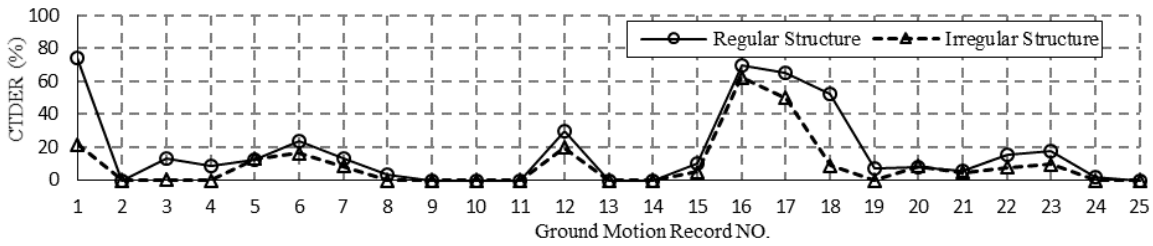


Figure 14. The average CTDER index for the first storey columns, versus the different ground motion records, in the case of the CP limit state.

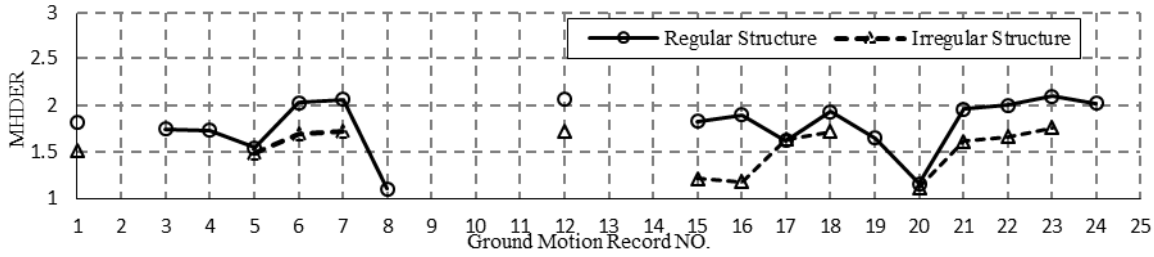


Figure 15. The average MHDER index for the first storey columns, versus the different ground motion records, in the case of the CP limit state.

Table 3. Summary of the proposed indices for the given structures at the CP limit state.

Structural Members	Beams		Base of Columns	
Structure type	Regular	Irregular	Regular	Irregular
CTDER	11%	11%	17%	10%
RDEE	53%	61%	72%	64%
MHDER	1.68	1.56	1.78	1.53

Confidence levels

The Cornell method (Cornell et al. 2002) is implemented in this section in order to calculate the confidence level in each case by taking into account the demand and capacity uncertainties. The confidence level is a statistical interpretation of the ratio of the factored demand to the factored capacity. One set of factors increases the demand whereas another set of factors decreases the capacity in order to account for different uncertainties as well

as different regional seismicity characteristics. The seismic demand is calculated by taking into account the Tehran metropolis, with k=4.07 and b=1.0, where k is the logarithmic slope of the hazard curve, and b is a coefficient which relates the incremental change in demand to an incremental change in ground shaking intensity, at the hazard level of interest, typically taken as having a value of 1.0 (Cornell et al. 2002). On the other hand, the capacity is obtained based on (1) FEMA350 by applying the previously mentioned rules on the IDA curves, and (2) ASCE41-13 by monitoring the first hinge which shows a demand excess beyond the limit state. A summary of confidence levels for the different structures and different algorithms is given in Table 4, from which it can be seen that all the confidence levels are above 96 % except in the case of the IO limit state, when using the ASCE41-13 regulations. Recalling the Cornell method, the parameter λ is equal to the ratio of the factored demand over the factored capacity. Additionally, based on the standard Gaussian distribution, λ=0.86 is identical to a 90 % confidence level, and higher values of the confidence level will be achieved by lower λ values. In other words, as can be seen in Table 4, all the λ values are below 0.86 except in the case of the IO limit state when using the ASCE41-13 regulations, in which the parameter λ is equal to 1.293 in the case of both the regular and the irregular structure. This value of λ results in a 7.9 % confidence level, which is not acceptable for design purposes. As these structures were designed based on up-to-date design standards, this is proof that the ASCE41-13 standard significantly underestimates the capacity corresponding to the IO limit state. Although the capacity estimates are quite different in the case of the CP limit state, nevertheless the confidence levels are above 96 % in the case of both the FEMA350 and the ASCE41-13 standards.

Table 4. Confidence levels for the IO and CP limit states, based on the FEMA350 and the ASCE41-13 regulations.

Performance Objective	Structure	Capacity Calculation Algorithm	*Sa(T1,5%)*	*D*	*γ*	γ_a	*C*	□	*λ*	K_x	*CL (%)*

IO limit state against the 50/50 hazard level	Regular	FEMA350	0.2g	0.0076	1.23	1.035	0.0200	1	0.484	5.15	> 99.7
		ASCE41-13	0.2g	0.0076	1.23	1.035	0.0076	0.9878	1.293	-1.41	7.93
	Irregular	FEMA350	0.2g	0.0063	1.05	1.035	0.0200	1	0.342	7.46	> 99.7
		ASCE41-13	0.2g	0.0063	1.05	1.035	0.0057	0.9329	1.293	-1.41	7.95
CP limit state against the 2/50 hazard level	Regular	FEMA350	0.47g	0.0165	1.17	1.085	0.0895	0.8711	0.268	4.10	> 99.7
		ASCE41-13	0.47g	0.0165	1.17	1.085	0.0432	0.9209	0.525	2.42	99.23
	Irregular	FEMA350	0.47g	0.0141	1.12	1.085	0.0841	0.8222	0.248	4.30	> 99.7
		ASCE41-13	0.47g	0.0141	1.12	1.085	0.0303	0.8288	0.683	1.77	96.15

Note: D=median structural demand; C=median structural capacity; γ=demand variability factor; γ_a= analysis uncertainty factor; □= resistance factor; λ=confidence parameter; Kx= standard Gaussian variate associated with the probability x of not being exceeded; CL=confidence level.

Conclusions

In this study, performance based design was investigated from two different points of views, as follows: (1) the FEMA350 guidelines, being a representative of system-oriented behaviour, and (2) the ASCE41-13 standard, being a representative of element-oriented behaviour. Two regular and irregular structures were assessed by these regulations, and the Immediate Occupancy and Collapse Prevention limit states were defined. Additionally, three new indices are proposed in order to justify the differences between the results obtained by using these two different algorithms. The results show that the element-oriented algorithm significantly underestimates the seismic capacity in the case of both the IO and the CP limit states. However, this underestimation results in a reduction of the confidence level, especially in the case of the IO limit state.

At the IO limit state, and based on the FEMA350 regulations, over 90 % of the beams experience a demand excess beyond the acceptable level. These beams stay, on average, above the acceptable threshold for 37.5 % of the motion duration. On average, the plastic rotation in these beams goes up to 280 % beyond the acceptable threshold. On the other hand, no columns experience any demand excess at this limit state.

At the CP limit state, and based on the FEMA350 regulations, over 57 % of the beams experience a demand excess beyond the acceptable level. These beams stay, on

average, above the acceptable threshold for 11 % of the motion duration. On average, the plastic rotation in these beams goes up to 62 % beyond the acceptable threshold. On the other hand, at their bases all the first storey columns experience a demand excess in the case of the majority of the used ground motion records. The demand excess in the first storey columns, on average in 13 % of the motion duration, reaches up to 66 % beyond the acceptable level.

In summary, the element-oriented algorithm significantly underestimates the seismic demand when compared to the system-oriented algorithm. It should be mentioned that the results presented in this paper are limited by the assumptions made, and that further investigations are necessary to shed more light on this important line of research.

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