Earthquakes and Structures, *Vol. 10, No. 4 (2016) 849-865*
DOI: http://dx.doi.org/10.12989/eas.2016.10.4.849

Upgrading equivalent static method of seismic designs to performance-based procedure

Reza Allahvirdizadeh*[1,2] and Mohammad Ali Mohammadi[1]

[1]*School of Civil Engineering, College of Engineering, University of Tehran, Tehran, Iran*
[2]*Engineering Optimization Research Group (EORG), College of Engineering, University of Tehran, Tehran, Iran*

(Received October 5, 2014, Revised December 2, 2015, Accepted January 14, 2016)

Abstract. Beside the invaluable advancements in constructing more secure buildings, the post-earthquake inspections have reported considerable damages. In other words, the modern buildings satisfactorily decrease fatalities but the monetary impacts still mostly remain an unsolved concern of the stakeholders, the insurance companies and society together. Therefore, the fundamental target of the researches shifted from current force-based seismic design regulations to the Performance-Based earthquake engineering (PBEE). At the moment, some probabilistic approaches, such as PEER framework have been developed to predict the performance of building at any desired hazard levels. These procedures are so time-consuming, to which many details are needed to be assigned. It causes their usage to be limited. On that account, developing more straightforward methods seems indispensable. The main objective of the present paper is to adapt an equivalent static method in different damage states. Consequently, constant damage spectrums corresponding to different limit states, soil types, ductility and fundamental periods are plotted and tri-linear formulas are proposed for further applications. Moreover, the sensitivity of outcomes to the employed hysteresis model, ductility, viscous damping and site soil type is investigated. Finally, a case study building with moment-resisting R.C. frame is evaluated based on the both of new and current methods to ensure applicability of the proposed method.

Keywords: performance-based seismic design; equivalent static analysis; damage index; constant damage spectrum; dynamic analysis

1. Introduction

Despite all the progresses in designing and constructing more secure against earthquake impacts, the state of possible damages and losses is still strongly under question. Moreover, the consequence inspection of the previous earthquakes cleared the deficiencies of the current viewpoint i.e., force-based seismic design (Naeim 2004, Sezen *et al*. 2000, Xue 2000). As an example, the primary role of deformations in damage state of structural and non-structural components has been proved; while, they are only checked at the final steps of the current design procedures (Priestley *et al*. 2007). On the other hand, casualities, economic losses, downtime side-

*Corresponding author, Researcher, E-mail: allahvirdizadeh@ut.ac.ir

http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

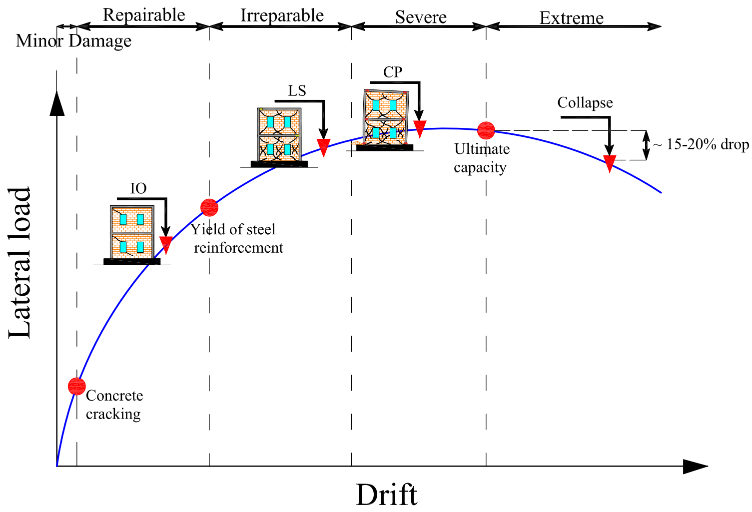


Fig. 1 Shematic relation between building performance and damage states

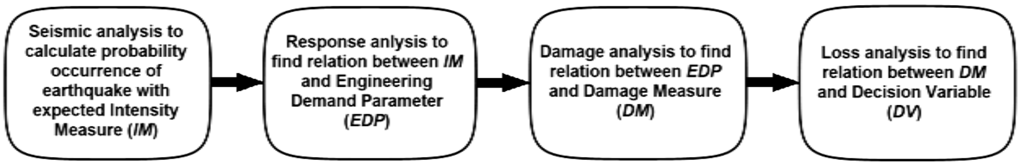


Fig. 2 Performance-based assessment framework of *PEER* (Cornell and Krawinkler 2000)

effects and induced uncertainties seem to be explicitly considered or even absent (Dhakal 2011). Hence, the researchers introduced a new concept known as Performance-Based Seismic Design (*PBSD*).

PBSD focuses on developing simple accurate response predicting tools to evaluate the probable damages at the various expected hazard levels. Subsequently, it proposes reliable thresholds meeting the objectives. Namely, the capacities will be compatible with the damage states (Behnam *et al.* 2006). A schematic relation between well-known performance limits (i.e., Immediate Occupancy (*IO*), Life Safety (*LS*) and Collapse Prevention (*CP*)) with the expected state of damages is shown in Fig. 1.

To limit the structural damage to expected thresholds and satisfy other expectations (e.g., injury or fatality of residents and the needed time for repair or demolition and reconstruction), a probabilistic performance-based assessment approach is suggested by the Pacific Earthquake Engineering Research Center (*PEER*). The proposed framework is shown in Fig. 2, which is now so time-consuming to be applied in the ordinary/common projects. In this regard, other researches have been conducted to develop more straightforward methods such as introducing more informative Damage Indices (*DI*) and finding relation between aforementioned parameters and the damages.

Several damage indices could be found in the literature, developed based on ductility, residual curvature, decayed strength, dissipated energy and etc. (Newmark and Rosenblueth 1971, Park *et al.* 1985, Wang and Shah 1987). Generally, well-defined *DI* is normalized; then, on the scale zero

Table 1 Relation between building performance and different levels of damage indices (Ghobarah 2004)

Degree of Damage	Element	Story	Global
Collapse	>0.8	>0.7	>0.6
Severe damage - Life safety - Partial collapse	0.6-0.8	0.5-0.7	0.4-0.6
Irreparable damage	>0.4	>0.3	>0.2
Repairable damage			
(a) Moderate	<0.4	<0.3	<0.2
(b) Light	<0.3	<0.2	<0.15
No damage	<0.2	<0.15	<0.1

to one in *DI*, respectively, the given element or building will be considered elastic or totally collapsed. It is believed that there is a shift between two situations: while the components experience earthquake damages and when the story or the whole building is gone under such experience. Therefore, different *DI* limits have been suggested for specific performance levels as illustrated in Table 1.

Given all the mentioned, it is vital for *PBSD* to develop analyses methods compatible with the aforesaid damage states. Therefore, capturing strength and stiffness degradations would be inevitable; this means that the nonlinear analyses are necessary. That also implies that more detailed properties should be introduced in comparing the former simplified elastic procedures. Hence, the final results will be drastically dependent on the designer's profession. In this regard, introducing simple analyses such as equivalent static, extracting approximating expressions through regression analysis and proposing the relation between inter-story drift ratio/ductility level/frequency shift with *DI* would be desirable (Aghagholizadeh *et al.* 2006, Erduran and Yakut 2004, Estekanchi and Arjomandi 2007, Habibi and Izadpanah 2012, Lu *et al.* 2009). The other solution, probably more acceptable one particularly for engineers, is to upgrade current approaches.

Current codes employ spectral ordinates based on the site classification and the fundamental period of desired structure to apply seismic actions. These inputs are modified by some factors related to the importance of the building, the lateral load bearing system and the construction material. But, they are mostly associated with the expected far-field ground motions and the specific viscous damping ratio (i.e., 5% of critical damping). On the other hand, the introduced limits are not as flexible and multi-disciplined as the *PBSD*s (Bommer and Pinho 2006, ISIRI2800 2005). Also, it has been shown that the current code-based lateral force distribution will cause the structures to undergo large inelastic deformations. Therefore, some researches have tried to introduce more straightforward compatible methods with the *PBSD*, such as producing optimal ground motion records based on the structural damage, plotting required yield strength to the target constant damage index and proposing a new distribution based on the inelastic behavior (Chao *et al.* 2007, Fajfar 1992, Jiang *et al.* 2013, HJ Jiang *et al.* 2012, Mikami and Iemura 2001, Moustafa 2011, Panyakapo 2004).

In this study, declared force level of the Iranian seismic design code (ISIRI 2800) is compared with the minimum required capacity to satisfy different damage states. For such reason, the constant damage spectrums for all the proposed soil types in ISIRI2800 and different ductility levels are plotted in a range of fundamental periods. Moreover, influence of viscous damping and return period of earthquake on final outcomes are investigated. Afterward, tri-linear formulas similar to those in the codes are proposed for all of the soil classifications. These simplified relations could be employed in further practical applications. Finally, a case study building with R.C. moment resisting lateral load bearing system is employed to evaluate the procedure applicability.

2. Theoretical background and assumptions

For developing the constant damage spectrums, the following assumptions are taken into account in the procedure:

• Based on the Iranian seismic design provisions (ISIRI 2800) which is adopted from UBC-97, equivalent static procedure is valid for regular buildings or the one with minor irregularity. Also, in the regular buildings, the height is limited to 50 m (ISIRI2800 2005, UBC-97 1997). These regulations could guarantee that mass participation factor of the first vibration mode is approximately more than 90%. In other words, the first mode of vibration has the highest contribution to the excited inertia and consequently the response of the building due to the ground motions. Therefore, the equivalent *SDOF* systems with bilinear behavior are considered as representative of the target buildings' global behavior. Considering all the aforesaid, the total induced lateral force of earthquake (i.e., base shear) would be calculated through the spectral ordinates of elastic spectrum and employing Eq. (1) (ISIRI2800 2005)

$$V = \frac{ABI}{R} W \tag{1}$$

Where, V is the base shear, A the mapped spectral acceleration, B the response coefficient of building, I the importance factor, R the response modification factor and W the effective weight (sum of the dead loads and a portion of live loads).

For the range of buildings mentioned in this article (those in regions with high seismicity), A equals to 0.35 and B is calculated from Eq. (2) (ISIRI2800 2005)

$$B = \begin{cases} 1 + S(\frac{T}{T_0}) & T \le T_0 \\ 1 + S & T_0 \le T \le T_S \\ (1+S)(\frac{T_s}{T})^{2/3} & T \le T_S \end{cases} \tag{2}$$

Where, S, T_0 and T_S are all dependent to the site classification and T is empirical fundamental period of the building, calculated by Eq. (3)

$$T = \alpha H^{3/4} \tag{3}$$

Where, H is height of the building from the base level and α is the constant value depending on the lateral load bearing system and the construction material. It is respectively 0.08, 0.07 and 0.05 for steel moment resisting frames, reinforced concrete moment resisting frames and other structural systems (ISIRI2800 2005).

Applying the height limit (i.e., 50 m from base) to Eq. (3) paves the way for the allowable valid period range of the equivalent static procedure. Therefore, the constant damage spectrums are

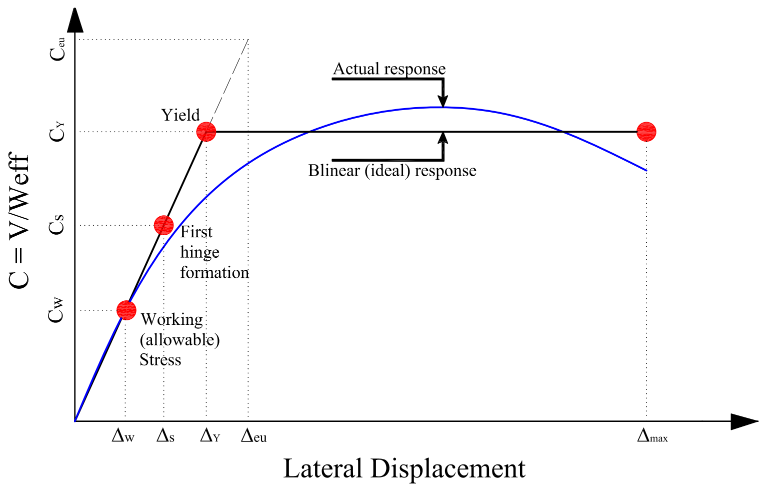


Fig. 3 Schematic relation between elastic and real response of structure

plotted up to 1.5 seconds.

The aforementioned base shear (Eq. (1)) is in the allowable (working) stress range, obtained through modifying the values from the elastic analysis and employing the response modification factor (*R*). This procedure makes applying elastic analysis to predicting nonlinear responses possible. The theoretical background is illustrated in Fig. 3. As given there, the response modification factor will be equal to Eq. (4)

$$R=\frac{C_{ue}}{C_W}=\frac{C_{eu}}{C_Y}\times\frac{C_Y}{C_S}\times\frac{C_S}{C_W}=R_\mu\times\Omega\times Y \tag{4}$$

Where, R_μ is ductility-based response modification factor, Ω is over-strength and Y is decreasing factor from the first hinge formation to the allowable stress. Y is approximately considered as 1.4 (ISIRI2800 2005). The other parameters are depicted in Fig. 3.

Where, C_{eu}, C_Y, C_S and C_w are respectively the normalized base shear coefficients obtained from elastic design spectrum, at yield point, at first hinge formation point and at working stress. Δ_W, Δ_S, Δ_Y and Δ_{eu} are lateral displacements corresponding to the aforementioned points. Finally, $\Delta_{\max}$ is the ultimate lateral displacement.

• The constant damage spectrum reports required yield strength (generally in normalized form) of considered *SDOF* at each period to satisfy the desired damage index. Actually, it is the same as C_Y depicted in Fig. 3. On that account, adapting this normalized coefficient to the equivalent static method will be aimed by applying the aforesaid transformation idea (Fig. 3) as Eq. (5)

$$C_{design-DI}=\frac{C_{y-DI}}{\Omega\times Y} \tag{5}$$

Where $C_{design\text{-}DI}$ is the design base shear coefficient and $C_{y\text{-}DI}$ is the normalized yield strength obtained from constant damage spectrum.

In this article, the well-known proposed damage index by Park and Ang is utilized to plot constant damage spectrums. It has been developed to assess the damage state in R.C. structures, but some researchers have employed it for steel structures (e.g., Kamaris *et al.* 2012).

It is as Eq. (6) (Park and Ang 1985)

$$DI_{PA} = \frac{\theta_m - \theta_y}{\theta_u - \theta_y} + \beta \frac{E_H}{M_y \theta_u} \tag{6}$$

Where, θ_m, θ_y and θ_u are the maximum, yield and ultimate/capacity rotations of the section, respectively. E_h is the total absorbed hysteretic energy, M_y is the yield moment capacity and β is the experimental-based constant parameter.

Since, the main objective of *PBSD* is to design structures which satisfy multi-level performances, two different performance levels (life safety and collapse prevention) and two hazard levels (10% and 2% possibility of occurrence in 50 years) are considered here. Based on Table 1, the target damage indices corresponding to the *LS* and *CP* levels are assumed to be 0.4 and 0.6, respectively (Ghobarah 2004, Jiang *et al.* 2013, Mikami and Iemura 2001).

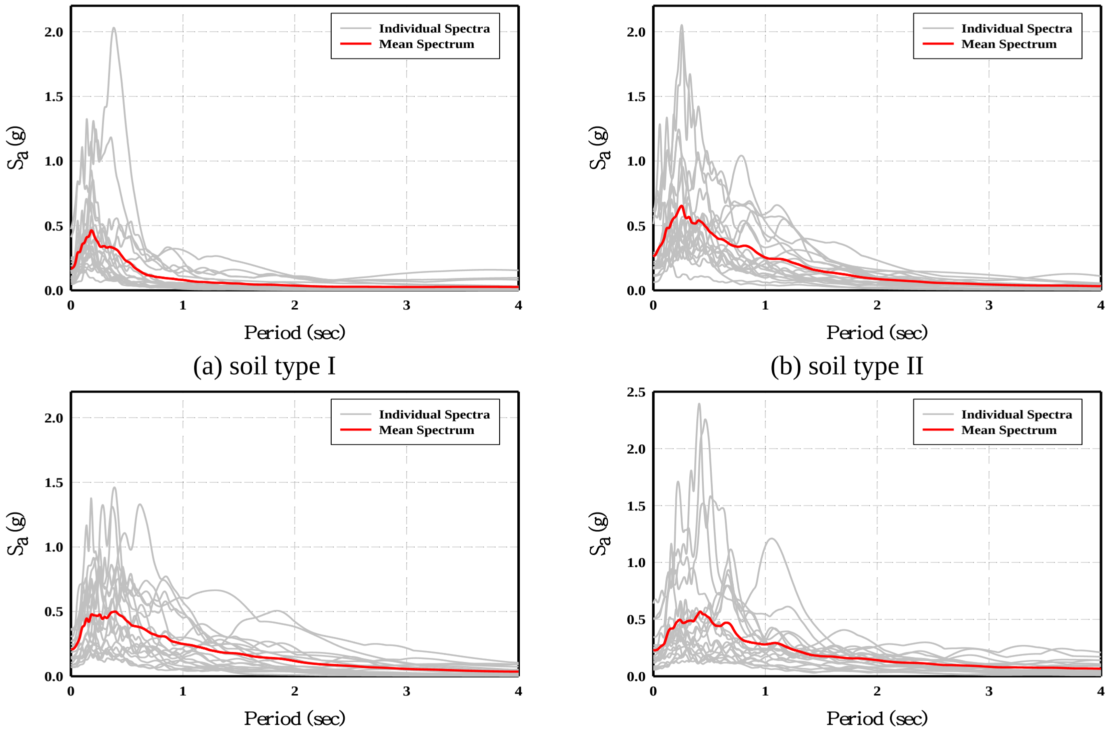
• Employed ground motion records are selected based on the soil type classification of the recording station, the magnitude, the closest distance to fault rupture and their significant duration. Hence, for each group of the selected records, shear wave velocity at station's site should be compatible with reported values in Table 2. Since, the categories of aimed hazard levels are as rare (475 years returning period) and very rare (2475 years returning period), they should be strong enough. Therefore, the magnitude of the selected ground motions is greater than 6.0. In the next steps, they are scaled to satisfy objectives of these hazard levels. On the other side, the applicability of equivalent static method is limited for the sites far enough from earthquake origin or fault rupture. In other words, far-field ground motions should be employed. To satisfy this aim, ground motions of recording stations with closest distance of at least 10 km from the fault rupture is selected. Finally, ISIRI2800 recommends that the significant duration of the selected ground motions be greater than 10.0 seconds (ISIRI2800 2005). This is achieved via the sorting records of each group based on their $D_{5\text{-}95}$. Individual and mean 5% damped elastic spectrums of the selected ground motions are shown in Fig. 4(a)-(d) (PEER 2006).

Pertaining to the PBEE idea, it is expected for the designed buildings to satisfy different performance objectives at a variety of probable future risk levels. In this regard, satisfying Collapse Prevention (CP) at Maximum Considered Earthquake (MCE) and Life Safety (LS) at Design Based Earthquake (DBE) are basic interests. The MCE target spectrum is achieved through multiplying DBE's one in 3/2 (ASCE7-10 2010). Therefore, the selected records are needed to be scaled to ensure the desired hazard level. Common code-based scaling will result specific scale factors for each of the assumed fundamental periods in SDOF systems. It has been showed that the aforementioned method could result in less displacement demands in comparison of some other new methods e.g., response spectrum matching (Allahvirdizadeh *et al*. 2013). Spectrum matching

Table 2 Soil classification based on the shear wave velocity (ISIRI2800 2005)

Soil Type	Description	Shear Wave Velocity (m/sec)
I	Hard Rock / Rock	$V_s^{*}>750$
II	Very Dense Soil and Soft Rock	$375<V_s<750$
III	Stiff Soil	$175<V_s<375$
IV	Soft Clay Soil	$V_s<175$

*V_s is average shear wave velocity at depth of 30 m from the base level.



(a) soil type I (b) soil type II

(c) soil type III (d) soil type IV

Fig. 4 damped (5%) elastic spectrum of selected ground motion records

includes nonlinear frequency manipulation to modify spectral shape, resulting in reducing *EDP* dispersion (Seifried 2013). On that account, the records are adjusted to the target response spectrums (ISIRI2800 *DBE* design spectrum and transformed *MCE* spectrum) using *SeismoMatch* software (Seismosoft 2013). The matched records are shown in Fig. 5(a)-(h). Possible changes in characteristics of the records, due to scaling method, are beyond the scope of the article, then being neglected.

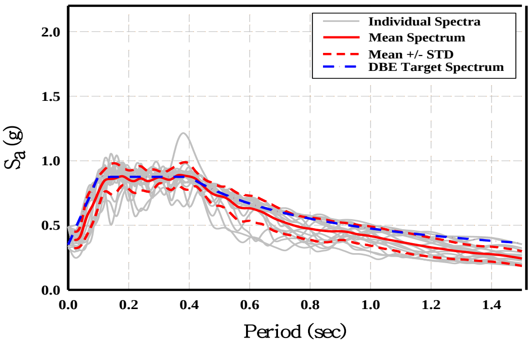
• In the next paragraphs, the sensitivity of constant damage spectrum to some possible important parameters such as ductility, damping ratio, employed hysteresis model and soil type of the site is evaluated.

As previously mentioned, applied response modification factor (R) is composed of ductility-based response modification factor, over-strength and transforming factor from the first hinge formation to allowable stress range. While, the ductility-based term is absent in the denominator of Eq. (5). Actually, it is embedded in the considered damage index in Eq. (6). But, in order to study the results sensitivity, the ductility demand could be approximated by the bilinear schematic response of buildings, as depicted in Fig. 3 as Eq. (7)

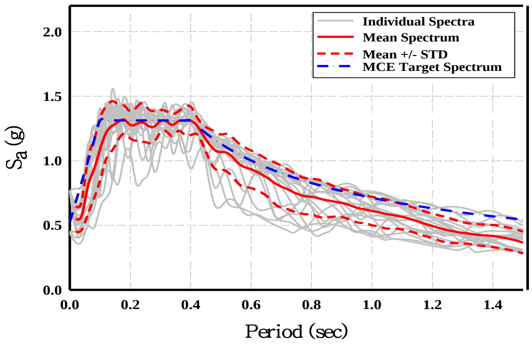
$$\mu_{demand}=\frac{\Delta_{\max}}{\Delta_Y}=\frac{\Delta_{\max}}{\Delta_S}\times\frac{\Delta_S}{\Delta_Y}=C_d\times\frac{C_S}{C_Y}=\frac{C_d}{\Omega} \qquad (7)$$

Where, μ_{demand} is the ductility demand and the others are same as previous.

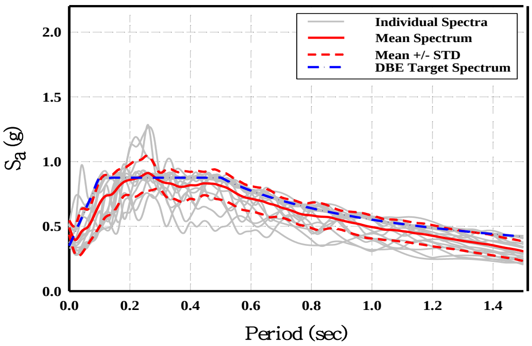
The design coefficients for some of the most popular structural systems in the Iranian practice



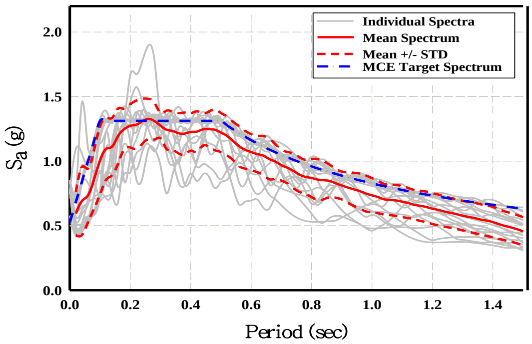
(a) *DBE* target spectrum of soil type *I*



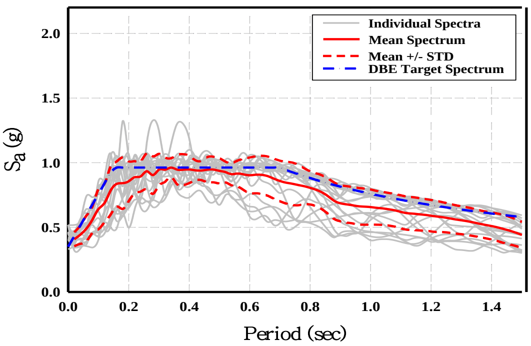
(b) *MCE* target spectrum of soil type *I*



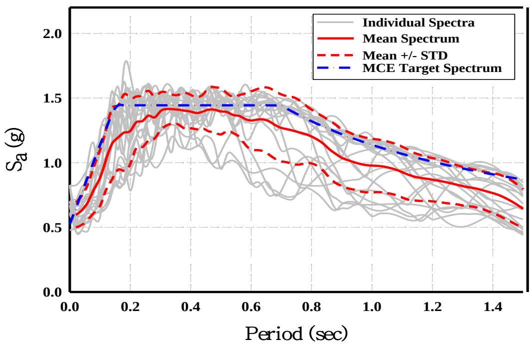
(c) *DBE* target spectrum of soil type *II*



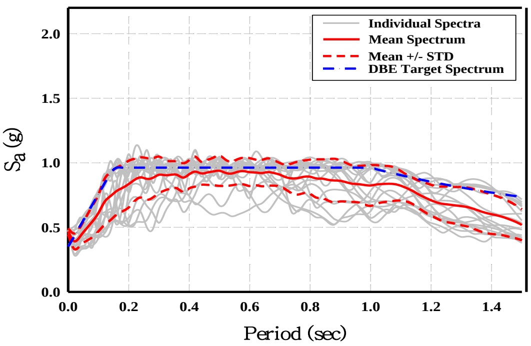
(d) *MCE* target spectrum of soil type *II*



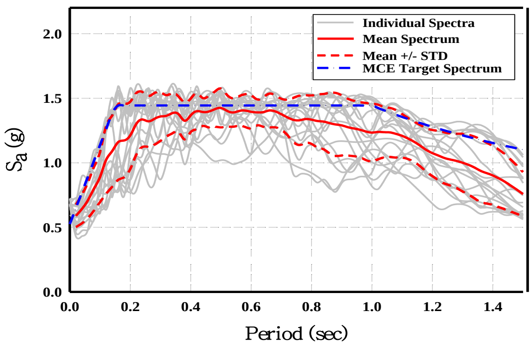
(e) *DBE* target spectrum of soil type *III*



(f) *MCE* target spectrum of soil type *III*



(g) *DBE* target spectrum of soil type *IV*



(h) *MCE* target spectrum of soil type *IV*

Fig. 5 Matched (scaled) selected ground motion records are collected in Table 3 (ISIRI2800 2005, ASCE7-10 2010). The response modification factors (*R*) are based on ASCE7-10, which should be multiplied by 1.4 to gain ISIRI2800 values. As it could be concluded, the approximated ductility demand will be in range of 1.5 to 2.0. While, the ductility capacity of the typical buildings expected to be in range of 3.0-5.0. Hence, the constant damage spectrums are plotted in ductility range of 1.5-5.0, as shown in Fig. 6. As it could be concluded, the damage spectrum is significantly sensitive to the ductility. As it is obvious, the maximum

Table 3 Design coefficients of some popular structural systems (ISIRI 2800, 2005, ASCE7-10 2010)

Seismic force resisting system	Response modification factor(R)	Over-strength factor(Ω)	Deflection amplification factor(C_d)
Special R.C. shear walls	5	2.5	5
Steel eccentrically braced frames	8	2	4
Steel ordinary concentrically braced frames	3.25	2	3.25
Steel special moment frames	8	3	5.5
Special R.C. moment frames	8	3	5.5
Intermediate R.C. moment frames	5	3	4.5

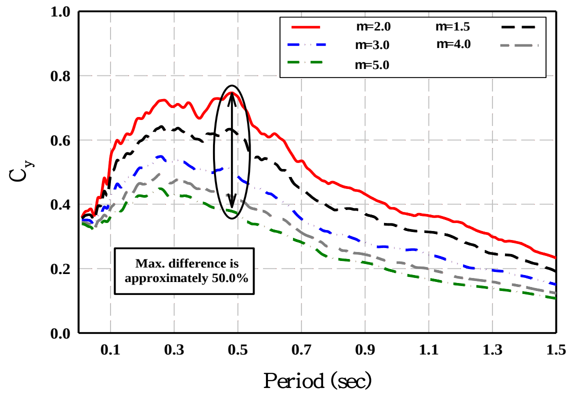


Fig. 6 Sensitivity of constant damage spectrum to ductility

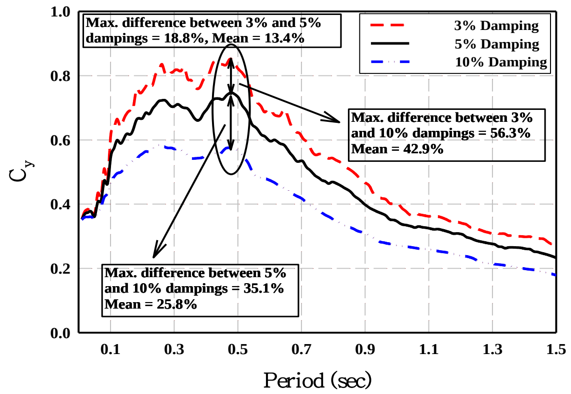


Fig. 7 Sensitivity of constant damage spectrum to damping ratio

values belong to the approximated ductility level from the schematic bilinear response. Hence, in further investigations of the current article, the ductility is conservatively taken 1.5.

Generally, the elastic spectrums proposed by seismic deign regulations (such as ISIRI 2800) are plotted for 5% of critical damping (ISIRI2800 2005), While the different materials have various inherent characteristics. Therefore, three values are taken to evaluate the influence of viscous damping ratio on outcomes of the constant damage spectrum (Fig. 7).

As it was expected, the less damping ratios result in the higher required yield strength. The mean difference for 3% and 5% (which are the most popular values in literature) is 13.4%. This difference in the case of 5% and 10% of critical damping increases up to 25.8%. Further investigations will be based on 5% viscous damping to enable comparison with current elastic spectrums. But making modifications in specific cases would be necessary.

The second term of Eq. (6) relates to the absorbed hysteresis energy and the monotonic energy capacity. On that account, the employed hysteresis model could affect the calculated damage index. Hence, sensitivity of the final outcomes to that is needed to be investigated. Based on studies of Jiang *et al.* modified Clough and Takeda hysteresis rules (which are appropriate for R.C. structures) lead to almost identical results (Jiang *et al.* 2013, Takeda *et al.* 1970). In this regard, bilinear (mostly used for steel structures modeling) and modified Clough hysteresis rules are compared. As it is evident in Fig. 8, almost the same results are obtained. Since the results of modified Clough are slightly greater, it has been employed in constant damage spectrums construction.

In order to evaluate the sensitivity of damage spectrums to the site soil conditions, the identical *SDOF* systems i.e., with same mass, viscous damping, ductility and hysteresis material behavior were considered. These models are excited by specific scaled ground motions compatible with the previously mentioned site conditions (Table 2). Constructed constant damage spectrums are illustrated in Fig. 9. It can be concluded that the site conditions has no major influences on the normalized yield strength during short periods, but the differences are significant particularly in descending branch of the spectrums. Furthermore, shifting periods seem so sensitive to the site soil conditions. Then, specific constant damage spectrums for each site soil type are constructed.

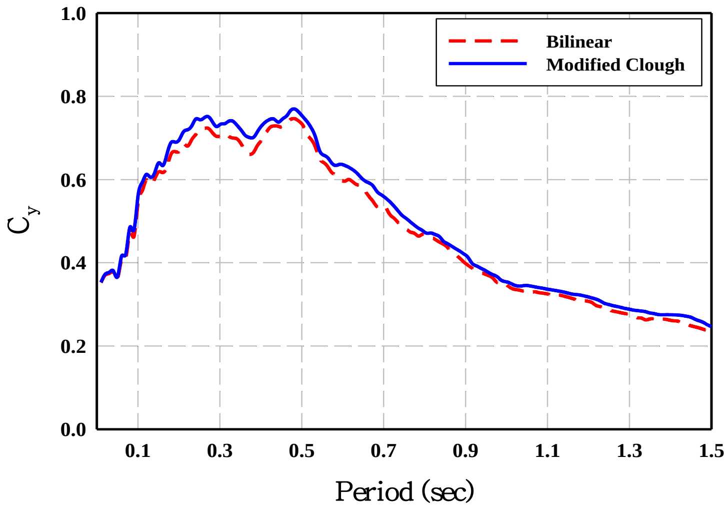


Fig. 8 Sensitivity of the constant damage spectrum to the employed hysteresis model

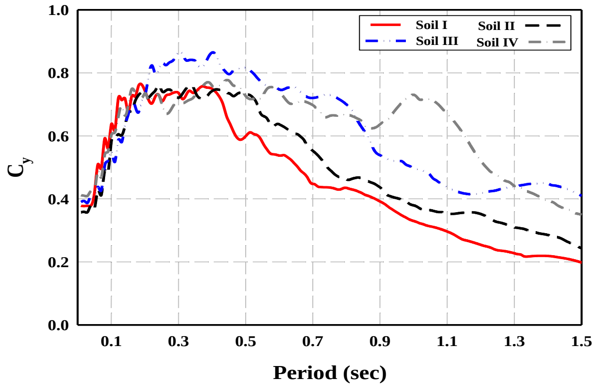


Fig. 9 Sensitivity of the constant damage spectrum to the site soil type

3. Proposing a new damage-based spectrum

The individual and mean constant damage spectrums corresponding to *LS* and *CP* performance levels are illustrated in Fig. 10(a)-(h).

These spectrums are obtained from analyzing the scaled considered ground motion records in all of the site classifications against each of the hazard levels. In order to make these spectrums applicable, they are needed to be formularized. For meeting such objective, a tri-linear spectrum similar to the proposed one by ISIRI 2800 is formulated (showed as a dashed line in Fig. 10(a)-(h)). The proposed equation is Eq. (8)

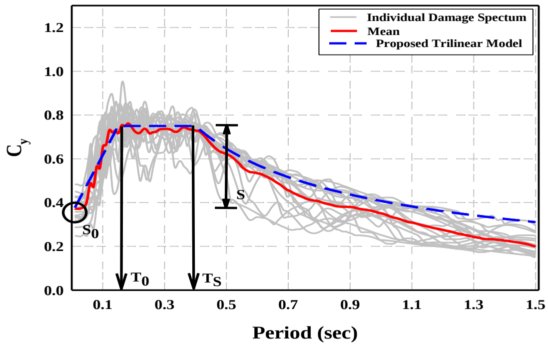
$$C_{y-DI} = \begin{cases} S_0 + S(\frac{T}{T_0}) & T \le T_0 \\ S_0 + S & T_0 \le T \le T_S \\ (S_0 + S)(\frac{T_S}{T})^{2/3} & T \ge T_S \end{cases} \tag{8}$$

Where, $C_{y\text{-}DI}$ is the normalized required yield strength, T_0 and T_S are the transition periods at the breaking points of the tri-linear spectrum, S_0 is the normalized required yield strength at period equal to zero and S is the increase in the normalized required yield strength from period zero to the flat part of the spectrum. T_0 and T_S are only function of the site classification, where S_0 and S are dependent on the performance level.

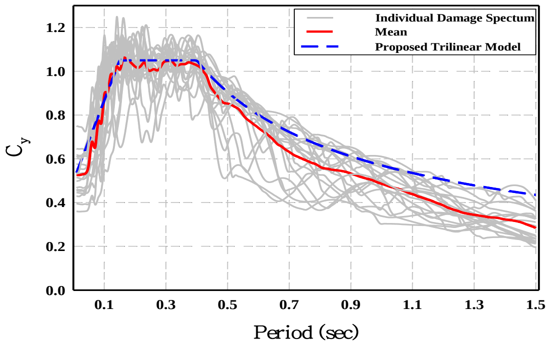
The suggested values for the former parameters are given in Table 4.

Table 4 Characteristics of the damage-based spectrum

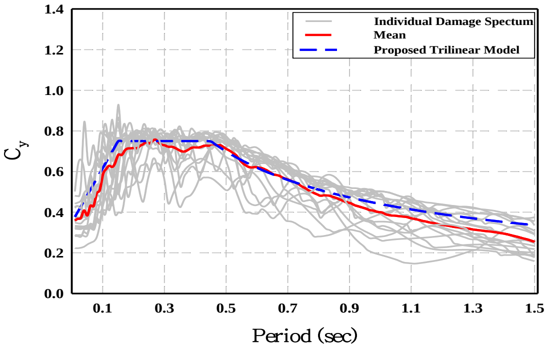
Soil Type	T_0	T_S	***LS***		***CP***	
			S_0	S	S_0	S
I	0.15	0.4	0.35	0.4	0.5	0.55
II	0.15	0.45	0.35	0.4	0.5	0.55
III	0.2	0.65	0.35	0.45	0.55	0.6
IV	0.2	0.85	0.35	0.45	0.55	0.6



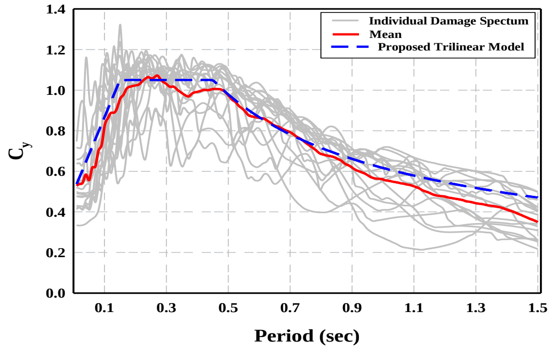
(a) *LS* performance level - Soil *I*



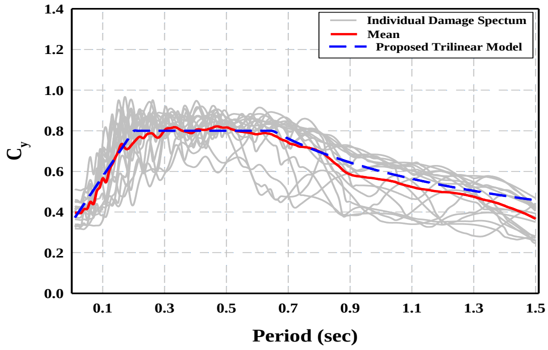
(b) *CP* performance level - Soil *I*



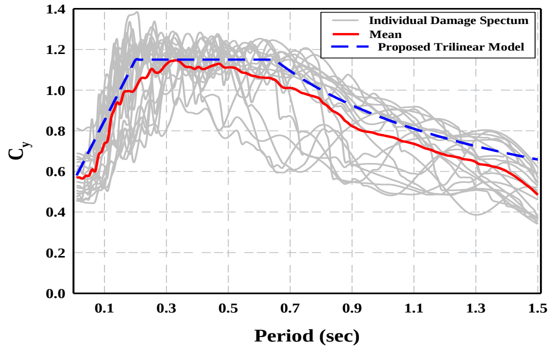
(c) *LS* performance level - Soil *II*



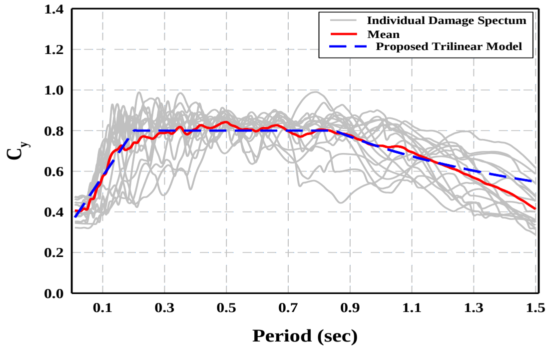
(d) *CP* performance level - Soil *II*



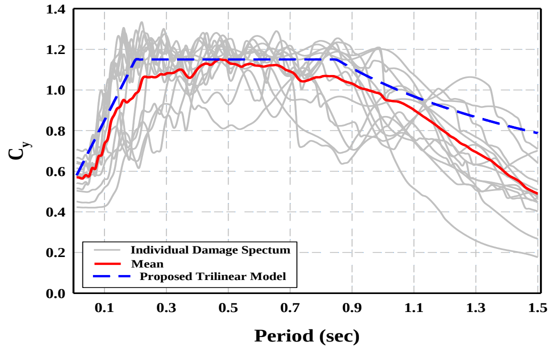
(e) *LS* performance level - Soil *III*



(f) *CP* performance level - Soil *III*



(g) *LS* performance level - Soil *IV*



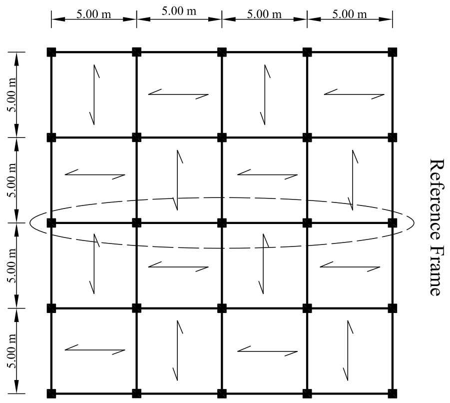
(h) *CP* performance level - Soil *IV*

Fig. 10 Constructed constant damage spectrums for all of the soil types at different performance levels

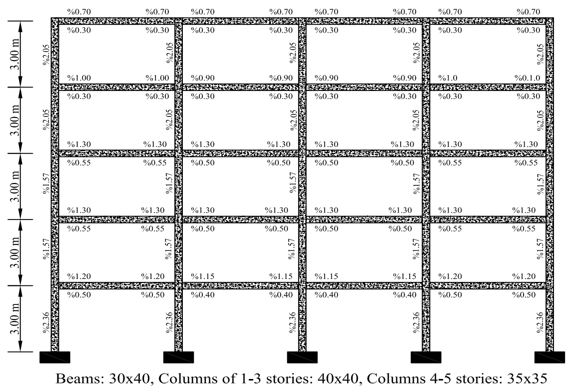
4. Case study example

In order to evaluate the efficiency of the proposed method, a 5-story moment-resisting R.C. frame is considered as the case study building. The ductility level is intermediate, namely, its *R* in the current seismic designs is 5.0 as given in Table 3. This lateral load bearing system is one of the most popular structural types in Iranian practice. Their plan and occupancy are assumed as typical regular residential. Also, the all stories' height is equal to 3.0 m and the soil type is considered as *II*. Section proportioning is done through following three procedures: the current Iranian seismic design regulations, the proposed procedure for *LS* and the *CP* performance objectives. The considered plan, the section dimensions and the reinforcement details are illustrated in Fig. 11.

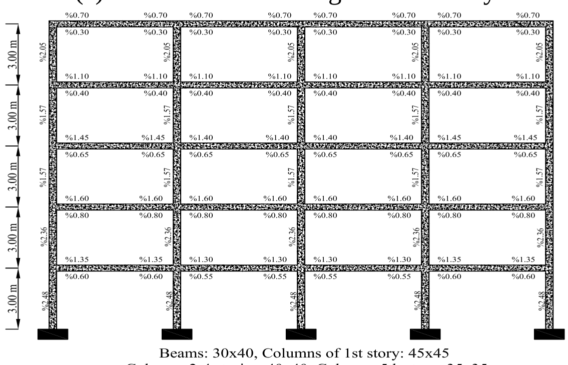
In order to capture nonlinear responses, the concentrated plasticity (plastic hinge) modeling method is utilized, where the recommended tri-linear behavior by FEMA-356 is applied to the mid of the sections' plastic length (FEMA-356, 2000). The base shear-roof displacement diagram of the pushover analysis under the first mode lateral load pattern is provided in Fig. 12. As it is evident, 28.4% over-strength in the *LS* new design method with respect to current codified procedure is induced to system. Hence, the ongoing seismic design methods may initially cause a more economical output. But, further investigations for comparing the post-earthquake monetary losses are required to determine rational ones. In other words, more controlled experienced



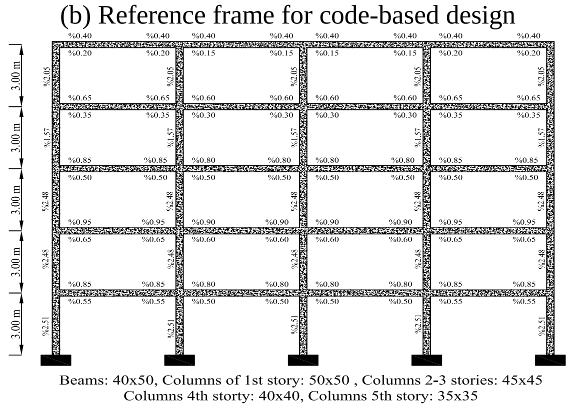
(a) Plan view of buildings in case study



(b) Reference frame for code-based design



(c) Reference frame for *LS* new method



(d) Reference frame for *CP* new method

Fig. 11 Plan view and considered reference frames of case study buildings

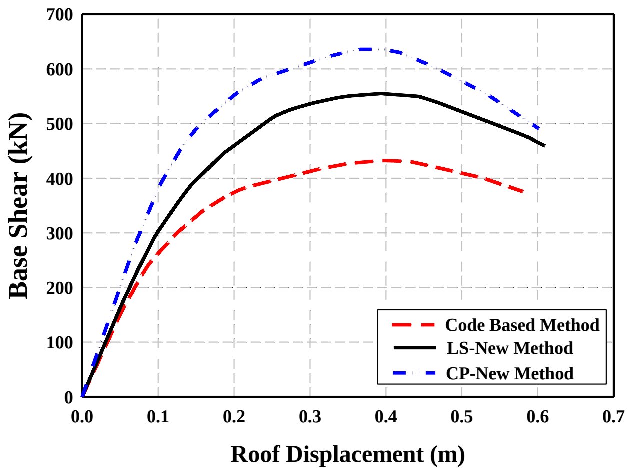


Fig. 12 Base shear- roof displacement diagram of different design methods and performance levels

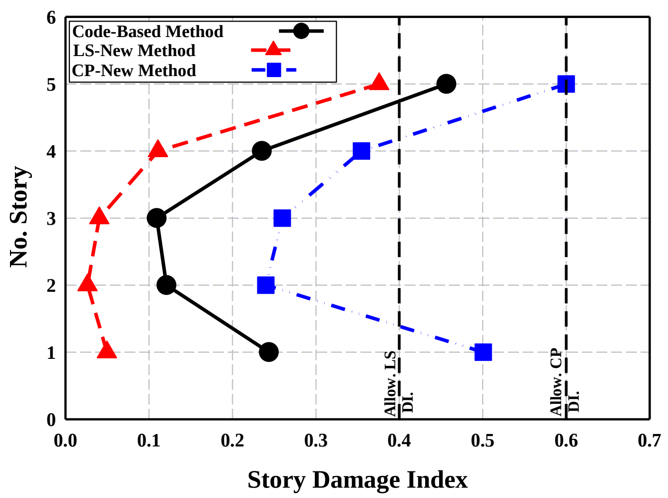


Fig. 13 Comparison between story level damage index of different design methods and performance levels

damages in the new method will decrease other important factors in the economics of the design i.e., the number of injuries/fatalities, down-time, the repair costs, etc. The over-strength at the *CP* performance objective of the new design method in comparison of the code-based design and the *LS* performance level of the new method is 47.1% and 14.6% respectively.

For checking the applicability, a set of seven ground motion records (other than previously selected ones) are taken to perform nonlinear time-history analysis. Calculated damage indices at elements are transformed to damage index at story level by employing Eq. (9) (Park and Ang 1985).

$$DI_{Story} = \left(\sum_{i=1}^{N} DI_{element,i} \times E_{H,i} \right) \Big/ \sum_{i=1}^{N} E_{H,i} \tag{9}$$

Where, DI_{story} and $DI_{element}$ are damage indices at story and element level respectively. E_H is the total absorbed energy by elements of the considered story and *N* is the number of elements at the story.

The Distribution of story damages at the height of case study buildings, obtained from various design methods, is illustrated in Fig. 13. The allowable normalized damage states are showed as the dashed lines.

Evidently the case designed based on the current code has surpassed the allowable life safety damage state. In other words, the aforementioned building would not satisfy the life safety performance level against an earthquake with 475-year returning period or based on Table 1 values the damages would be irreparable, while the designed buildings using the new procedure are acceptably included in both of the *LS* and *CP* performance levels. No comparison is made for *CP* performance level, since there is no threshold for that in ISIRI 2800. In addition, the story damage index distribution in the building height in all performance levels makes it clears that the traditional force distribution results in an almost large dispersion. Therefore, further studies are needed to introduce a more optimal story shear distribution as an objective in more economical designs.

5. Conclusions

In the present paper, the equivalent static procedure in the current seismic design regulations as the most popular analysis method for the regular buildings is modified to be compatible with the performance-based earthquake engineering objectives. For meeting this aim, the damage index as one of the common parameters in the related state of damage on the element, story and structure level, in performance objectives is considered. On that account, the required capacity at each damage state corresponding to the main performance levels (i.e., life safety and collapse prevention) and risk levels (i.e., earthquakes with 10% and 2% probability of exceeding in 50 years) has been intended to evaluate the damage states. In order to do that, the equivalent *SDOF* system as the representative of the buildings with the first mode dominant behavior was utilized to conduct nonlinear time-history analysis and plot constant damage spectrums. Then, the modification factors (i.e., over-strength and transforming factor from the first hinge formation to the allowable stress) are taken to obtain the yield strength in each fundamental period to calculate the required strength at any working stress levels. The sensitivity of results to the system ductility, viscous damping, the employed hysteresis model and the site soil type are investigated to propose a more stable procedure. Finally, tri-linear formulas for all of the soil classifications of the Iranian seismic design regulations are recommended as a function of fundamental period. In the end, the damage state investigation of a case study building has shown that the ongoing design methods (e.g., ISIRI 2800) could lead to unreliable results through surpassing the desired performance level, while new procedure satisfies acceptable thresholds. The results show that new method would induce more over-strength to the system, compared to the codified procedure. This over-strength affects the initial cost of constructions, while taking into account the post-earthquake costs and the insurance issues could make the additional costs reasonable. In addition, investigating distribution of the damage index at the height of building illustrate the deficiencies of the current base-shear distribution. Therefore, it causes a single story to control the design or in other words, the considerable capacity of other stories would remain useless.

References

Aghagholizadeh, M. and Massumi, A. (2012), "Relation between dynamic characteristics and damage index of RC-MRFs using non-linear incremental dynamic analyses", *The 15th World Conference on Earthquake Engineering (15WCEE)*, Lisbon, Portugal.

Allahvirdizadeh, R., Khanmohammadi, M. and Marefat, M.S. (2013), "Investigating effects of scaling and selecting earthquake ground motions on performance-based design of R.C buildings", *4th International Conference on Concrete and Development*, Tehran, Iran.

ASCE7-10 (2010), *Minimum Design Loads for Buildings and Other Structures*, Reston.

Behnam, B., Sebt, M.H. and Vosoughifar, H.M. (2006), "Evaluating quality seismic damage index for urban residential buildings", *Int. J. Civ. Eng.*, **4**(2), 136-145.

Bommer, J.J. and Pinho, R. (2006), "Adapting earthquake actions in Eurocode 8 for performance-based seismic design", *Earthq. Eng. Struct. Dyn.*, **35**(1), 39-55.

Calvi, G.M., Pinho, R., and Crowley, H. (2006), "State-of-the-knowledge on the period elongation of RC buildings during strong ground shaking", *First European Conference on Earthquake Engineering and Seismology*, Geneva, Switzerland.

Chao, S.-H., Goel, S.C. and Lee, S.-S. (2007), "A seismic design lateral force distribution based on inelastic state of structures", *Earthq. Spectra*, **23**(3), 547-569.

Cornell, C.A. and Krawinkler, H. (2000), *Progress and challenges in seismic performance assessment*, PEER Center News 2000, 3, 1-3.

Dhakal, R.P. (2011), *Structural Design for Earthquake Resistance: Past, Present and Future*, Report for Canterbury Earthquakes Royal Commission: Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand.

Erduran, E. and Yakut, A. (2004), "Drift based damage functions for reinforced concrete columns", *Comput. Struct.*, **82**(2), 121-130.

Estekanchi, H. and Arjomandi, K. (2007), "Comparison of damage indexes in nonlinear time history analysis of steel moment frames", *Asian J. Civ. Eng. (Building and Housing)*, **8**(6), 629-646.

Fajfar, P. (1992), "Equivalent ductility factors, taking into account low-cycle fatigue", *Earthq. Eng. Struct. Dyn.*, **21**(10), 837-848.

FEMA-356 (2000), *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, Washington D.C., USA.

Ghobarah, A. (2004), "On drift limits associated with different damage levels", *Proceedings of an International Workshop on Performance-based Seismic Design Concepts and Implementation*, Bled, Slovenia.

Habibi, A.R. and Izadpanah, M. (2012), "Drift based damage functions for reinforced concrete moment resisting frames", *J. Seismol. Earthq. Eng.*, **14**(2), 117-129.

ISIRI2800 (2005), *Iranian code of practice for seismic resistant design of buildings*, 3rd version, Tehran, Iran. (in Persian)

Jiang, H., Fu, B. and Chen, L. (2013), "Damage-control seismic design of moment-resisting RC frame buildings", *J. Asian Architect. Build. Eng.*, **12**(1), 49-56.

Jiang, H., Lu, X., Fu, B. and Chen, L. (2012), "Constant-damage yield strength spectra", *Proceedings of the 15th World Conference of Earthquake Engineering (15WCEE)*, Lisbon, Portugal.

Kamaris, G.S., Vallianatou, Y. and Beskos, D.E. (2012), "Seismic damage estimation of in-plane regular steel moment resisting and x-braced frames", *Bull. Earthq. Eng.*, **10**(6), 1745-1766.

Lu, Y., Gu, X. and Wei, J. (2009), "Prediction of seismic drifts in multi-storey frames with a new storey capacity factor", *Eng. Struct.*, **31**(2), 345-357.

Mikami, T. and Iemura, H. (2001), "Demand spectra of yield strength and ductility factor to satisfy the required seismic performance objectives", *Proceedings of JSCE*, Japan.

Moustafa, A. (2011), "Damage-based design earthquake loads for single-degree-of-freedom inelastic structures", *J. Struct. Eng.*, **137**(3), 456-467.

Naeim, F. (2004), "Impact of the 1994 Northridge earthquake on the art and practice of structural engineering", *Struct. Des. Tall Spec. Build.*, **13**(5), 373-389.
Newmark, N.M. and Rosenblueth, E. (1971), *Fundamentals of Earthquake Engineering*, Prentice Hall, Englewood Cliffs.
Panyakapo, P. (2004), "Evaluation of site-dependent constant-damage design spectra for reinforced concrete structures", *Earthq. Eng. Struct. Dyn.*, **33**(12), 1211-1231.
Park, Y.-J. and Ang, A.H.S. (1985), "Mechanistic seismic damage model for reinforced concrete", *J. Struct. Eng.*, **111**(4), 722-739.
PEER. (2006), PEER NGA Database, Available at http://peer.berkeley.edu/nga
Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007), *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, Italy.
Seifried, A.E. (2013), "*Response spectrum compatibilization and its impact on structural response assessment*", Ph.D. thesis, Stanford University, California.
Seismosoft. (2013), SeismoMatch v 2.1. Available at http://seismosoft.com.
Sezen, H., Elwood, K.J., Whittaker, A.S., Mosalam, K.M., Wallace, J.W. and Stanton, J.F. (2000), *Structural engineering reconnaissance of the August 17, 1999 earthquake: Kocaeli (Izmit), Turkey*: Pacific Earthquake Engineering Research Center (PEER), California.
Takeda, T., Sozen, M.A. and Nielsen, N.N. (1970), "Reinforced concrete response to simulated earthquakes", *J. Struct. Div.*, **96**(12), 2557-2573.
UBC-97 (1997), *Uniform Building Code*, Whittier, California.
Wang, M.-L. and Shah, S.P. (1987), "Reinforced concrete hysteresis model based on the damage concept", *Earthq. Eng. Struct. Dyn.*, **15**(8), 993-1003.
Xue, Q. (2000), "Need of performance-based earthquake engineering in Taiwan: a lesson from the Chichi earthquake", *Earthq. Eng. Struct. Dyn.*, **29**(11), 1609-1627.

CC