Structural Engineering and Mechanics, *Vol. 65, No. 1 (2018) 53-68*
DOI: https://doi.org/10.12989/sem.2018.65.1.053

Damage controlled optimum seismic design of reinforced concrete framed structures

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(Received April 2, 2017, Revised October 24, 2017, Accepted October 27, 2017)

Abstract. In this paper, an innovative procedure is proposed for the seismic design of reinforced concrete frame structures. The main contribution of the proposed procedure is to minimize the construction cost, considering the uniform damage distribution over the height of structure due to earthquake excitations. As such, this procedure is structured in the framework of an optimization problem, and the initial construction cost is chosen as the objective function. The aim of uniform damage distribution is reached through a design constraint in the optimization problem. Since this aim requires defining allowable degree of damage, a damage pattern based on the concept of global collapse mechanism is presented. To show the efficiency of the proposed procedure, the uniform damage-based optimum seismic design is compared with two other seismic design procedures, which are the strength-based optimum seismic design and the damage-based optimum seismic design. By using the three different seismic design methods, three reinforced concrete frames including six-, nine-, and twelve-story with three bays are designed optimally under a same artificial earthquake. Then, to show the effects of the uniform damage distribution, all three optimized frames are used for seismic damage analysis under a suite of earthquake records. The results show that the uniform damage-based optimum seismic design method renders a design that will suffer less damage under severe earthquakes.

Keywords: structural damage; collapse mechanism; seismic damage analysis; reinforced concrete; construction cost; optimization

1. Introduction

The widespread seismic design procedure based on the strength principle fails to directly consider the inelastic demands of structural systems during moderate to severe earthquakes. In theory, the seismic design of structures is satisfactory when their limit capacity is more than their seismic demand during the earthquake. It can be stated that the most important challenge regarding the seismic design is the identification of the most proper seismic demand from structural response parameters.

Current design codes use such parameters as the ductility ratio, the global response modification factor (often termed the R-factor) and the Peak Ground Acceleration (PGA) in the seismic design. Mahin and Bertero (1981), Zahrah and Hall (1984), Fajfar (1992), Basu and Gupta (1996) showed that ductility is not a reliable index by itself because it does not take into account the effects of the duration of strong motion, frequency content, and cumulative inelastic deformations. In addition, laboratory studies showed that ductility cannot provide an acceptable index for seismic damage (Banon and Veneziano 1982). Regarding the PGA, past earthquakes showed that this parameter does not correlate with the observed damages, due to lack of information on the duration of the strong motion (Housner 1975, Kennedy 1981, Miyakoshi and Hayashi 2000, Elenas and Meskouris 2001). In effect, not only is the strength-based design a reliable method for seismic design, but also the parameters used to obtain the force demand on the structure are not efficient. For instance, the R-factor used to simplify the elastic analysis process can be used to approximately predict the expected inelastic demands of the structure subjected to the design loads. Since this is a global factor, the role of the structural elements' behavior is not considered directly or efficiently for the analysis and design. The lessons learned from the past destructive earthquakes (e.g., 1989 Loma Prieta, 1990 Manjil-Rudbar, 1994 Northridge, and 1995 Kobe earthquakes) resulted in the conclusion that the seismic design derived from the single parameter of strength and its dependent parameters do not capture the expected inelastic demands of structures during severe earthquakes, and the structures may suffer major, unexpected, and irreparable degrees of damage under the earthquakes.

Over the last two decades, the structural design codes experienced numerous revisions. In this regard, researchers and engineers in the structural earthquake engineering fields developed new and practical design methodologies to guarantee suitable seismic performance for structures during earthquake excitations. One of the modern methodologies that has received significant attention is the performance-based seismic design (PBSD) (Kunnath 2005). The conceptual framework of PBSD developed over the last 20 years by the various guidelines such as Structural Engineers Association of California (SEAOC) (1995), Applied Technology Council (ATC) (1997), and Federal Emergency Manage Agency (FEMA) guidelines (2000).

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ISSN: 1225-4568 (Print), 1598-6217 (Online)

Several studies proposed damage-based analysis and design methods in the context of PBSD. Some of these studies emphasized the seismic damage analysis methods (e.g., Fajfar and Gaspersic 1996, Moustafa 2011)), and the others proposed damage-based design methodologies (e.g., (Chung *et al*. 1993, Cruz and Lopez 2004, Gharehbaghi *et al*. 2011, Hajirasouliha *et al*. 2012, Heidari and Gharehbaghi 2015)).

A satisfactory design candidate could be obtained using the optimization procedure (Lagaros *et al*. 2006). In the field of optimum seismic design of RC structures, several studies were carried out. Ganzerli *et al.* (2000) first proposed a formulated procedure that became a milestone in the performance-based optimum seismic design of structures. Zou *et al*. (2004) also presented a multi-objective optimization process for the performance-based seismic design of RC structures considering life-cycle cost. Lagaros and Papadrakakis (2007) optimized RC structures considering two strength- and performance-based methods using the multi-objective optimization technique. Fragiadakis and Papadrakakis (2004) also dealt with the performance-based optimum seismic design of RC plane framed structures considering two concepts: inelastic inter-story drift ratio, and mean annual frequency of earthquake events. Hajirasouliha *et al*. (2012) presented an efficient performance-based seismic design of RC moment resisting frames considering both nonlinear static and nonlinear dynamic analysis methods. Some studies were reported by the Author and colleagues on the optimization of RC frames under time history earthquake loads (Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015, Gharehbaghi *et al*. 2016). Recently, seismic design optimization of RC frame structures considering soil-structure interaction effects and reliability constraints was also reported (Khatibinia *et al*. 2013, 2015, Yazdani *et al*. 2016).

Considering the studies so far reported in the literature and buildings damages observed during the past destructive earthquakes, in this paper, an innovative and fully automated procedure is proposed for the PBSD of RC frame structures. The main objective of the proposed procedure was to minimize the construction cost and maximize the structural safety using the uniform damage distribution concept. For this purpose, as the use of structural optimization has been recommended for PBSD of RC structures (e.g. Lagaros *et al*. 2006), this automated procedure was structured in the framework of an optimization problem. The initial construction cost and the real valued version of particle swarm optimization (PSO) were considered as an objective function and the optimizer algorithm, respectively. The successful application of this evolutionary algorithm could be found in the works recently reported by author *et al*. (Gharehbaghi *et al*. 2011, Gharehbaghi and Salajegheh 2011, Gharehbaghi *et al*. 2012, Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015, Gharehbaghi *et al*. 2016). Moreover, two other design methods were considered to show the effectiveness of the proposed procedure and the importance of the uniform damage distribution. In this regard, three design methods including strength-based optimum seismic design (SD), damage-based optimum seismic design (DD), and uniform damage-based optimum seismic design (UDD) were evaluated and compared together under a suite of spectrum compatible earthquake records. The SD method that is the common design procedure is based on the strength principle conforming to the ACI318 (2011) and the Iranian Code of Practice for Seismic Resisting Design of Buildings (called 2800-code) (2004). In the DD method, ACI318 (2011) provisions and the levels of local and global damage based on the modified Park-Ang damage index (Kunnath *et al*. 1992) were considered as design constraints. For this purpose, a damage pattern for both structural elements and overall structure is proposed attempting to reach a desired collapse mechanism of global type (Montuori *et al*. 2015, Montuori and Muscati 2015, 2016) under severe earthquakes. The only difference between the UDD and DD methods is that the uniform distribution of damage over the stories was considered as another acceptance criterion for UDD. Both the DD and UDD are considered as PBSD methods. In this context, three six-, nine- and twelve-story RC frames were optimally designed using these three seismic design methods under a single artificial earthquake compatible with the 2800-code elastic response spectrum. During the design optimization process, linear time-history analysis (LTHA) was carried out for SD, and a non-linear time-history analysis (NLTHA) was carried out for both DD and UDD. To demonstrate the effects of damage distribution, the seismic damage analysis (SDA) was performed considering ten spectrum-compatible earthquake records. These records were then applied to the NLTHA of optimized frames achieved by SD, DD and UDD. After that, a post-processing program was developed to perform seismic damage analysis of the optimized frames. The entire process was performed through a link between MATLAB (2010) and OPENSEES (2012) as an object-oriented platform.

2. Seismic damage analysis

Seismic Damage Analysis (SDA) is performed to assess the vulnerability of the structures. In fact, SDA of the structures is the estimation of the damage state by the post-processing of the structural responses using a local and/or global damage index.

2.1 Damage index

The amplitude of the loads by themselves is not sufficient to evaluate the seismic demand on a structure, since the structure’s strength, stiffness, and energy dissipation capacity depend upon the number of load cycles (Khashaee 2004). This understanding is essential for the seismic design of the structures. In the current seismic design approaches for reinforced concrete (RC) structures, the strength and the ductility concepts are used as the design criteria. Besides, the PBSD method considers the parameters based on the force and the deformation as acceptance criteria. Researchers have not been directly considering the effects of hysteretic energy dissipation in PBSD till now. In this context, many researchers and engineers considered the hysteretic energy dissipation as the damage index to compute the degree of damage (Fajfar

1992, Cosenza *et al*. 1993). Therefore, for the seismic performance evaluation and the vulnerability assessment of structures, damage indices play a basic role. Damage can be quantified numerically using damage indices, which are commonly defined as a number between 0.0 and 1.0.

There are several well-known damage indices such as Park-Ang (Park and Ang 1985), Roufaiel-Meyer (Roufaiel and Meyer 1987), and Ghobarah (Ghobarah *et al*. 1999), which have received great attention among researchers. Damage indices are presented as local and global and are given in four forms: non-cumulative, deformation based-cumulative, energy-based cumulative and combined indices. In the maximum deformation based-damage indices, effects of the inelastic excursions are neglected. In effect, a damage index is well-defined if it has a good correlation with the observed damage status after an earthquake (e.g., combined indices).

2.2 Modified Park-Ang damage index

Park and Ang (1985) first proposed a damage index for SDA of RC structures. The Park-Ang damage index is one of the indices widely used for damage assessment. The index is taken into account as a combined index defining the linear combination of the maximum displacement and the hysteretic energy dissipation for a structural element. Since the inelastic behavior of a structure is confined to the plastic zones of some elements, it is difficult to find a relation between elements and stories' deformations using local plastic rotations near the end of elements. Hence, Kunnath *et al.* (1992) modified the Park-Ang damage index based on the deformation of the elements section. The modified damage index is defined as

$$DI_{PA,K} = \frac{\theta_m - \theta_y}{\theta_u - \theta_y} + \frac{\beta}{\theta_u M_y} \int dE_H \tag{1}$$

where θ_m is the maximum rotation during the loading history; θ_u is the ultimate rotation capacity of the section; θ_y is the recoverable rotation when unloading; M_y is the yield moment; and $\int dE_H$ is the hysteretic energy dissipated during the response history; and β is a constant parameter that is computed based on the work reported by Park and Ang (1985).

The two additional indices based on modified Park-Ang damage index, story and overall damage indices (Kunnath *et al.* 1992), are computed using weighting factors based upon hysteretic energy dissipation at components and story levels, respectively

$$DI_{Story} = \sum_{i=1}^{Ne} (DI_i)_{Component} (\lambda_i)_{Component}$$
$$(\lambda_i)_{Component} = \left(\frac{Ei}{\sum Ei} \right)_{Component} \tag{2}$$

$$DI_{Overall} = \sum_{i=1}^{Ne} (DI_i)_{Story} (\lambda_i)_{Story}$$
$$(\lambda_i)_{Story} = \left(\frac{Ei}{\sum Ei} \right)_{Story} \tag{3}$$

where λ_i is the energy weighting factors; and E_i is the total absorbed energy by the *i*th component or story. *Ne* and *Ns* are the number of structural elements (components) and stories. As presented in Table 1, Park *et al*. (1987) calibrated the overall damage index with the observed damage states of nine RC buildings.

3. Design optimization procedure

3.1 Formulation of optimization problem

Generally, an optimization problem can be divided into two groups: constrained and unconstrained problems. Due to the material capacities, deformation restrictions and cost limitations, the optimal design of RC frame structures is a constrained optimization problem. A constrained optimization problem can be expressed as follows

$$\begin{aligned} &Minimize\ \ F(\boldsymbol{x})\ \ Subjected\ to\ \ g_i(\boldsymbol{x}) \le 0.0 \\ &(i = 1, 2, ..., m; x_j \in R^d,\ j = 1, 2, ..., n) \end{aligned} \tag{4}$$

where $F(\boldsymbol{x})$ represents the objective function; $g_i(\boldsymbol{x})$ is the behavioral constraint; m and n are the number of constraints and the design variables, respectively; R^d is a given set of discrete values from which the design variables $\boldsymbol{x}_j$ take values. In the present study, to convert the constrained structural optimization problem into an unconstrained one, an exterior penalty function method was used by constructing a function as follows (Gharehbaghi and Khatibinia 2015)

$$\Phi(x, r_p) = F(x) + r_p \sum_{i=1}^{m} \left(\max\{ (\frac{g_i(\boldsymbol{x})}{\bar{g}_i(\boldsymbol{x})} - 1), 0\} \right) \tag{5}$$

where Φ is the pseudo objective function, and $\bar{g}_i(\boldsymbol{x})$ is the ultimate limit of $g_i(\boldsymbol{x})$ (Vanderplaats 1984). r_p is the positive penalty parameter that was considered to be equal to 25. In the optimization problem, the objective is typically to minimize the structural weight or the construction cost of the structure, under some constraints. In this paper, the construction cost (initial cost) of the structure was considered as the objective function. Moreover, the design variables were the dimensions and reinforcement steel ratio of cross section of structural elements. Herein, the objective

Table 1 Interpretation of structural damage index (Park *et al*. 1987)

Degree	Physical Appearance	Damage Index	Damage State
Collapse	Partial or total collapse of building	>1.0	Loss of building
Severe	Extensive crashing of concrete; disclosure of buckled reinforcement	0.4-1.0	Beyond repair
Moderate	Extensive large cracks; spalling of concrete in weaker elements	0.25-0.4	Repairable
Minor	Minor cracks; partial crushing of concrete in columns	0.1-0.25	Minor damage
Slight	Sporadic occurrence of cracking	<0.1	No damage

function is defined as

$$F = \sum_{i=1}^{Ne} \left(C_C A_{Ci} L_i + C_S A_{Si} L_i + C_F A_{Fi} \right) \quad (6)$$

where C_C and A_{Ci} are the cost per unit volume and the gross cross-sectional area of the *i*th element related to concrete, respectively. C_S and A_{Si} are the cost per unit weight and area of steel bars in the cross section of the *i*th element; C_F and A_{Fi} are also the cost per unit area of form-work and its area in the cross section of the *i*th element; and L_i is the length of the *i*th element. The construction cost chosen as the objective function of the optimization procedure consists of three cost components mentioned.

3.2 Constraints of optimization problem

In the optimization procedure, the design criteria are applied as the problem constraints. In this paper, four sets of constraints were considered as the design criteria. The first set of constraints is related to the practical aspects and initial cross section conditions. The second set is the constraints employed to control the capacity of beams and columns in accordance with the code recommendations for the design of RC structures against the combination of gravity loads, considering the strong-column weak-beam (SCWB) concept. The third set includes the constraints utilized to check the capacity criteria and seismic provisions due to gravity and time-history earthquake loads. Finally, the last set is assigned to measure the degree of damage to elements, stories, and structures.

3.2.1 The first set of constraints

To design the structural elements, these constraints, related to the practical aspects and initial cross section conditions, were considered based on the ACI318 (2011) design code. These constraints are expressed through the following limitations

$$\rho_{b,\min} \le \rho_b \le \rho_{b,\max} \quad (7)$$

$$1\% \le \rho_c \le 4\% \quad (8)$$

$$ds = \frac{b - 2c - 2d_{bt} - N_{bl} d_{bl}}{N_{bl} - 1} \ge ds_{all} \quad (9)$$

$$\{b^t, b^t, n_b^t, A_s^t\} \le \{b^b, b^b, n_b^b, A_s^b\} \quad (10)$$

in which ρ_c and ρ_b represent the reinforcement steel ratio of the cross-sections of columns and beams, respectively; $\rho_{b,\min}$ and $\rho_{b,\max}$ are the minimum and maximum allowable reinforcement steel ratio of the cross-sections of beams, respectively (which are calculated based on ACI318 (2011) design code); and *b*, *c*, d_{bt}, d_{bl} and N_{bl} are the width of the cross-sections, cover, the diameter of the transverse reinforcement, the diameter, and the number of the longitudinal reinforcement, respectively. Also, in Eq. (10), *b*, *h*, n_b and A_s (with superscripts *t* and *b* representing the top and bottom of a story level) are the width, depth, and the number of the longitudinal reinforcement and the total area of the longitudinal reinforcement for beams and columns which are in the same direction between two subsequent stories, respectively. Also, *ds* and ds_{all} are the distances between side-by-side of longitudinal reinforcement and its allowable value, respectively. The value of ds_{all} for columns and beams is defined as follows

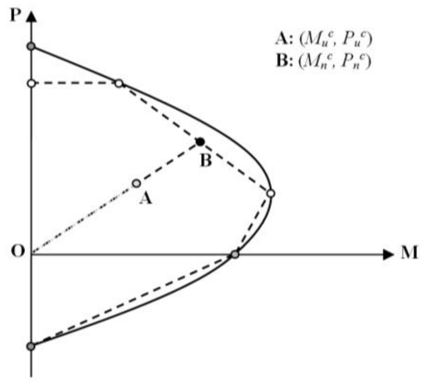

Fig. 1 Original and idealized P-M interaction curve

$$ds_{all} = \begin{cases} \max\{25mm, d_{bl}, 1.33d_{\max}\}_{beams} \\ \max\{40mm, 1.5d_{bl}, 1.33d_{\max}\}_{columns} \end{cases} \quad (11)$$

where $d_{\max}$ is the diameter of the greatest aggregate of the concrete. It is important to note that, in accordance with the ACI318 (2011), the value of the allowable reinforcement steel ratio for cross section of columns is limited to 8%, but the common value of 4% was considered herein.

3.2.2 The second set of constraints

The second set of constraints was used for controlling the capacity of beams and columns against the combination of gravity loads

$$M_u^b \le \phi_b M_n^b \quad (12)$$

in which M_u^b, M_n^b and ϕ_b are the externally factored applied moment due to gravity loads, the nominal flexural strength and the strength reduction factor for beams, respectively.

To check the capacity of columns under gravity loads, the combination of axial load and bending moment applied to the cross section of columns should be investigated. An idealized P-M interaction curve with characterized points shown in Fig. 1 was introduced in the literature. More details on the characterized points can be found in the work of Lee and Ahn (2003). In this paper, the idealized P-M curve was utilized for controlling the columns capacity. Based on this curve, it can be written as

$$\begin{aligned} &\frac{L_{OA}}{L_{OB}} \le 1.0 \Rightarrow \\ &\sqrt{\left(M_u^c\right)^2 + \left(P_u^c\right)^2} \le \sqrt{\phi_c\left(M_n^c\right)^2 + \phi_c\left(P_n^c\right)^2} \end{aligned} \quad (13)$$

where L_{OA} and L_{OB} are the lengths of lines *OA* and *OB*, respectively, illustrated in Fig. 1. Also, M_u^c, M_n^c, P_u^c, P_n^c and ϕ_c are the externally factored applied moment due to gravity loads, the nominal flexural strength, the externally factored applied axial force caused by gravity loads, the nominal axial strength and the strength reduction factor for columns, respectively. The values of ϕ_b and ϕ_c were calculated based on ACI318 (2011).

According to the ACI318 (2011) design code, the SCWB concept should be satisfied particularly in seismically active zones by the following relationship

$$\frac{M_c^t + M_c^b}{M_b^l + M_b^r} \geq 1.2 \tag{14}$$

in which M_c^t and M_c^b are the moment capacity of columns at the top and bottom of a structural joint; also, M_b^l and M_b^r are the moment capacity of beams at the left and right of a structural joint. The requirement should be satisfied for all of the structural joints. Note that the restriction was considered as one of the design constraints of the SD method.

3.2.3 The third set of constraints

The constraints expressed in the forms of Eqs. (15) and (16) should also be considered for the time-history effects of earthquake loads. Since an earthquake load is applied to the structures as time-history, the capacity of beams and columns should be checked for critical conditions. In the case of beam elements, the critical condition is defined as the maximum of an externally applied moment during time-history loads. Also, in the case of columns, the critical condition is assigned to the critical combination of the axial load and the bending moment applied to the section that can be defined as a function depending on the time. Accordingly, Eqs. (12) and (13) can be extended for beams and columns respectively, as follows (Gharehbaghi and Fadaee 2012)

$$\max\left(M_u^b(t)\right) \leq \phi_b M_n^b \tag{15}$$

$$\begin{aligned} &\max\left(\frac{L_{OA}(t)}{L_{OB}}\right) \leq 1.0 \Rightarrow \\ &\max\left(\sqrt{\left(M_u^c(t)\right)^2 + \left(P_u^c(t)\right)^2}\right) \leq \\ &\sqrt{\phi_c\left(M_n^c\right)^2 + \phi_c\left(P_n^c\right)^2} \end{aligned} \tag{16}$$

in which $M_u^c(t)$ and $P_u^c(t)$ are the externally factored applied moment and axial force at the time of t of earthquake loads. Note that the critical conditions related to the columns should not be considered by the combination of the maximum of bending moment and the maximum of axial loads during an earthquake simultaneously (Gharehbaghi and Fadaee 2012). In this paper, it was assumed that the shear capacity of the structural elements satisfies the code requirements.

One of the most important design constraints under seismic loading is the inter-story drift ratio (ISDR). The permissible ratio of the limitation is different depending on the characteristics of structure. In this paper, according to the recommendations of the 2800-code (2004), permissible values related to the constraint were considered as follows

$$ISDR \leq \begin{cases} 0.025 \; for \; T < 0.7\sec \\ 0.020 \; for \; T \geq 0.7\sec \end{cases} \tag{17}$$

where T represents the vibration period of structure obtained from a free vibration analysis.

3.2.4 The fourth set of constraints

The degree of damage should be considered based on the performance expectations and hazard levels. From an engineering point of view, in proportion to moderate earthquakes, the allowable overall damage index was selected in accordance with the maximum acceptable damage of 0.25 (minor level of damage based on modified Park-Ang damage index) to maintain the structure's serviceability with only minor repairs. Based on theory of plastic mechanism control aiming at reaching a collapse mechanism of global type, damage should occur at the beam ends as dissipative zones, while all the columns should remain in the elastic range except for base section of first storey ones (Chung *et al*. 1993, Montuori *et al*. 2015, Montuori and Muscati 2015, 2016). Hence, a damage pattern considering the different degrees of damage is proposed for beams and columns. These constraints may be encapsulated as

$$\begin{aligned} &\left\{DI_{cb}, DI_{cs}, DI_b, DI_{Overall}\right\} \leq \\ &\left\{0.1, 0.05, 0.25, 0.25\right\} \end{aligned} \tag{18}$$

in which DI_{cb} is the damage index of the bottom ends of columns at the first story, DI_{cs} is the damage index at the top ends of columns at the first story and at each ends of the columns at other stories. DI_b and DI_o are the damage indices of beams and the overall structural damage index, respectively. Also, to prevent the damage concentration during severe earthquakes, the distribution pattern of the story damage index (SDI) was considered as another constraint (only for UDD) to achieve the uniform distribution of damage over the height of the structure. Thus, it is desirable to use a reasonable difference among SDI of two stories to be less than a user-defined value as

$$\begin{aligned} &SDI_i - SDI_j = \tau \leq \tau_{all} \\ &(i, j = 1, 2, ..., Ns) \end{aligned} \tag{19}$$

where SDI_i and SDI_j are the damage indices at the ith and jth story. τ_{all} is a user-defined value for the uniform distribution of SDI. Based on the degrees of damage listed in Table 1 and the time of implementing the optimization process as well as the experience of the author gained from investigating the results of seismic damage analysis of several RC frames, in this paper, τ_{all} was considered to be equal to a reasonable value of 0.14. The effectiveness of the value is studied in the section of results. It should be noted that, depending on the decision of designer and the time of implementing the optimization process, τ_{all} could be considered less than the mentioned allowable value.

3.3 Optimization algorithm

The binary model of the PSO algorithm inspired from the social behavior of such animals as fishes, insects and birds, was proposed by Kennedy and Eberhart (2001). It involves a number of particles initialized randomly in the feasible search space of the problem domain while these particles are referred to as swarm representing a potential solution of the optimization problem. Salajegheh *et al*. (2008) reported a successful application of the binary PSO algorithm in the design optimization of structures under time-history earthquake loads. In the current study, an improved version of the PSO algorithm (i.e., the real valued model of PSO) was employed to implement the optimization procedure. The successful applications of this version of PSO were reported in works (Gholizadeh and Salajegheh 2009, Gharehbaghi *et al*. 2011, Gharehbaghi and Salajegheh 2011, Gharehbaghi *et al*. 2012, Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015, Gharehbaghi *et al*. 2016). In this model, the decimal values of the design variables are utilized in the optimization process instead of their binary codes. In this case, the length of the particles is shortened, and so the convergence of the algorithm could be achieved with a lower effort and a higher speed.

4. Optimum seismic design methodologies (SD, DD, and UDD)

In this study, three optimum seismic design methods are presented and assessed. The first one is the strength-based optimum seismic design method (SD) which, in turn, is a code-based design method. The second and third methods are the damage- and uniform damage-based optimum seismic design methods (DD and UDD), respectively. The last two methods are taken into account as the PBSD methods. The next two subsections describe the SD, DD and UDD methods in detail.

4.1 SD method

The seismic design of structures based on the single parameter of strength is currently used by most of structural engineers/designers across the world. In this paper, the optimum version of the method, i.e., SD, which is mostly used in the literature of design optimization field (e.g., Gharehbaghi and Fadaee 2012, Gharehbaghi and Khatibinia 2015), is presented to be assessed then. In this method, the construction cost was regarded as the objective function. The cross-sectional dimensions and steel reinforcement ratio were considered as the design variables of the defined optimization problem. In addition, the allowable cross section condition, practical aspects, capacity and ISDR were taken into account for the design optimization constraints.

The overall steps of SD are mentioned herein. The SD is started by optimizer while the arbitrary number of frames are randomly designed using the cross sections existing in the related pre-defined section database. The cross sections used herein conform to the ACI318 (2011) code requirements which are mathematically expressed using Eqs. (7)-(11). Then the frames are modeled using the finite element analysis software OPENSEES (2012). After that, considering the combinations of gravity loads, a static analysis is carried out for all the randomly designed frames. In the next step, using the static responses, all cross sections of beams and columns are controlled using the Eqs. (12)-(14). The rejected frames are penalized, and the passed ones from the previous step are subjected to a simultaneous action of gravity and seismic loading, and linear time-history analysis (LTHA) is carried out then. Using the recorded dynamic responses, all cross sections of beams and columns are controlled again using the Eqs. (15)-(17) and the rejected frames are penalized. Then, the pseudo-objective function of all the passed and rejected frames is computed, and the best design candidate of the iteration (having minimum value of pseudo-objective function) is selected. Considering the best solution of previous iteration, PSO algorithm intelligently updates the design variables of each group of the beams and columns. The previous steps from modeling the frames to the selection of best solution are iterated until the optimum design candidate is reached. The load combinations used in SD is expressed in the next section. The entire process of SD has been shown in the right side of Fig. 2.

4.2 DD and UDD methods

The seismic resistant design of structures is a trade-off between minimizing the construction cost and maximizing the structural seismic safety. This paper aiming at proposing a new and innovative methodology based on damage control and uniform distribution of damage over the height of the structure, i.e., UDD, to reach an optimum seismic design candidate having minimum construction cost and least structural damage. The idea of uniform damage distribution was utilized to prevent damage concentration and formation of an undesired collapse mechanism particularly during severe earthquakes. The damage concentration occurs due to lack of optimum distribution of force, deformation, and energy demands and capacities of the structural elements over the height of the structure; and it can be eliminated by making the optimum use of materials. For this purpose, the controlling criteria of seismic design based on current design codes were modified. Considering the concept of desired collapse mechanism of global type (Montuori *et al*. 2015, Montuori and Muscati 2015, 2016), the structural damage and its distribution among the stories were considered as other acceptance criteria of seismic design, in addition to the code specifications. These criteria were taken into account as the constraints of the optimization problem. Since this aim requires defining allowable degree of damage, a damage pattern, extracted from the concept of desired collapse mechanism of global type (Montuori *et al*. 2015, Montuori and Muscati 2015, 2016), were considered (using Eqs. (18) and (19)).

The overall steps of UDD are mentioned herein. The UDD is started by optimizer whilst the arbitrary number of frames are randomly designed using the cross sections existing in the related pre-defined database. The used cross sections comply with the ACI318 (2011) code requirements which are mathematically described using Eqs. (7)-(11). Next, the frames are modeled using the finite element analysis software OPENSEES (2012). Then, considering the combinations of gravity loads, a static analysis is carried out for all the randomly designed frames. In the next step, using the static responses, the capacity of all cross sections of beams and columns is controlled using the Eqs. (12) and (13). The rejected frames are penalized, and the passed ones from the previous step are subjected to a simultaneous action of gravity and seismic loading, and NLTHA is

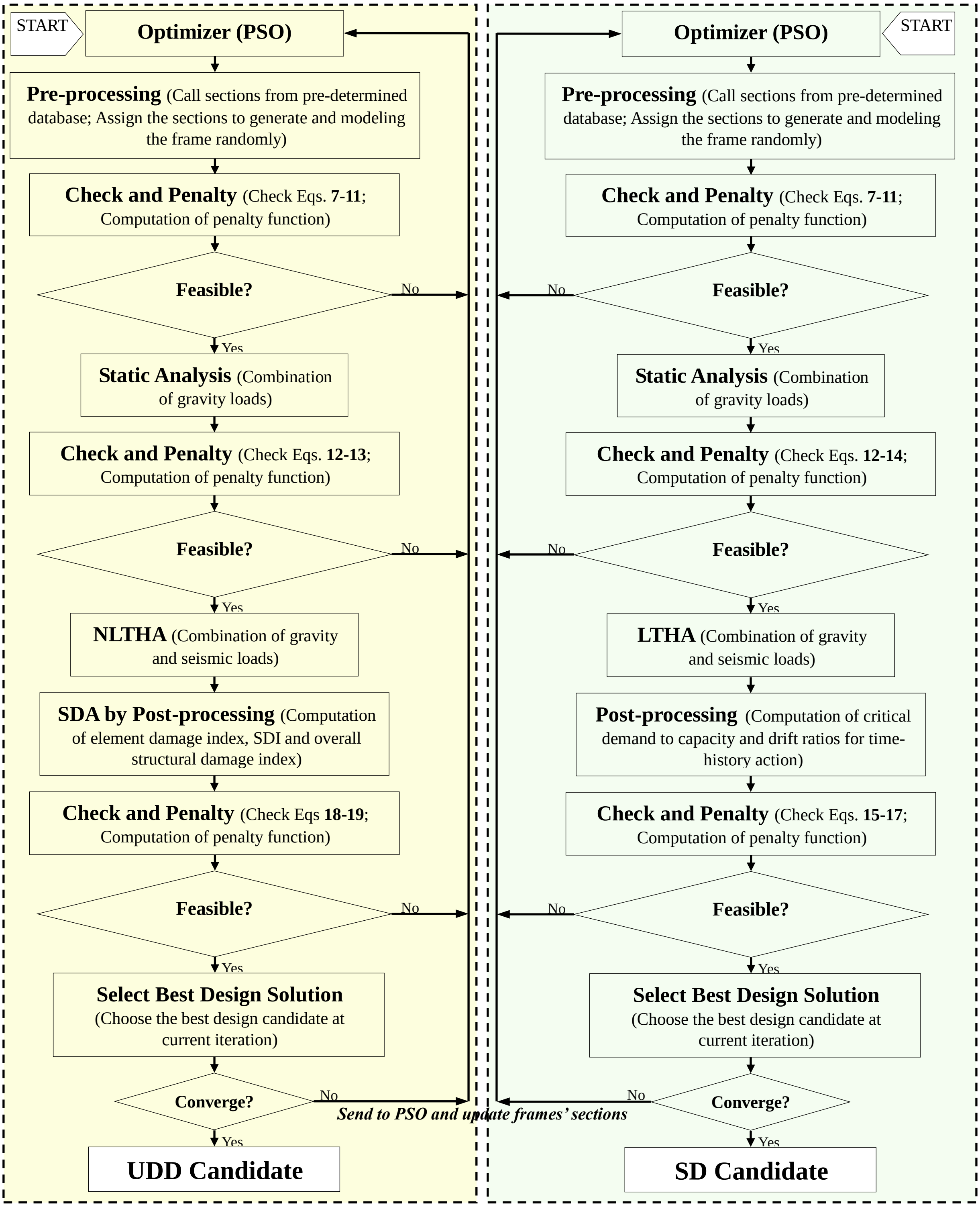

Fig. 2 Flowcharts of UDD (left) and SD (right) procedures

carried out then. By using post-processing on the recorded nonlinear dynamic responses, SDA is conducted to derive the damage degree of elements, stories and overall structure. After that, the damage index of beams and columns, overall damage index, and distribution of damage over the height of the frames are controlled using the Eqs. (18) and (19). The pseudo-objective function of all the passed frames and rejected ones are computed, and the best design candidate of the iteration (having minimum value of pseudo-objective function) is chosen. Considering the best solution of previous iteration, PSO algorithm intelligently updates the design variables of each group of the beams and columns. The previous steps from modeling the frames to the selection of best solution are iterated until the optimum design candidate is reached. Therefore, using the idea, the optimum use of material could be reached based on the

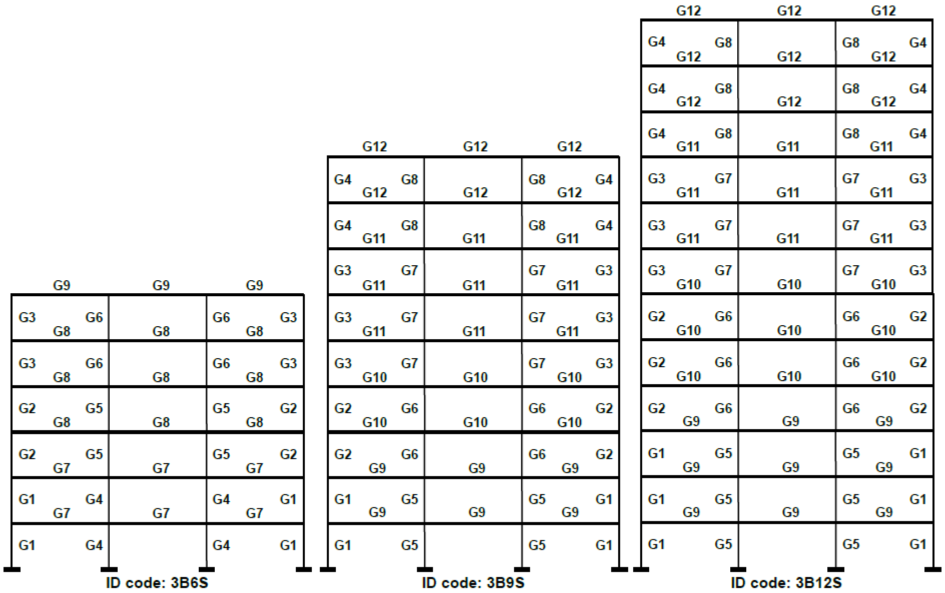

Fig. 3 Geometry, elements classification and identification code of illustrated RC frames

designer's objective. The load combinations used in UDD is described in the next section. The entire process of UDD has been shown in the left side of Fig. 2.

In order to highlight and to show the effects of uniform damage distribution on the optimum design candidate achieved by the UDD, another optimum seismic design methodology, i.e., damage-based optimum seismic design (DD) was adopted. All the steps and details of the DD is exactly similar to those of the UDD except for the last design constraint on the uniform distribution of damage over the height of the frames (mathematically expressed using Eq. (19)), which was not considered in the DD.

5. Worked examples

5.1 Frame geometry and section database

To demonstrate the efficiency of the proposed methodology, i.e., UDD, three six-, nine- and twelve-story RC frames with three bays were considered. As shown in Fig. 3, the beams and columns were separately classified using group numbers, from G1 to G9 for six- and from G1 to G12 for nine- and twelve-story frames, respectively. The length of the beams and the height of the columns in each of the bays and stories are constant and were set to be equal to 5.0 m and 3.3 m, respectively.

In the case of a pre-defined section database, two databases for beams and columns were generated in which all of the essential section properties such as dimensions and steel reinforcements were located. These databases include several rectangular sections conforming to the ACI318 (2011) design code requirements. In the case of beam sections, the height of sections were chosen in the range of 400 to 900 mm associated with two widths of 350 and 400 mm; and regarding the column sections, it was assumed that the width and height of sections are the same in the range of 400 and 900 mm. In addition, in the prepared database, 50 mm was considered as the difference between the dimensions of sequential sections. The diameter of longitudinal bars was between 12 and 28 mm in the database, by the step of 2 mm. The minimum concrete cover was also assumed to be equal to 40 mm. It was also assumed that the transverse bars with 10 mm diameter are used for the shear control of the sections and the space between these bars is designed in a way that the sections have the intermediate ductility. Based on an engineering judgment, it is not expected that a section more than 700-mm height be used for cross section dimension of beams and columns of six-story frame during the design optimization process. So, to reduce the length of search space and increase the convergence rate of the optimization process, the range of the heights of sections from 750 to 900 mm was not considered for the six-story frame. The cost units of the construction components were also considered based on the work presented by Gharehbaghi and Fadaee (2012) to be equal to C_C=60\$/m^3, C_S=7065\$/m^3, and C_F=18 \$/m^2 respectively.

5.2 Structural modeling, loadings and analyses

All of the static and dynamic analyses of illustrated RC frames during the optimization process were carried out using the finite element analysis software OpenSees (2012). For modeling and analyzing the structures in OpenSees (2012), properties of materials and types of elements should be defined. In this paper, the *uniaxialmaterial Concrete01* was employed for concrete behavior modeling. This material is based on the modified Kent-Park concrete model (Kent and Park 1997). The behavior of reinforcement bars was modeled by *uniaxialmaterial Steel01*. The material considers a bilinear model for the behavior of steel bars. The constitutive parameters of the concrete are f_c, f_{cu}, ε_o and ε_{cu} which are the concrete compressive strength, residual strength, strain at the peak strength and strain corresponding to the residual

Table 2 The considered properties of concrete and steel reinforcement materials

Components	f_c (MPa)	f_{cu} (MPa)	ε_o	ε_{cu}	E_c (MPa)	γ_s (N/mm^3)
Concrete Core	29.85	6.36	0.0021	0.0122	24870	2.45e-5
Concrete Cover	28.00	5.97	0.0020	0.0035	24870	2.45e-5
Components	f_y (MPa)	f_{yt} (MPa)	α_s	ε_{su}	E_s (MPa)	γ_s (N/mm^3)
Steel Reinforcement	400	280	0.01	0.10	210000	7.7e-5

strength. The related parameters of the steel bars are f_y, α_s and ε_{su} that are the yield stress, the hardening ratio and the ultimate strain of longitudinal bars, respectively. These models have been described in more details in the References (Kent and Park 1997, OPENSEES 2012). The moderate confinement ratio was considered for each RC section (Kunnath *et al*. 1992). As shown in Table 2, γ_c, γ_s, E_c and E_s are the weight per unit volume of the concrete material, the weight per unit volume of the steel material, the modulus of elasticity of the concrete material and the modulus of elasticity of the steel material, respectively. Also, f_{yt} is the yield stress of the transverse bars. The assigned values to these defined parameters have been listed in Table 2. Beams and columns were modeled using force-based *non-linear beam-column element* considering the spread plasticity along the length of the elements. A fiber section model was used at each integration point of the elements.

In the case of loadings, the required load combinations provided by ACI318 (2011), FEMA (2000) guideline, and 2800-codes (2004) should be considered. As regards to the gravity and earthquake loads, five load combinations were considered

$$\begin{aligned} &comb1: 1.4D \\ &comb2: 1.2D + 1.6L \end{aligned} \tag{20}$$

$$\begin{aligned} &comb3: 0.9D \pm E \\ &comb4: 1.2D + 0.5L \pm E \end{aligned} \tag{21}$$

$$comb5: 1.1(D + \mu L) \pm E \tag{22}$$

where D and L are the uniform dead and live loads on the beams, respectively, and μ is the effective ratio of live loads. In this study, the values of the dead and live loads were considered as 5.884 kN/m^2 and 1.961 kN/m^2 for stories, 6.374 kN/m^2 and 1.471 kN/m^2 for roof level; by considering an inter-frame of 5.0 m distance, these distributed loads were obtained 29.42, 9.805, 31.87, 7.355 kN/m, respectively. Also, the value of μ was selected as 0.20 (2800-code 2004). A single-artificial ground motion record matched to the Iranian response spectrum with soil type III was used as the earthquake load (E) mentioned in Eqs. (21) and (22). Depending on the aforementioned three design methods, the following load combinations were used: the load combinations of *comb1* to *comb4* were employed in the SD method; and the load combinations of *comb1*, *comb2* and *comb5* were used for both DD and UDD methods. Note that, because of utilizing NLTHA in both the DD and UDD methods, the load combination of *comb5* recommended by FEMA guideline (FEMA, 2000) was used for these methods.

It was assumed that the frames locate in a region with relatively high seismic hazard level. In this regard, by using SeismoArtif program (SeismoSoft programs 2013), an application capable of generating artificial earthquake records matched to a specific target response spectrum using different calculation methods and varied assumptions, an artificial earthquake record based on Housner-Trapezoidal envelope function (available in the SeismoArtif program) was generated and matched to the Iranian elastic response spectrum. The envelope parameters were selected such that the total duration is about 20 s, and the rise and level time were selected to be 5 s and 15 s, respectively. The PGA of 0.3 g was used in accordance with the hazard levels classified by 2800-code (corresponding to the probability of occurrence 10% in 50 years) (2004). Note that, after generating the matched artificial record, the used PGA is multiplied by all acceleration values of the record. The code proposes an R-factor equal to 7.0 for RC frames with intermediate ductility (considering a ratio of 1.4 for load and resistant factor design method). The Newmark-β method was employed in the numerical integration of the equations of motion. The Rayleigh damping with 5% damping ratio and the P-Δ effects were considered in the analyses. In addition, the effects of shear and bond-slip in reinforcement bars at the end of elements were ignored.

5.3 Optimum results and discussion

After the implementation of the optimization procedure based on the SD, DD and UDD using a link between the PSO code in MATLAB (2010) and the structural model in OPENSEES (2012), three optimum results were achieved for each illustrated RC frame. The optimum properties of the frames such as width-height (called "$b\times h$") and total steel ratio (SR) of cross section of classified elements, have been separately listed in Tables 3-5. Concerning the 3B-6S, 3B-9S and 3B-12S frames, variations of both $b\times h$ and SR over the groups of elements and the height of optimized frames corresponding to SD, DD and UDD has not a justifiable

Table 3 $b\times h$ and SR of the optimized 3B-6S using SD, DD and UDD

Element Type	Groups Number	SD		DD		UDD	
		$b\times h$ (mm)	SR (%)	$b\times h$	SR (%)	$b\times h$	SR (%)
Columns	G1	650×650	1.20	450×450	1.52	600×600	1.12
	G2	450×450	1.99	400×400	1.92	450×450	1.52
	G3	450×450	1.52	400×400	1.92	450×450	1.52
	G4	650×650	1.49	600×600	2.51	600×600	1.75
	G5	550×550	1.68	550×550	2.51	550×550	2.08
	G6	450×450	2.51	500×500	2.04	400×400	2.51
Beams	G7	350×600	1.09	350×500	1.44	350×600	0.77
	G8	350×400	1.29	350×500	1.44	350×550	0.84
	G9	350×400	1.29	350×400	1.15	350×400	1.15
Total Construction Cost ($)		11850		11679		11430	

Table 4 *b*×*h* and SR of the optimized 3B-9S using SD, DD and UDD

Element Type	Groups Number	SD *b*×*h* (mm)	SD SR (%)	DD *b*×*h*	DD SR (%)	UDD *b*×*h*	UDD SR (%)
Columns	G1	700×700	1.55	600×600	2.11	650×650	1.80
	G2	650×650	1.20	600×600	2.11	650×650	1.80
	G3	550×550	1.33	550×550	1.68	650×650	1.80
	G4	500×500	1.61	450×450	2.51	450×450	1.99
	G5	700×700	1.55	700×700	1.85	600×600	2.12
	G6	650×650	1.80	600×600	1.41	550×550	1.02
	G7	500×500	2.04	550×550	1.68	550×550	1.02
	G8	500×500	2.04	550×550	1.33	400×400	1.41
Beams	G9	400×600	1.18	400×700	1.22	400×500	1.41
	G10	400×400	1.77	400×450	1.27	400×500	1.41
	G11	400×400	1.77	400×450	1.27	400×450	1.27
	G12	400×400	1.77	400×400	1.43	400×450	1.27
Total Construction Cost ($)		21236		21412		20622	

Table 5 *b*×*h* and SR of the optimized 3B-12S using SD, DD and UDD

Element Type	Groups Number	SD *b*×*h* (mm)	SD SR (%)	DD *b*×*h*	DD SR (%)	UDD *b*×*h*	UDD SR (%)
Columns	G1	750×750	1.12	700×700	1.85	800×800	1.41
	G2	750×750	1.12	700×700	1.04	650×650	2.14
	G3	600×600	1.41	700×700	1.04	600×600	1.12
	G4	500×500	1.61	500×500	2.04	400×400	2.51
	G5	750×750	1.35	800×800	1.19	650×650	1.20
	G6	650×650	1.49	600×600	1.75	650×650	1.20
	G7	650×650	1.20	550×550	1.02	650×650	1.20
	G8	500×500	2.04	550×550	1.02	650×650	1.20
Beams	G9	400×700	1.09	400×550	1.38	400×600	1.27
	G10	400×500	1.52	400×550	1.38	400×600	1.27
	G11	400×400	1.90	400×500	1.26	400×550	0.92
	G12	400×400	1.27	400×500	1.26	400×400	1.27
Total Construction Cost ($)		30377		30313		30293	

rule. In fact, in some groups of elements, *b*×*h* increased while in other ones the value has decreased; and the same is true about SR. Generally, this comparison is of great importance in terms of the strength and stiffness distribution over the height of the structures and the construction cost.

One of the most important parameters related to the design of structures under earthquake ground motion is the construction cost. Since this parameter was chosen as an objective function for the three given seismic design optimization methods, the construction cost of the optimum design is investigated herein. Regarding the 3B-6S frame, as shown in Table 3, the construction cost for UDD is equal to $11430 which is about 2% and 4% less than that of the DD and SD, respectively. According to Table 4, for the 3B-9S frame, the construction cost of UDD is equal to $20622 which is about 4% and 3% less than that of the DD and SD, respectively. Also, concerning the construction cost of the

Table 6 The seismic results of optimized 3B-6S using SD, DD and UDD (PGA=0.3 g)

Story Level	SD ISDR	SD SDI	DD ISDR	DD SDI	UDD ISDR	UDD SDI
1st	0.0029	0.0033	0.0075	0.0891	0.0052	0.0616
2nd	0.0049	0.0179	0.0131	0.1955	0.0089	0.1439
3rd	0.0134	0.2569	0.0140	0.1394	0.0110	0.1566
4th	0.0199	0.3786	0.0110	0.0635	0.0116	0.1480
5th	0.0191	0.1470	0.0110	0.0812	0.0148	0.1213
6th	0.0104	0.0205	0.0085	0.0167	0.0103	0.0216
Minimum (Maximum) SCWB Ratio	1.35 (4.37)		1.55 (3.72)		1.06 (4.67)	
Overall Damage Index	0.2412		0.1256		0.1240	

Table 7 The seismic results of optimized 3B-9S using SD, DD and UDD (PGA=0.3 g)

Story Level	SD ISDR	SD SDI	DD ISDR	DD SDI	UDD ISDR	UDD SDI
1st	0.0035	0.0074	0.0024	0.0019	0.0061	0.0414
2nd	0.0074	0.0572	0.0041	0.0054	0.0116	0.1125
3rd	0.0119	0.0879	0.0086	0.0648	0.0138	0.1160
4th	0.0149	0.1363	0.0116	0.1351	0.0130	0.0910
5th	0.0180	0.1859	0.0157	0.2053	0.0124	0.1210
6th	0.0187	0.1563	0.0168	0.1712	0.0138	0.1247
7th	0.0153	0.0533	0.0139	0.0773	0.0135	0.0778
8th	0.0099	0.0094	0.0102	0.0133	0.0136	0.0296
9th	0.0056	0.0100	0.0060	0.0150	0.0068	0.0117
Minimum (Maximum) SCWB Ratio	1.22 (7.15)		1.31 (8.58)		0.45 (10.20)	
Overall Damage Index	0.1184		0.1272		0.0974	

3B-12S frame as depicted in Table 5, the value for UDD is equal to $30293 which is less than the other methods (by less than 1%).

From an engineering point of view, a structure with less construction cost is accepted if it has an efficient performance with regard to the other design candidates. To investigate the seismic performance of the optimized frames designed by SD, DD and UDD versus the construction cost of them, the frames were evaluated by subjecting them to a single-artificial record (considered as the seismic design load) through the NLTHA and SDA. The inelastic results related to the SD, DD and UDD particularly including the inter-story drift ratio (ISDR), story damage index (SDI) and the overall damage index, have separately been listed in Tables 6-8. As shown in these Tables, in terms of the structural damage, the SDA-based results indicate that the overall damage index of the optimized 3B-6S and 3B-9S frames based on the UDD is less than that of the SD and DD. For 3B-12S, the overall damage index for DD is less than that of the UDD and for UDD is less than that of the SD. In fact, for the PGA level of 0.3 g, all optimized frames based on the three methods are safe against the excitation (except for the 3B-6S frame designed by SD in terms of local damage), and have experienced a minor level of overall damage based on Table 1. The SDI of the optimally designed frames by the SD and DD show that some stories

Table 8 The seismic results of the optimized 3B-12S using SD, DD and UDD (PGA=0.3 g)

Story Level	SD		DD		UDD	
	ISDR	SDI	ISDR	SDI	ISDR	SDI
1st	0.0026	0.0030	0.0037	0.0081	0.0047	0.0332
2nd	0.0045	0.0111	0.0073	0.0607	0.0094	0.1006
3rd	0.0052	0.0226	0.0090	0.0968	0.0113	0.1350
4th	0.0074	0.0375	0.0102	0.0957	0.0123	0.1414
5th	0.0090	0.0704	0.0108	0.1054	0.0121	0.1163
6th	0.0101	0.1083	0.0121	0.1230	0.0113	0.1024
7th	0.0147	0.1715	0.0148	0.1724	0.0120	0.1448
8th	0.0194	0.2539	0.0153	0.1261	0.0121	0.1176
9th	0.0214	0.2617	0.0129	0.0559	0.0105	0.0832
10th	0.0201	0.1750	0.0097	0.0080	0.0111	0.0671
11th	0.0130	0.0236	0.0064	0.0058	0.0104	0.0252
12th	0.0063	0.0155	0.0037	0.0048	0.0071	0.0182
Minimum (Maximum) SCWB Ratio	1.49 (6.54)		0.67 (6.20)		0.17 (7.29)	
Overall Damage Index	0.1494		0.0966		0.1023	

have suffered more damage compared with those from the UDD, while for 3B-6S and 3B-12S designed by SD, the maximum SDI exceeded 0.25. Also as presented in these Tables, regarding the optimized frames by SD, the damage concentration can be observed and it may increase with increasing PGA. Based on Eq. (19), the maximum difference between the SDI (maximum τ) of optimized frames by SD is about 0.36, 0.25 and 0.18 for the 3B-6S, 3B-9S and 3B-12S, respectively. The maximum τ is equal to 0.18, 0.2 and 0.18 for optimized frames using DD that are more than 0.14. Finally, in relation to the three optimally designed frames by UDD, the uniform damage distribution has been achieved, while the maximum τ is about 0.12, 0.12 and 0.14 for 3B-6S, 3B-9S and 3B-12S, respectively. Hence, it can be stated that the damage distribution pattern of optimized frames by UDD is more uniform than those of by DD.

6. Seismic evaluation of optimized frames

The seismic performance of all optimally designed frames based on SD, DD and UDD methods were also investigated under a suite of code compatible earthquake ground motions. Ten real earthquake ground motions listed in Table 9 were selected from the PEER ground motion database (2012). Since the optimized frames were designed optimally under a single-artificial ground motion record matched to the Iranian elastic response spectrum with soil type III, the records were chosen based upon a soil type corresponding to the United State Geological Survey (USGS) (2012) with the site class C. Then, using SeismoMatch (Seismosoft programs 2013), an application capable of adjusting earthquake records to match a specific target response spectrum using the wavelets algorithm, the chosen real earthquake ground motion records were matched to the Iranian elastic response spectrum (with soil type III). Fig. 4 shows the target response spectrum and

Table 9 Details of original strong motion records

Earthquake Name-deg °	Date	Station *	Distance **	PGA (g)	*Mw*
Cape Mendocino-090	04.25.92	Petrolia	09.50	0.66	7.1
Imperial Valley-230	10.15.79	El Centro Array #11	12.60	0.38	6.5
Kocaeli-270	08.17.99	Duzce	12.70	0.35	7.4
Landers-095	06.28.92	El Monte-F.A.	136.1	0.04	7.3
Loma Prieta-090	10.18.89	Capitola	14.50	0.44	6.9
Morgan Hill-000	04.24.84	Gilroy Array #3	14.60	0.19	6.2
Northridge-270	01.17.94	Canyon Country-W.L.C	13.00	0.48	6.7
Parkfield-065	01.17.94	Cholame #2	00.10	0.47	6.1
Superstitn Hills-090	11.24.87	El Centro Imp.	13.90	0.26	6.7
Whittier Narrows-090	10.01.87	Arcadia-Cam.	12.20	0.30	6.0

* USGS class (C), ** Closest to fault rupture (km)

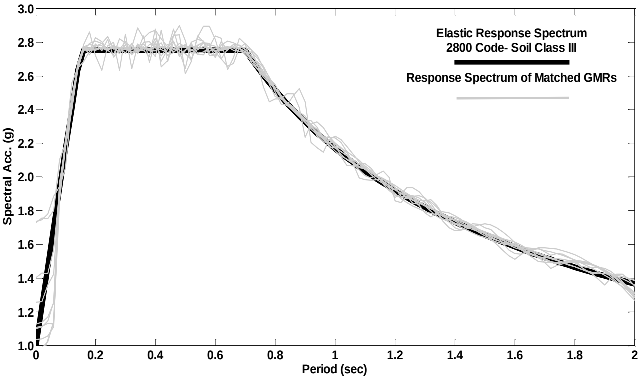

Fig. 4 Elastic response spectrum of 2800-code corresponding to soil class III and response spectrums of matched earthquake ground motion records

spectrums of matched earthquake ground motion records. Two levels of PGA, 0.3 g (probability of occurrence 10% in 50 years corresponding to the design base earthquake level), and 0.45 g (probability of occurrence 2% in 50 years with 1.5 times of 0.3 g corresponding to the maximum considered earthquake level (ASCE/SEI 7-10 2010)) were considered for NLTHA and SDA of the optimized frames. A PGA of 0.6 g was also selected to compare the design methods by assessing the seismic performance of the optimized frames when the more inelastic excursions with higher amplitude are occurred.

Figs. 5-7 show the average SDI and the average ISDR based on these ten matched records for the three levels of PGA. These figures have been separated based on the considered seismic demand parameters, i.e., SDI and ISDR. Based on the recommendations of FEMA guideline (FEMA, 2000) on the RC framed structures analyzed using time-history method, the maximum allowable ISDRs of 2% and 4% should be considered for the performance objectives of Life Safety (LS) and Collapse Prevention (CP), respectively. As depicted in Figs. 5-7, regarding the PGA of 0.3 g (i.e., design base earthquakes level) in the case of the 3B-6S frame designed by SD, the SDI has exceeded allowable value and damage has been concentrated in the two mid-stories while the ISDR is in the permissible range. The results for the other optimized frames by the three

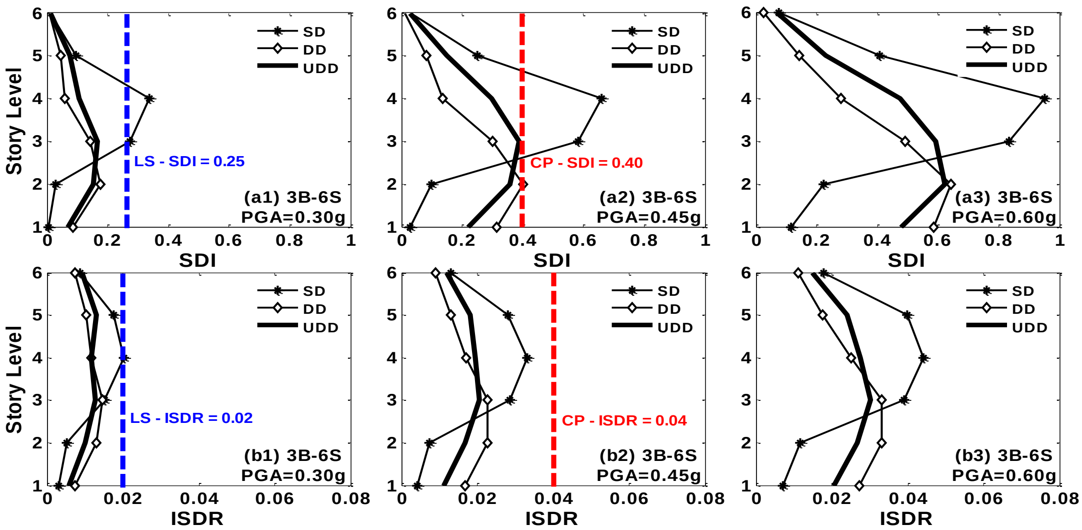

Fig. 5 The average SDI (a1-a3) and ISDR (b1-b3) for optimized 3B-6S

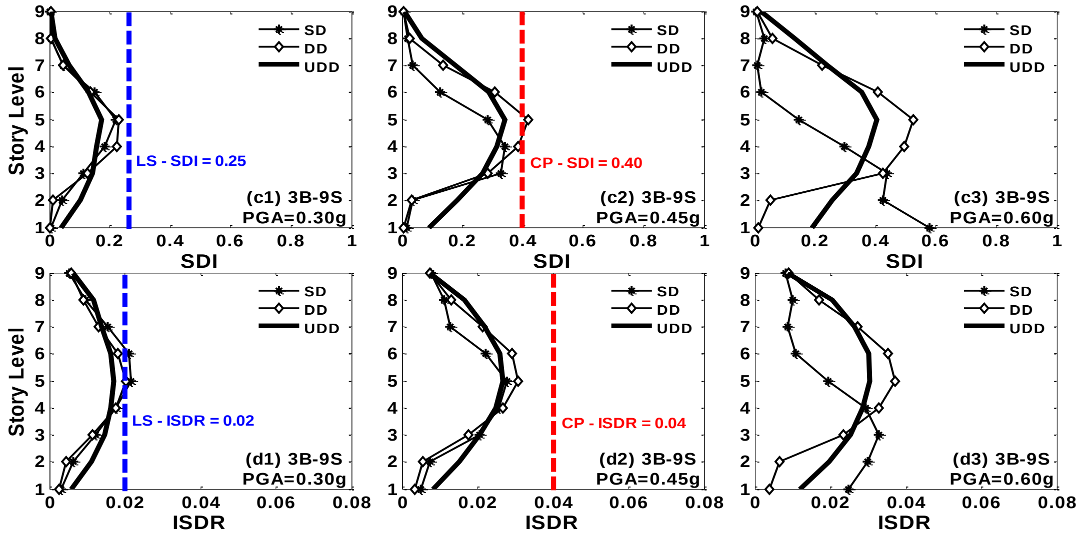

Fig. 6 The average SDI (c1-c3) and ISDR (d1-d3) for optimized 3B-9S

methods in terms of either SDI or ISDR are in about the permissible ranges. So, generally speaking, the DD, UDD and almost even the SD methods (except for the 3B-6S frame) can lead to obtain a design candidate that is feasible under design base earthquakes level.

In relation to the PGA of 0.45 g, it is obvious that for SD method, SDI (and in some cases ISDR) has been concentrated and exceeded the allowable limits into one or a few stories, while the damage has been distributed uniformly for the case of UDD. The demand parameters for DD vary in a range between those of the SD and UDD. Concerning the maximum SDI of 3B-6S, it has suffered 42% and 39% less degree of damage for UDD and DD, compared with the SD, respectively. For 3B-12S, these ratios are 30% and 19%. Furthermore, for the 3B-9S frame, in terms of the maximum SDI and ISDR, the DD has an undesirable condition compared with the SD and UDD, and in terms of SDI and ISDR distribution, the UDD has a better condition with respect to the others. It should be noted that, the maximum SDI of the some optimized frames using SD and DD is more than 0.4 (severe damage) while their corresponding ISDR is less than 0.04. Hence, it can be stated that, in some cases, the designed structures based on the demand parameter of ISDR are rejected in terms of the SDI.

The seismic performance of the frames due to a PGA of 0.6 g shows that the SD and DD based design frames have much higher SDI and ISDR compared to those of the UDD. For example, in relation to the maximum SDI, 3B-6S, 3B-9S and 3B-12S frames designed by SD have respectively suffered nearly 37%, 31% and 21% more damage compared with those of the UDD (30% on average of three frames). These ratios are roughly 32%, 10% and 36% about maximum ISDR, which are very different for 3B-9S and 3B-12S. As shown in Fig. 9-c3, one of the significant differences between SDI and ISDR is related to the first storey of 3B-9S optimized frame using SD. As depicted in this figure, by increasing PGA from 0.45 g to 0.6 g, damage is concentrated in the first storey of the frame whilst the problem cannot be concluded based on its corresponding ISDR. The ISDR, that is almost equal to the average chord rotation of columns of a story of interest, is a deformation-

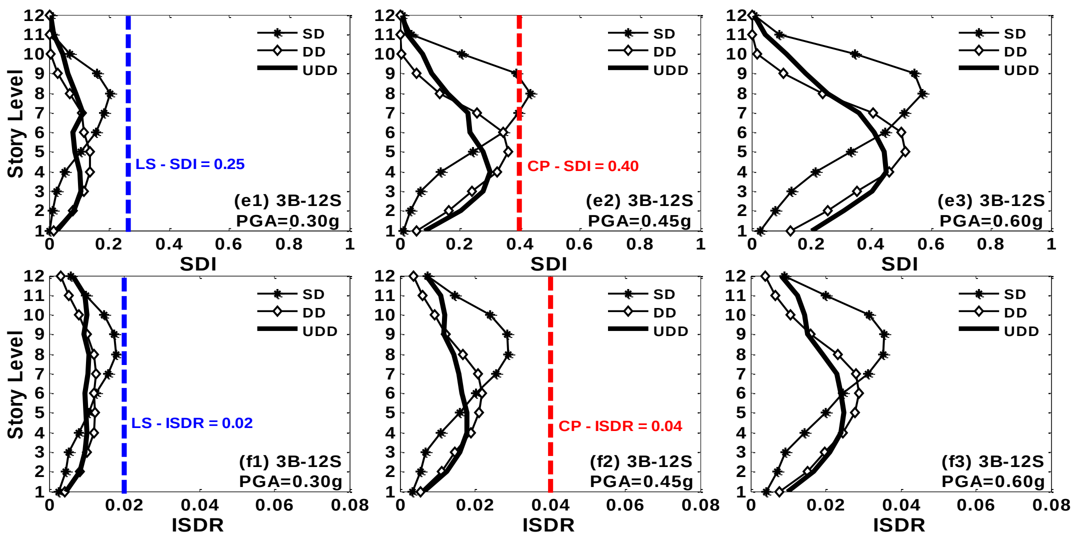

Fig. 7 The average SDI (e1-e3) and ISDR (f1-f3) for optimized 3B-12S

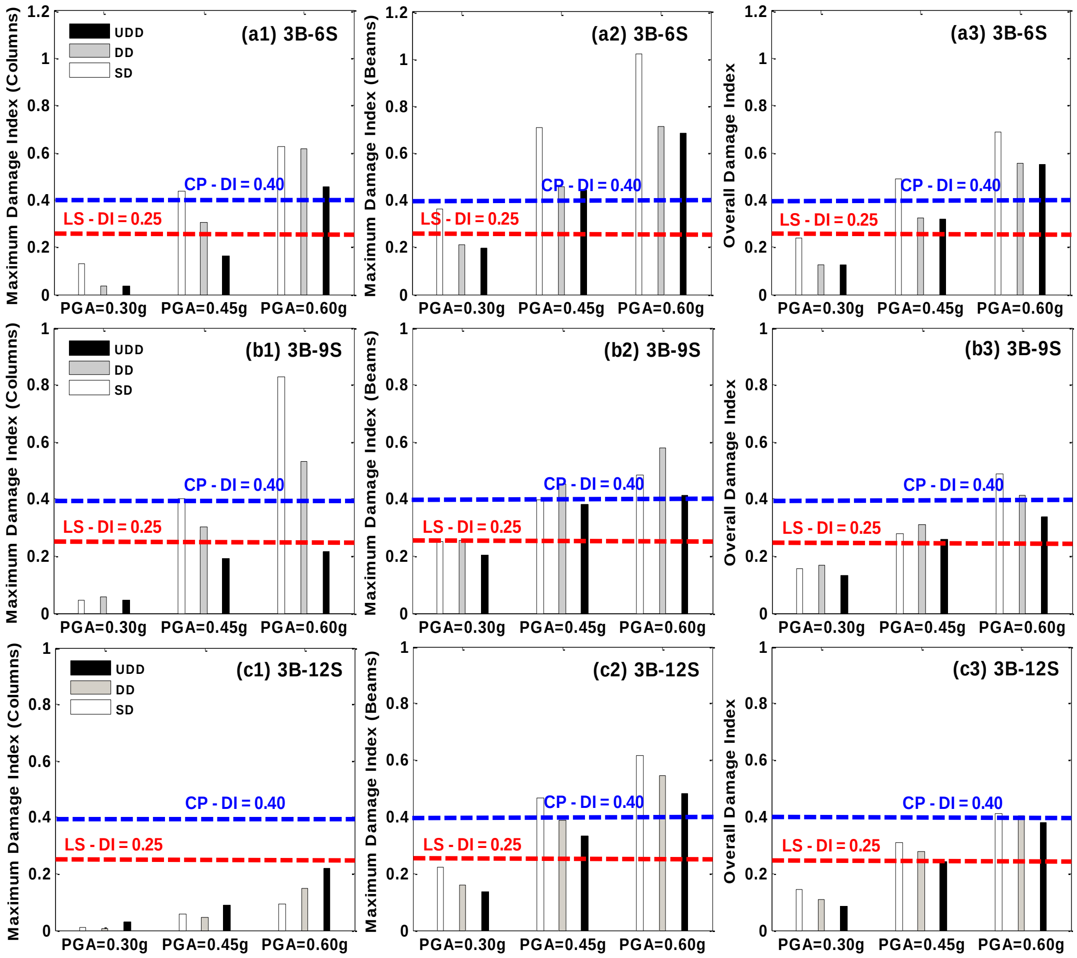

Fig. 8 Maximum damage index of columns (a1, b1, c1), beams (a2, b2, c2), and overall damage index (a3, b3, c3) of optimized frames

based index; and the SDI that has been consisted of two terms including the combination of normalized deformation and hysteretic energy dissipation indices weighted by dissipated hysteretic energy in the story of interest, is a deformation and energy-based combined index. Therefore, it could be concluded that the main difference between the distribution pattern of the SDI and ISDR over the height of the frames particularly during severe earthquakes is because of the portion of hysteretic energy dissipation considered in SDI compared with ISDR, and as shown in Figs. 5-7, it is rather sensible for the taller RC frames. It should be noted that a high SDI based on a modified Park-Ang damage

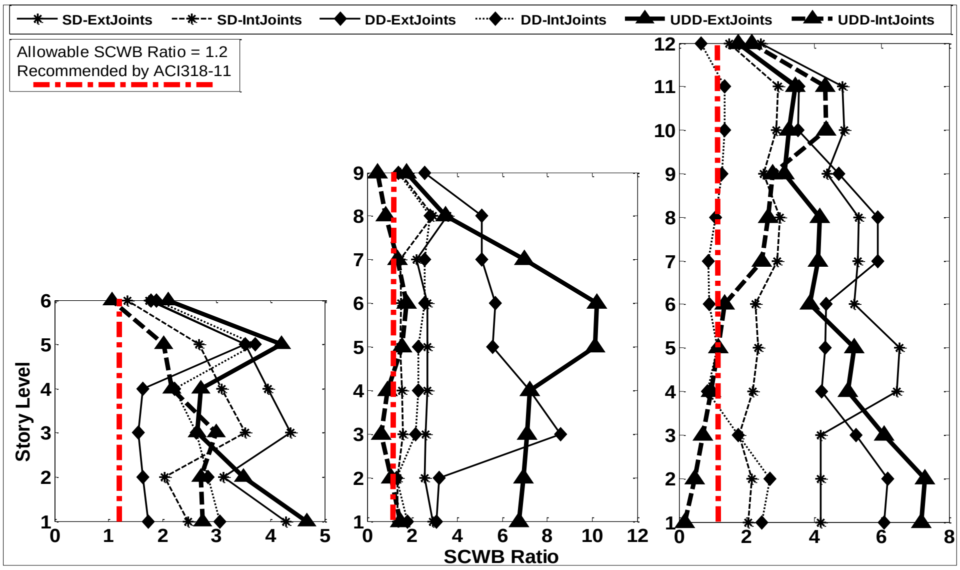

Fig. 9 The distribution of SCWB ratio of external and internal joints over the height of optimized 3B-6S, 3B-9S and 3B-12S frames using SD, DD and UDD

index could be due to the degree of damage of beams and/or columns. Therefore, the maximum damage index of beams and columns and the overall damage index for each optimally designed frame by SD, DD and UDD at each three levels of PGA have been shown in Fig. 8. Based on this figure, the maximum damage index of beams and columns regarding the 3B-6S and 3B-9S optimized frames by SD and DD is more than that of the UDD, while this difference is increased by increasing in PGA. Concerning the 3B-12S optimized frame, the damage index of columns designed by SD is only less than that of the UDD, whereas they lie in the acceptable range for each PGA level. In relation to the maximum damage index of beams of the 3B-12S optimized frame, the index for SD is more than that of the DD, and the DD is more than that of the UDD. Another remarkable point inferred from Fig. 8 is that the overall damage index for all optimized frames by the UDD is less than those of the SD and DD for each PGA level. It is important to note that the damage index of 0.4 is the boundary of reparability of the elements and the structure. In general, regarding the PGA of 0.45 g (maximum considered earthquakes), all the optimized frames using UDD has the local and overall damage indices of less than 0.4 except for the maximum damage index of the beams of 3B-6S. From an engineering point of view, since the maximum damage index of the columns of this frame is more important than that of the beams in terms of global collapse, the frame is almost acceptable compared with those of designed by SD and DD.

Next, the SCWB concept is discussed herein. The SCWB ratios for internal and external joints located in each story level of all optimized frames have been shown in Fig. 9. As shown herein, for all optimized frames designed by SD, the ratio is more than 1.20 and less than 4.37, 7.15 and 6.54 for 3B-6S, 3B-9S and 3B-12S optimized frames, respectively. Based on the current codes recommendations, it is expected that the desired collapse mechanism of global type occurs during the severe earthquakes.

According to the damage distribution pattern shown in Figs. 5-7, and the maximum local and overall damage indices depicted in Fig. 8, particularly about PGA of 0.45 and 0.6g, unlike the SCWB ratios of more than 1.2, it can be concluded that the damage concentration has been formed in the optimized frames by the SD, and SCWB could not guarantee the formation of the desired collapse mechanism. The number of the SCWB ratios of less than 1.2 for all optimized frames by UDD is: one internal joint at roof level for 3B-6S, five internal joints at different story levels of 3B-9S, and four internal joints at the first to fourth story levels of 3B-12S. Although there are several joints having SCWB ratios of less than 1.2, the distribution of SDI and even ISDR is more uniform than those of the SD and DD, and the status of both local and global damage indices shows the optimized frames by UDD are repairable under the PGA of 0.45 g. Moreover, about the 3B-12S optimized frames by UDD, despite the minimum SCWB ratio of 0.17, the columns have suffered a minor level of damage (less than 0.25), and in accordance with the damage distribution pattern shown in Fig. 7, and the maximum local and overall damage indices depicted in Fig. 8-(c1-c3), the damage distribution pattern of stories is more uniform than that of the SD and DD methods. According to the local and global damage indices of the optimized frames depicted in Figs. 5-8, the SD is led to damage concentration and forming the undesired collapse mechanism whilst the UDD is resulted in the uniform damage distribution and formation of the desired collapse mechanism of global type. The DD has a status between the SD and UDD in terms of both local and global damage indices and damage distribution pattern at the storey levels.

Finally, using the combined Park-Ang damage index, which includes the influence of hysteretic energy demand, and a new design criterion based on the damage pattern, which is extracted from the desired collapse mechanism of global type, in the UDD method would yield safer and more economical frames compared with the SD and DD methods.

7. Conclusions

Three fully automated optimum seismic design methods (SD as a code-based design method; DD and UDD as PBSD methods) were presented and investigated through three six-, nine-, and twelve-story frames with three bays. The following important points can be highlighted:

- Using the idea of uniform damage distribution, an efficient PBSD method named as UDD was proposed. The aim of the UDD is to reach an optimum design candidate with a reasonable construction cost and minimum structural damage, applicable for structural earthquake engineers.
- A novel damage pattern was proposed to use in the UDD. The effectiveness of the proposed damage pattern extracted from the theory of collapse mechanism of global type was shown. In addition, the efficacy of the allowable value of τ_{all} to reach the uniform damage distribution over the height of the structures during moderate to severe earthquakes was revealed.
- For the earthquakes with PGA of 0.3 g (design-based earthquakes), DD and UDD result in the satisfactory optimum design candidates in terms of both SDI and ISDR. This is also true for SD in terms of the overall damage index not local.
- Concerning the PGA of 0.45 g (maximum considered earthquakes) and 0.6 g in which large and cyclic deformations are formed in the structural elements, the single parameter of strength cannot be taken as an appropriate demand parameter in the seismic design. To clarify, for the PGAs of 0.45 g and 0.6 g in the case of the optimized frames based on the SD, despite having a SCWB ratio greater than 1.20, the frames have suffered a severe degree of damage due to damage concentration in one or more stories. This is also true regarding the DD. In order to eliminate the damage concentration, the idea of uniform damage distribution was proposed. The optimized frames using SD have the more (about 1 to 4%) construction cost, suffering about 30% (on average) more SDI compared with those of the proposed UDD. In fact, the effectiveness of the uniform distribution damage idea is more remarkable for severe earthquakes.
- It was revealed that in some cases, the distribution patterns of SDI and ISDR are different. In addition, based on the parameters used in SDI compared with ISDR, it can be stated that the most influential parameters to make such difference is related to hysteretic energy dissipated through the beams and columns of each storey.

The properties of the optimized frames can be investigated in order to design of new structures. For example, in accordance with the value of 1.2 recommended by ACI318 (2011), the value of SCWB at the joints of optimized frames using SD, DD and UDD, and its distribution pattern over the height of the structures may be useful to be studied more. Generally, the difference between the SCWB ratios of the internal joints of the optimized frames using UDD and 1.2 of ACI318 (2011) is less than that of the external joints.

Finally, the results show that the concepts of damage and uniform damage distribution are influential to seismic design of RC structures under strong ground motions, rather than the concept of strength.

Acknowledgments

My special thanks go to Prof. Eysa Salajegheh in Department of Civil Engineering at University of Kerman, Iran, and to Prof. Shahram Pezeshk in Department of Civil Engineering at University of Memphis, United States, for their invaluable comments.

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