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An Efficient Performance-Based Seismic Design Method for Reinforced Concrete Frames

Iman Hajirasouliha [1*], Payam Asadi [2], Kypros Pilakoutas [3]

[1] *Department of Civil Engineering, The University of Nottingham, Nottingham, UK*

[2] *Civil Engineering Department, Sharif University of Technology, Tehran, Iran*

[3] *Department of Civil & Structural Engineering, The University of Sheffield, Sheffield, UK*

* *Corresponding author. Email*: *i.hajirasouliha@nottingham.ac.uk*

SUMMARY

In this paper, a practical method is developed for performance-based design of reinforced concrete (RC) structures subjected to seismic excitations. More efficient design is obtained by redistributing material from strong to weak parts of a structure until a state of uniform deformation or damage prevails. By applying the design algorithm on 5, 10 and 15 storey RC frames, the efficiency of the proposed method is initially demonstrated for specific synthetic and real seismic excitations. The results indicate that, for similar structural weight, designed structures experience up to 30% less global damage compared to code-based design frames. The method is then developed to consider multiple performance objectives and deal with seismic design of RC structures subjected to a group of earthquakes. The results show that the proposed method is very efficient at controlling performance parameters and improving structural behaviour of RC frames.

KEY WORDS: Structural optimization; Structural damage; Reinforced concrete frames; Seismic performance; Non-linear dynamic behaviour

INTRODUCTION

Seismic design of structures is commonly based on strength or force considerations rather than displacement as the seismic design codes generally use lateral inertia forces to account for seismic ground motion effects. The distribution of these static forces (and therefore, stiffness and strength) is based implicitly on the elastic vibration modes [1, 2]. Therefore, as structures exceed their elastic limits in severe earthquakes, the use of inertia forces corresponding to elastic modes may not lead to the optimum distribution of structural properties [2, 3].

The need for finding cost-efficient and optimum structural designs has led to the development of different structural optimization methodologies. Optimum design of structures for seismic loads has been studied by many researchers over the past decades [4-8]. The conventional methods used in these studies are usually gradient-based solution strategies that require the satisfaction of some specific mathematical conditions. Due to the difficulty in calculating appropriate expressions for optimisation constraints, these methods cannot be practically applied for optimum design of non-linear structures subjected to seismic excitations.

The newly developed performance-based design methods [9-11], which are also a good indicator of future direction for seismic design codes, tend to take into account the non-linear seismic response of structures. These methods directly address inelastic deformations to indentify the levels of damage during severe seismic events. In one of the early attempts, Beck et al. [12] developed an optimization methodology for performance-based design of simple elastic structural systems operating in an uncertain dynamic environment. Ganzerli et al. [13] combined the contemporary concept of performance-based design with structural optimization methods. They introduced a nonlinear analysis-based approach where the performance-based constraints were applied in terms of plastic rotations of beams columns of frames, as suggested by FEMA guidelines [11]. Zou and Chan [14] presented a new optimisation procedure based on the "optimality criteria" concept. In this approach, by using the principle of virtual work and the Taylor series approximation, the nonlinear seismic response of an RC frame is expressed in terms of element design variables. In their proposed methodology, the inelastic drift response of the structure is determined by performing a non-linear push-over analysis, and therefore, higher mode effects are not taken into account. Fragiadakis and Papadrakakis [15] proposed a reliability-based optimisation approach based on nonlinear response history analysis. In this method, an evolutionary optimization algorithm is used to locate the most efficient design in terms of cost and performance through generating appropriate databases of RC beam and column sections.

This paper presents a simplified method for performance-based design of RC structures subjected to seismic excitations. This method is based on the concept of developing uniform distribution for deformation demands, and is capable of controlling different performance parameters such as inter-storey drift and cumulative damage. The proposed design method is examined for different performance targets and seismic loading scenarios.

PROPOSED METHODOLOGY

During strong earthquakes the deformation demand in code-based designed structures (especially capacity design) is not expected to be uniform [2, 3, 16]. As a result, the deformation demand in some parts of the structures does not necessarily utilize the maximum level of seismic capacity. If the strength of underused elements is decreased incrementally, for a ductile structure, it is expected to eventually obtain a status of uniform deformation demand. In such a case, the dissipation of seismic energy in each structural element is maximized and the material capacity is fully exploited. Previous studies showed that the seismic performance of such a structure is near optimal, and it undergoes less damage in comparison with code-based designed structures having similar structural weight [3, 17]. Therefore, in general, it can be assumed that a status of uniform deformation demand is a direct consequence of the optimum use of material. It should be pointed out that mathematically the optimum structure with minimum structural weight cannot necessary be shown to be the one with uniform deformation demand. Nevertheless, the proposed method has capability to decrease the required structural weight by exploiting fully the material capacity. Consequently, the aim of this study is to find a near optimum or more efficient design rather than a global optimum solution.

MODELLING AND ASSUMPTIONS

To demonstrate the method, three RC frames with 5, 10, and 15 storeys (as shown in Figure 1) were examined. The buildings were assumed to be located on a soil type D of the IBC-2009 [18] (and ASCE 7-05 [19]) category, with the design spectral response acceleration at short periods and 1-sec period equal to 1.1g and 0.64g, respectively. Frame members were designed to support gravity and lateral loads determined in accordance with the minimum requirements of IBC-2009 [18] and ACI 318-08 [20]. The uniform gravity loads have been considered as 54 and 48 kN/m for interior storeys and roof, respectively. The RC frames were assumed to satisfy intermediate ductility requirements. The frames had square columns decreasing in dimensions with height. Full details of the geometry of the structures are given in Figure 1.

In order to predict the seismic response of the RC frames, nonlinear time-history analysis was carried out using computer program IDARC [21]. The Properties of RC members are calculated by fibre models, and the solutions are obtained using step-by step integration of equations of motion using Newmark beta method. Rayleigh damping model with a constant damping ratio of 0.05 was assigned to the first mode and to any mode at which the cumulative mass participation exceeds 95%. Spread plasticity models were employed to model non-linear behaviour of beam and column elements. In this model, the plastic length is updated at each step of the analysis as a function of the instantaneous moment diagram in the element by using a yield penetration model combined with the spread plasticity formulation. The formulation can capture the change in the plastic hinge length and variation of the stiffness in beam and column elements under single or double curvature conditions. More information can be found in IDARC [21].

Four medium to strong ground motion records obtained from PEER online database [22] were used, EQ1: the 1992 Landers earthquake YER270 component with PGA (Peak Ground Acceleration) of 0.25g, EQ2: the 1994 Northridge earthquake NWH360 component with a PGA of 0.59g, EQ3: the 1995 Kobe earthquake KJM090 component with a PGA of 0.60g, and EQ4: 1992 Cape Mendocino earthquake PET090 component with PGA of 0.66g. All of these excitations correspond to sites of soil profile similar to type D of IBC-2009 [18] and were recorded in a low to moderate distance from the epicentre (less than 25 km) with rather high magnitudes (i.e. Ms>6.7). A synthetic earthquake (SEQ_{IBC}) was also generated using the SIMQKE program [23], having a close approximation to the elastic design response spectrum of IBC-2009 [18] with a PGA of 0.4g.

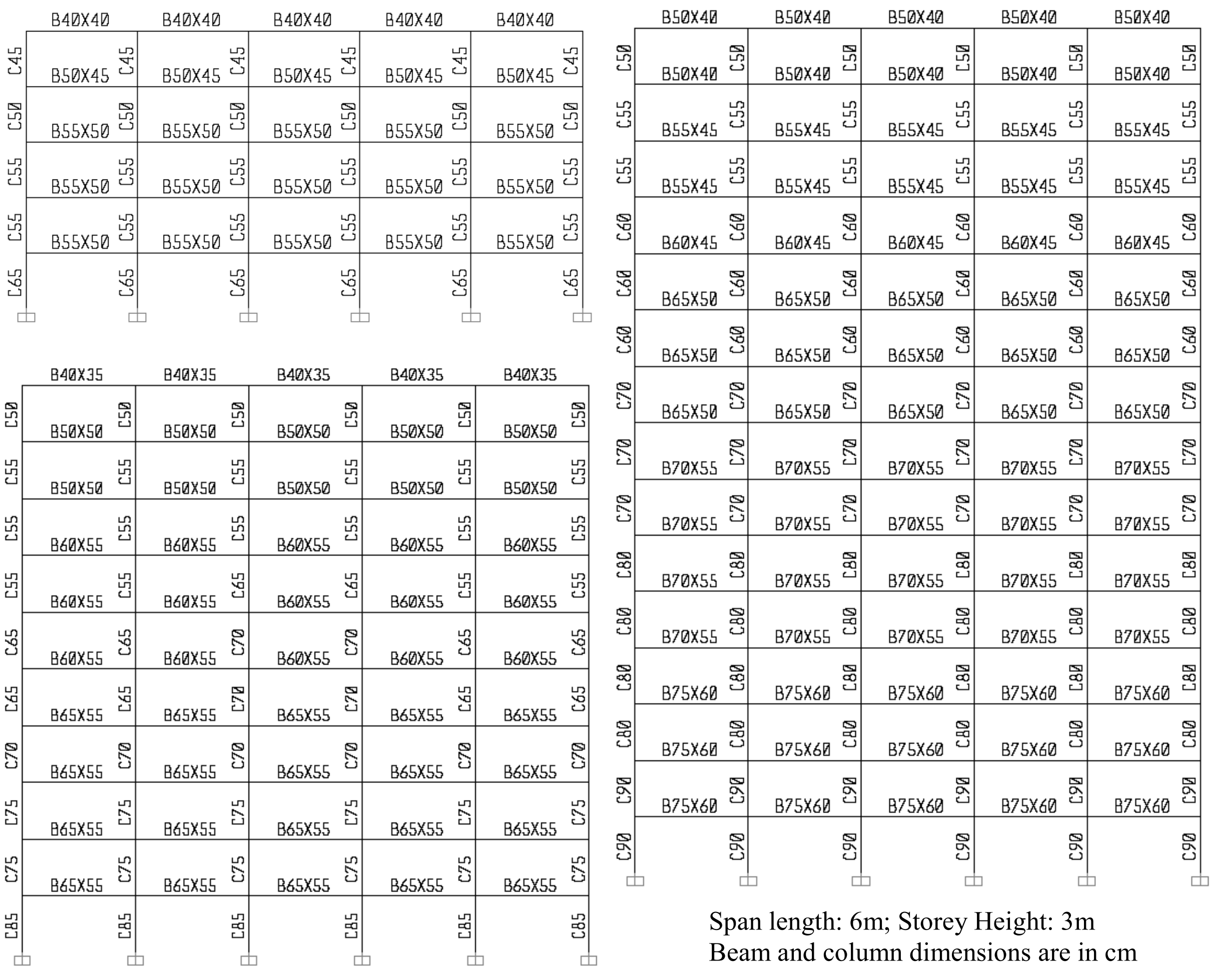

Figure 1. Beam and column dimensions and typical geometry of 5, 10 and 15 storey RC frames.

OPTIMUM SEISMIC DESIGN FOR SINGLE EARTHQUAKE EXCITATION

While conventional RC buildings are expected to remain in the elastic state during small earthquakes, they experience non-linear deformations under medium to strong earthquakes. The concrete section plays a more dominant role in providing lateral stiffness, and therefore, it is mainly responsible for controlling elastic drift under small earthquake loading. Although the concrete volume can significantly affect the cost of RC frames, in practice the dimensions of beam and column elements are usually determined at the initial stage of the design mainly to satisfy building code requirements (particularly to control lateral storey drift). In the proposed design method, the preliminary design should satisfy serviceability limit states under frequent earthquakes (mainly based on elastic behaviour), and then the design is optimized for higher performance levels such as Life Safety and Collapse Prevention. Within the nonlinear response range, the reinforcement ratio of structural elements is considered to be the main design variable, as flexural reinforcement plays a dominant role in controlling inter-storey drift and providing the required ductility. However, it is possible to optimise the size of beam and column elements in the initial design by utilizing one of the existing elastic response optimization methods such as Zou and Chan [24]. In this study, it is assumed that adequate shear confinement reinforcement is provided for each member, which is roughly proportional to the amount of flexural reinforcement. For simplicity, the compression steel ratio, ρ', is assumed to be linearly related (almost 50%) to the tension steel reinforcement ratio, ρ, for beams and identical for columns [14]. Consequently, the tension steel reinforcement ratio, ρ, can be considered as the major design variable in the proposed design method. The minimum and maximum ρ for columns was considered to be 1% and 4%, respectively. Based on ACI 318-08 [20], the minimum ρ for RC beams was 0. 35% and the maximum ρ was calculated to avoid brittle failure due to concrete crushing without steel yielding. It should be noted that most beams end up with a much lower ρ than the maximum, and this should ensure adequate ductility if suitable detailing is provided.

In this study, the iterative optimum design procedure developed by Hajirasouliha and Moghaddam [17] for optimum design of shear-building (mass-spring) models is extended for more efficient seismic design of RC frame structures. The optimisation target could be either (a): to minimise structural damage by optimizing the distribution of a specific total amount of reinforcement or (b): to obtain a structure with minimum total amount of reinforcement to satisfy a prescribed performance level. The proposed optimization algorithms for these two different scenarios are explained in detail in the following sections.

Minimum structural damage

In performance-based design methods, design criteria are expressed in terms of achieving specific performance targets during a design level earthquake. Performance targets could be satisfied by controlling the level of stress, displacement or structural and non-structural damage. The target of the design is to find the optimum distribution of reinforcement in a RC frame to minimize the expected structural damage for a design earthquake. The proposed method in this study can optimize the design of RC structures for different types of performance targets such as deformation, acceleration or cumulative damage. Among the different developed damage indexes in the literature, Park and Ang index [25] is one of the most adopted models for damage analysis of RC structures. The Park and Ang damage model accounts for damage due to maximum inelastic excursions, as well as damage due to the history of deformations. In this study, a modified Park and Ang damage model [21] is used to evaluate structural damage in beam and column elements during seismic excitations. In this model, the element end section damage is calculated using the following equation:

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_h \qquad (1)$$

Where θ_m is the is the maximum rotation attained during the loading history; θ_u is the ultimate rotation capacity of the section; θ_r is the recoverable rotation when unloading; β is the Park and Ang model constant parameter equal to 0.1 [21]; M_y is the yield moment; and E_h is the dissipated energy in the section. The element damage is then selected as the biggest damage index of the end sections. Storey and overall damage indices are computed using the weighted average of the local element damage indices based on the dissipated hysteretic energy of components.

In an attempt to reach uniform damage distribution in all structural elements, the following design procedure was employed:

1. The initial structure is designed for gravity and seismic loads based on a seismic design code, such as IBC-2009 [18]. The preliminary distribution of steel reinforcement is selected for all structural elements to obtain the most efficient initial design. The dimensions of beam and column elements are determined at this stage and remain unchanged during the design process. Although the near optimum design is not dependent on the initial structure, the speed of convergence is dependent on the preliminary distribution of steel reinforcement. Using a more educated starting design (e.g. based on a design-code spectrum), results in a faster convergence to the final solution.

2. The structure is subjected to the design seismic excitation, and the Park and Ang damage index is calculated for all beam and column elements.

3. The Coefficient of Variation *(COV)* of damage indices for beams (COV_b) and columns (COV_c) is calculated. If both COV_b and COV_c are small enough (e.g. less than 0.1), the structure is considered to be practically optimum. Otherwise, the design algorithm proceeds to iterations.

4. During the iterations, the distribution of longitudinal reinforcement in beam and column elements is modified. Longitudinal reinforcement is shifted from elements with lower damage index to the elements which experienced higher damage by using the following equations:

$$[(\rho_{beam})_i]_{n+1} = \left[\frac{(DI_b)_i}{(DI_b)_{ave}}\right]^{\alpha} [(\rho_{beam})_i]_n \tag{2}$$

$$[(\rho_{col})_i]_{n+1} = \left[\frac{(DI_c)_i}{(DI_c)_{ave}}\right]^{\beta} [(\rho_{col})_i]_n \tag{3}$$

   where $[(\rho_{beam})_i]_n$ and $[(\rho_{col})_i]_n$ are the tension steel reinforcement ratio of the ith beam or column element at nth iteration, respectively. $(DI_b)_i$ and $(DI_b)_{ave}$ are Park and Ang damage index for the ith beam and average of damage indices for all beam elements, respectively. Similarly, $(DI_c)_i$ and $(DI_c)_{ave}$ are damage indices for the ith column and average of damage indices for all column elements, respectively. α and β are convergence parameters ranging from 0 to 1. The nonlinear dynamic behaviour of RC structures is rather sensitive to changes in the reinforcement ratio of beam and column elements. Therefore, to obtain good convergence in numerical calculations, any alternation in the reinforcement ratio should be applied incrementally. For this purpose, convergence parameters α, β are used in the proposed design process. It will be shown in the following sections that these parameters play an important role in the convergence of the problem. In this study, convergence parameters α, β were set to be 0.1. It should be noted that using equations 2 and 3 can lead to different damage levels for beam and column elements to satisfy the strong-column/weak-beam design concept.

5. The longitudinal reinforcement ratios for all beam and column elements are scaled such that the total reinforcement weight remains unchanged.

6. The new RC frame is then analyzed to ensure that it can sustain the gravity loads (i.e. design dead and live loads). If any member fails, its longitudinal reinforcement is increased accordingly

to ensure the final design is capable of resisting gravity loads based on the capacity design concept. The design procedure is then repeated from step 2 until the *COV* of damage indices for both beam and column elements become small enough.

The above design algorithm has been used for more efficient seismic design of 5, 10 and 15-storey RC frames (shown in Figure 1) subjected to the five selected earthquake ground motions. The results indicate that, for similar total steel reinforcement weight, near optimum design structures always experience more uniform damage distribution and relatively less global damage index as compared with structures designed according to conventional design methods. For example, Figure 2 shows the distribution of storey damage indices for the three near optimum and conventionally designed RC frames subjected to SE_{QIBC}.

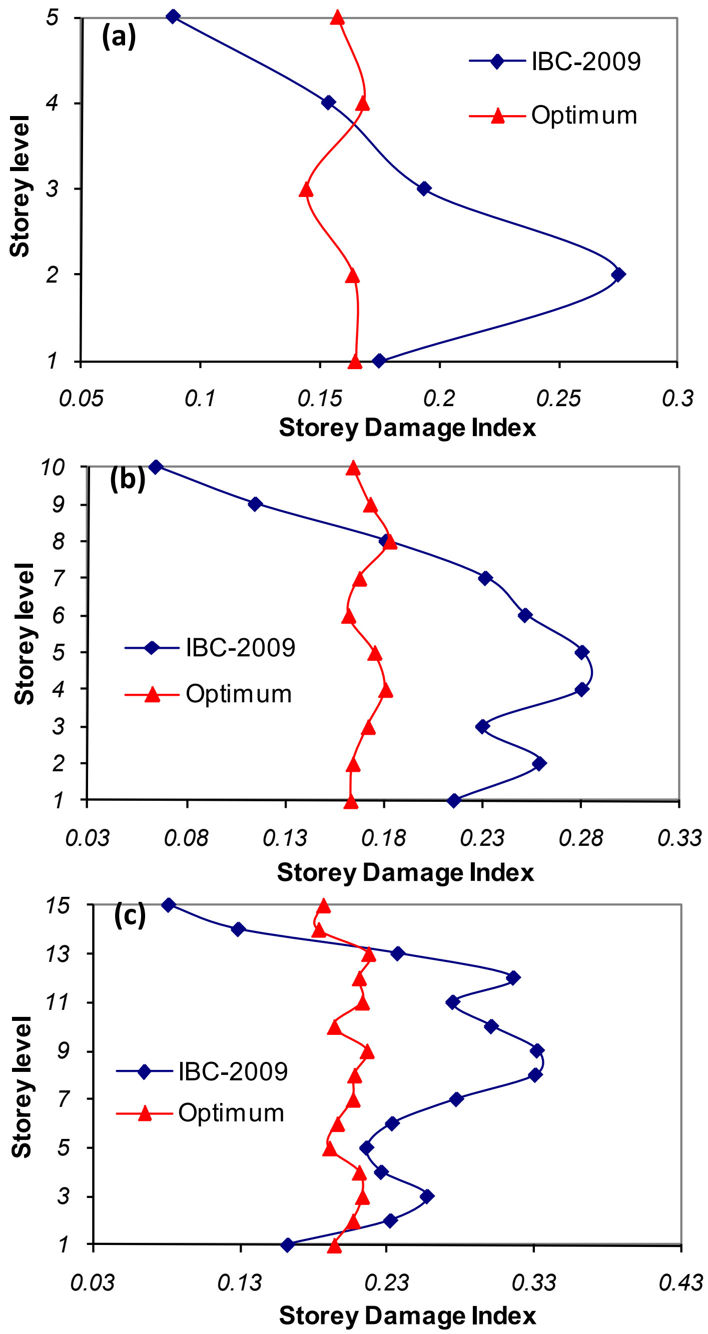

Figure 2. Storey damage distribution of IBC-2009 and near optimum design models subjected to the SEQ_{IBC}, (a): 5 storey model; (b): 10 storey model; (c): 15 storey model.

The global *DI* and *COV* of storey damage indices for IBC-2009 and near optimum design models are compared in Table 1. It can be deduced that for the same structural weight, near optimum models experience up to 30% less global damage. The proposed design method is capable of preventing high local structural damage as the performance parameters (i.e. structural damage indices) are directly controlled in the proposed design procedure. It should be mentioned that in practice uniform damage distribution may not be achieved if uniformity of the section properties and minimum reinforcement requirements are considered as design constraints. However, the proposed method always leads to a more efficient design by exploiting better the capacity of structural materials.

Table 1. Global *DI* and *COV* of storey damage indices for the IBC-2009 and near optimum design models subjected to the synthetic earthquake.

MODEL	*IBC-2009 Model*		*Near Optimum Model*	
	Global *DI*	***COV* of Storey Damage Indices**	**Global *DI***	***COV* of Storey Damage Indices**
5-Storey	0.203	0.38	0.160	0.06
10-Storey	0.243	0.34	0.171	0.04
15-Storey	0.274	0.30	0.203	0.06

Sensitivity analyses are performed by using convergence parameters α, β equal to 0.05, 0.1 and 0.5 for more efficient seismic design of the 5-storey frame. Figure 3 compares the variation of global damage index from IBC-2009 to the final solution for different α and β values. It is shown that as α and β increase from 0.05 to 0.1, the convergence speed increases without any fluctuation. However, for α and β equal to 0.5, the method is not stable and the problem does not converge to the final solution. It is concluded that using convergence parameters equal to 0.1 results in the best convergence for the design problems, as the proposed method practically converged to the final solution in less than 6 steps. Numerous analyses carried out in the present study indicate that, for RC frames, an acceptable convergence is usually obtained by using α and β value 0.1 to 0.2.

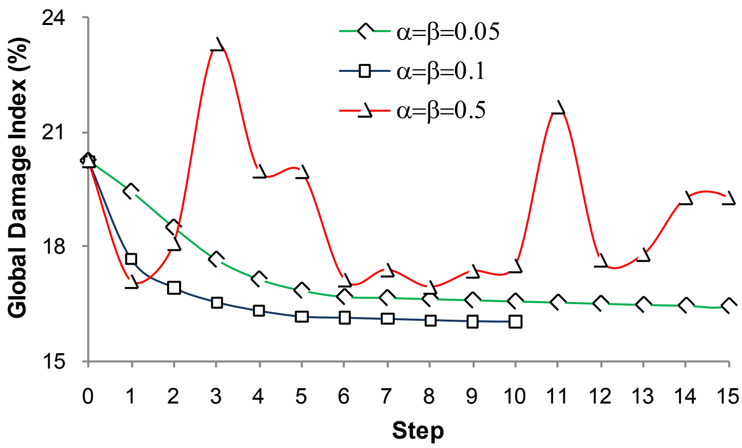

Figure 3. Variation of global damage index from IBC-2009 to final design for different α, β values, 5-storey frame subjected to SEQ_{IBC}.

To investigate the efficiency of the proposed design method for a set of different seismic excitations, Figure 4 compares the distribution of storey damage indices for near optimum and conventionally designed 10-storey frames subjected to the four selected seismic records. The results indicate that near optimum design models always experience more uniform structural damage and less maximum storey damage compared to the IBC-2009 models. It can be deduced from Table 2 that, for the same reinforcement weight, using the proposed design method on average resulted in a 25% reduction in the global damage index of the RC frames. The results indicate that decreasing the *COV* of element damage indices was always accompanied by a decrease in the global damage index of the RC frames. This is in agreement with the concept of uniform distribution of deformation demands as explained in the previous sections.

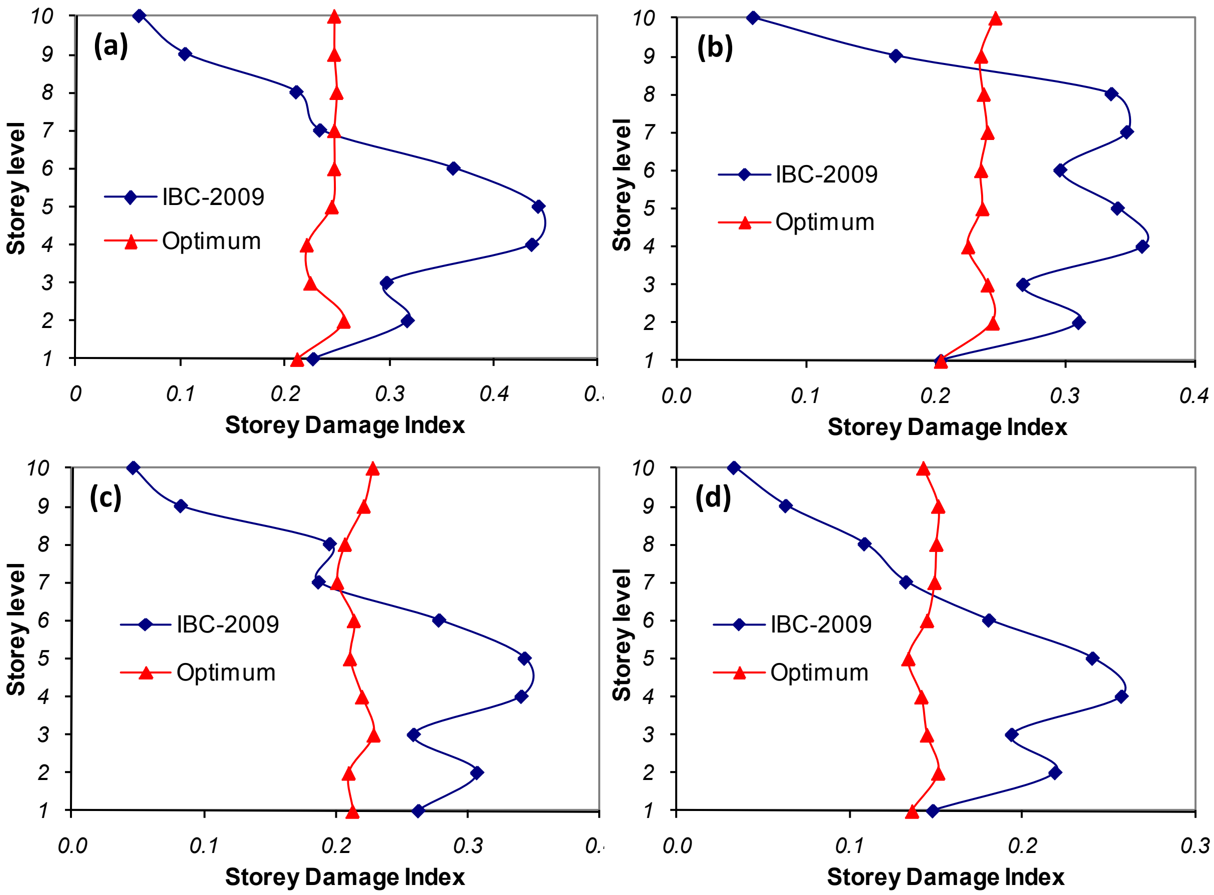

Figure 4. Storey damage distribution of IBC-2009 and near optimum design 10-storey frames subjected to (a): Northridge; (b): Kobe; (c): Lander; (d): Cape Mendocino.

Table 2. Comparison of Global *DI* and *COV* of storey damage indices for IBC-2009 and near optimum design 10-storey models subjected to four different earthquake ground motions.

Record	*IBC-2009 Model*		*Near Optimum Model*	
	Global *DI*	*COV* of Storey Damage Indices	Global *DI*	*COV* of Storey Damage Indices
Northridge	0.324	0.47	0.233	0.06
Kobe	0.301	0.36	0.225	0.05
Landers	0.190	0.46	0.145	0.04
Cape Mendocino	0.271	0.44	0.213	0.04

Minimum structural weight

The proposed more efficient design concept was also used to obtain a structure with minimum required reinforcement weight to satisfy a prescribed performance level. Performance-based design guidelines, such as SEAOC Vision 2000 [10] and FEMA 356 [11], place limits on acceptable values of response parameters; implying that exceeding these limits is a violation of a performance level. In this study, inter-storey drift is considered as the failure performance criterion. This response parameter is widely used as an indicator to measure the level of damage to the structural and non-structural components mainly because of the simplicity and convenience associated with its estimation. The following design algorithm was utilized to obtain the minimum weight of the structure:

1. The initial structure is designed based on a selected seismic design code, such as IBC-2009 [18].

2. The structure is subjected to the design seismic excitation. Inter-storey drifts are calculated and compared with the target value. If all of the calculated inter-storey drifts are close enough to the performance target (e.g. 5% tolerance), the RC structure is considered to be practically optimum. Otherwise, the design algorithm is continued.

3. Whilst storeys with inter-storey drift higher than the target value violate the performance objective, in the storeys with inter-storey drift less than the target value the material is not fully utilized. Steel reinforcement plays a significant role in controlling the inter-storey drift of an RC frame within its inelastic range of behaviour. Therefore, longitudinal reinforcement is reduced or increased accordingly. To achieve this, the following equation is used in this study:

$$\left[(A_{storey})_j\right]_{n+1} = \left[\frac{\Delta_j}{\Delta_{target}}\right]^{\gamma} \left[(A_{Storey})_j\right]_n \tag{4}$$

where $[(A_{storey})_j]_n$ is total longitudinal reinforcement weight in j^{th} storey (beam and column elements) at n^{th} iteration. γ is the convergence factor equal to 0.1. Δ_j and Δ_{target} are maximum and target inter-storey drifts at j^{th} storey, respectively. This modified reinforcement weight is distributed to the beams and columns of that storey based on their local damage index by using equations 2 and 3. The capacity design concept can be ensured by selecting appropriate Δ_{target} for each storey.

4. The longitudinal reinforcement ratios are scaled to reach the calculated reinforcement weight of A_{storey}.

5. The new RC frame is then analyzed to ensure that it can sustain the design gravity loads. If any member fails, its longitudinal reinforcement is increased accordingly. In this study, RC frames are designed to satisfy the strong-column/weak-beam design concept. As a result, plastic hinges are mostly formed in the beam ends under the design earthquake. Therefore, additional flexural reinforcement should be provided in the beam elements, and this is usually more than the required reinforcement to sustain gravity loads. The proposed design procedure is repeated from step 2 until the *COV* of inter-storey drifts decreases to an acceptable level. As the final design frames have to resist the design gravity loads, it is not usually possible to reach a very uniform inter-storey drift distribution, especially when the effect of gravity loads is dominant..

The above algorithm has been applied for more efficient seismic design of the three RC frames shown in Figure 1. The target drift ratio was considered to be equal to 1.5%, a value associated with the Life Safety (LS) performance level according to ATC-40 [9] and SEAOC Vision 2000 [10]. Figure 5 compares the inter-storey drift distribution for IBC-2009 [18] and near optimum design models subjected to the synthetic spectrum-compatible earthquake (SEQ_{IBC}). This synthetic earthquake is representative of the design spectrum, and therefore, can be utilized to evaluate the performance level of the designed RC frames. It is shown that using the proposed design method leads to a structure with a rather more uniform inter-storey drift distribution that is closer to the target value. This is particularly true for the top storeys where the effect of gravity loads on the columns is not significant.

The influence of the convergence parameter γ on the convergence of the proposed method is investigated for the 10-storey frame. Figure 6 compares the variation of required longitudinal reinforcement weight from IBC-2009 to the final solution for different γ values. It is shown that as γ increases from 0.05 to 0.1, the convergence speed increases without any fluctuation. However, for γ equal to 0.5, the method is not stable anymore and the problem does not converge to the final solution. It is concluded that using γ equal to 0.1 results in the best convergence for this design example. It is shown that by using γ equal to 0.1the proposed design method practically converged to the final solution in less than 10 steps without any fluctuation. The results of this study indicate that an acceptable convergence for RC frames is usually obtained by using γ value 0.1 to 0.2.

The steel reinforcement ratio of beam and column elements and the total required reinforcement weight for the 5, 10 and 15 storey IBC-2009 and near optimum design solutions are summarized in Tables 3 to 5. The results indicate that, for the same performance level (i.e. Life Safety), using the proposed design method resulted in 34 to 47% reduction in the required longitudinal reinforcement weight. In this study, at the initial stage of the design, the dimensions of beam and column elements are determined to meet code drift limitations. This has led to RC columns with relatively large dimensions and low reinforcement ratio especially for the 10 and 15 storey frames. As the code-specified minimum reinforcement ratio is controlled at each design iteration, the reduction in the longitudinal steel reinforcement of columns is practically limited. Therefore, as shown in Tables 3 to 5, a larger reduction in the longitudinal steel reinforcement is usually obtained for the beams rather than the columns.

It should be mentioned that increasing the amount of flexural steel reinforcement may not fully control the overturning bending effects in high-rise buildings. As a result, the proposed method may not lead to a uniform inter-storey drift response in high-rise frame structures in which overturning bending effects are significant, and more research is recommended for this type of building. However, the additional drift due to the axial deformation of columns at lower storeys does not usually contribute to the damage in structures [26]. Therefore, the proposed design method can always improve the seismic behaviour of RC frames by controlling the inelastic inter-storey drifts due to the rotation of plastic hinges.

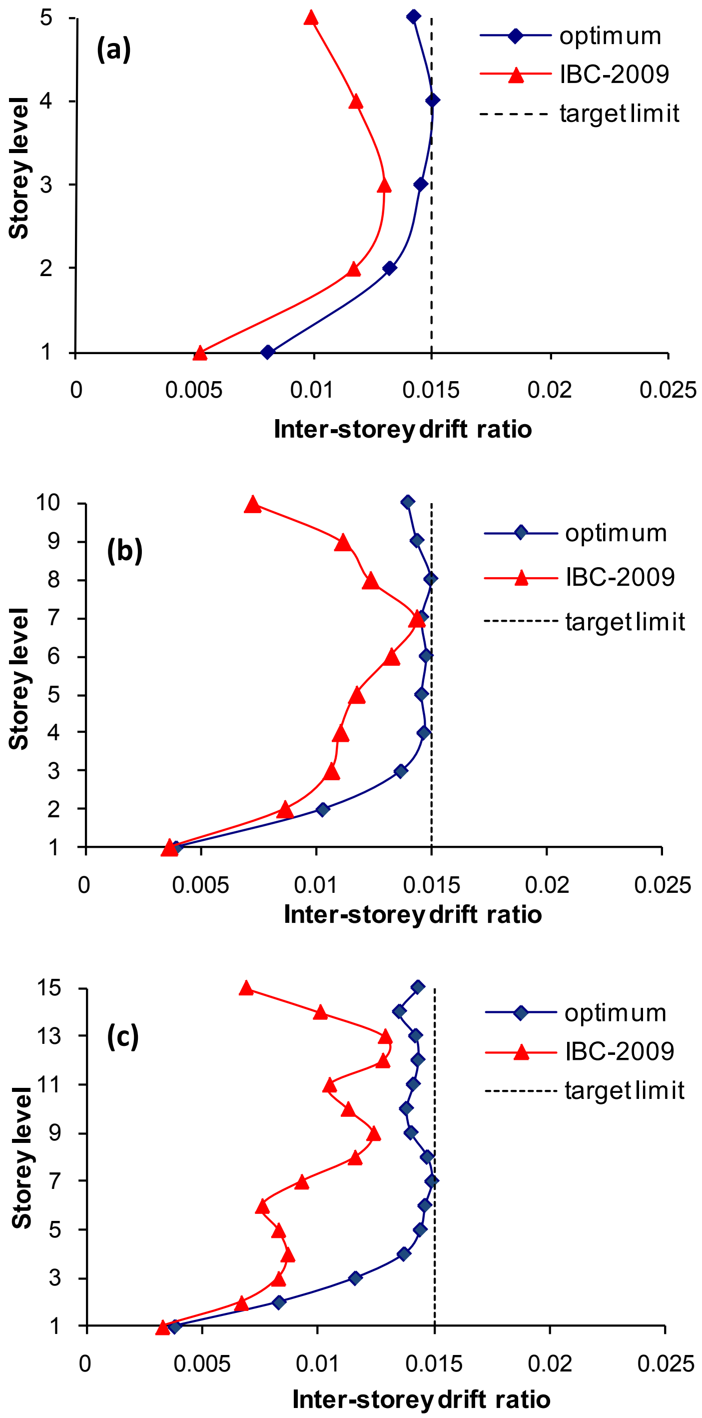

Figure 5. Inter-storey drift distribution of IBC-2009 and near optimum design models subjected to SEQ_{IBC}, (a): 5 storey model; (b): 10 storey model; (c): 15 storey model.

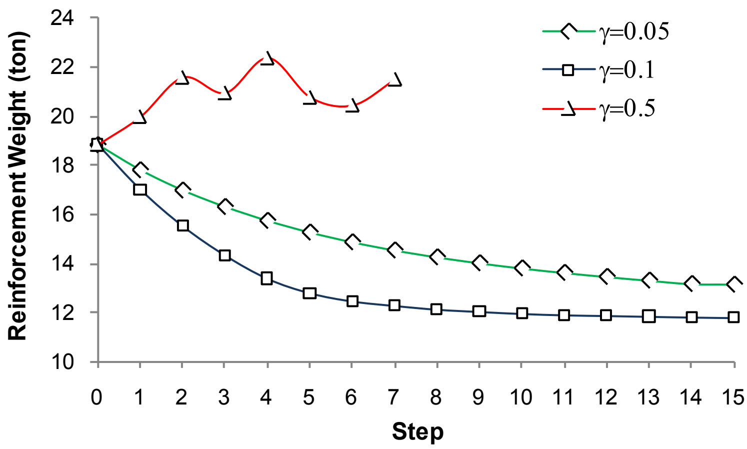

Figure 6. Variation of required longitudinal reinforcement weight from IBC-2009 to final design for different γ values, 10-storey frame subjected to SEQ_{IBC}.

Table 3. Comparison of steel reinforcement ratio and total required longitudinal reinforcement weight for 5-storey IBC-2009 and near optimum design models.

	Reinforcement Ratio (IBC-2009)			**Reinforcement Ratio (Near Optimum)**		
Storey	**Exterior Columns**	**Interior Columns**	**Beams**	**Exterior Columns**	**Interior Columns**	**Beams**
1	2.72%	2.86%	0.96%	1.12%	1.15%	0.35%
2	2.39%	2.83%	1.32%	1.40%	1.53%	0.88%
3	1.39%	1.83%	1.27%	1.07%	1.13%	0.46%
4	1.64%	2.18%	1.40%	1.16%	1.32%	0.85%
5	1.78%	1.78%	1.46%	1.00%	1.00%	0.70%
Total Reinforcement Weight	9578 (kg)			4953 (kg)		

Table 4. Comparison of steel reinforcement ratio and total required longitudinal reinforcement weight for 10-storey IBC-2009 and near optimum design models.

	Reinforcement Ratio (IBC-2009)			**Reinforcement Ratio (Near Optimum)**		
Storey	**Exterior Columns**	**Interior Columns**	**Beams**	**Exterior Columns**	**Interior Columns**	**Beams**
1	1.92%	1.59%	0.64%	1.14%	1.10%	0.36%
2	1.65%	1.28%	0.97%	1.18%	1.07%	0.36%
3	1.17%	1.21%	1.07%	1.00%	1.00%	0.57%
4	1.12%	1.24%	1.10%	1.00%	1.00%	0.63%
5	1.17%	1.31%	1.13%	1.00%	1.00%	0.67%
6	1.10%	1.24%	1.07%	1.00%	1.00%	0.57%
7	1.35%	1.68%	1.04%	1.13%	1.18%	0.51%
8	1.32%	1.54%	1.28%	1.07%	1.11%	0.70%
9	1.17%	1.29%	1.20%	1.00%	1.00%	0.68%
10	1.44%	1.35%	1.60%	1.00%	1.00%	0.54%
Total Reinforcement Weight	18836 (kg)			11811 (kg)		

Table 5. Comparison of steel reinforcement ratio and total required longitudinal reinforcement weight for 15-storey IBC-2009 and near optimum design models.

	Reinforcement Ratio (IBC-2009)			Reinforcement Ratio (Near Optimum)		
Storey	**Exterior Columns**	**Interior Columns**	**Beams**	**Exterior Columns**	**Interior Columns**	**Beams**
1	1.82%	1.28%	0.53%	1.22%	1.07%	0.36%
2	1.23%	1.05%	0.73%	1.06%	1.00%	0.36%
3	1.24%	1.07%	0.87%	1.08%	1.00%	0.36%
4	1.18%	1.04%	1.01%	1.00%	1.00%	0.56%
5	1.13%	1.00%	1.02%	1.00%	1.00%	0.59%
6	1.07%	1.00%	1.00%	1.00%	1.00%	0.53%
7	1.20%	1.12%	1.04%	1.00%	1.00%	0.49%
8	1.08%	1.08%	1.03%	1.00%	1.00%	0.45%
9	1.04%	1.06%	1.12%	1.00%	1.00%	0.51%
10	1.32%	1.39%	1.12%	1.08%	1.11%	0.50%
11	1.23%	1.38%	1.25%	1.05%	1.07%	0.60%
12	1.08%	1.32%	1.31%	1.00%	1.06%	0.55%
13	1.20%	1.42%	1.24%	1.00%	1.08%	0.46%
14	1.07%	1.11%	1.26%	1.00%	1.00%	0.53%
15	1.18%	1.08%	0.89%	1.00%	1.00%	0.37%
Total Reinforcement Weight	28409 (kg)			18625 (kg)		

OPTIMUM SEISMIC DESIGN FOR MULTIPLE PERFORMANCE OBJECTIVES

The proposed design concept can be easily extended to obtain a structure with minimum required reinforcement weight that satisfies multiple performance objectives. To achieve this, in the proposed design algorithm, Equation (4) should be substituted with the following equations:

$$\left[(A_{storey})_j\right]_{n+1} = \left[DR_j\right]^{\gamma}\left[(A_{Storey})_j\right]_n \tag{5}$$

$$DR_j = Max\left[\frac{(\Delta_j)_1}{(\Delta_{target})_1}, \frac{(\Delta_j)_2}{(\Delta_{target})_2}, \ldots, \frac{(\Delta_j)_k}{(\Delta_{target})_k}\right] \tag{6}$$

where $(\Delta_j)_k$ and $(\Delta_{target})_k$ are maximum and target inter-storey drifts of the j^{th} storey for k^{th} performance objective, respectively.

The efficiency of the proposed design method is demonstrated by applying the design algorithm on the 10-storey RC frame (Figure 1). In this study, the following performance objectives are considered: (a) Life Safety (LS) performance level in case of rare earthquakes (probability of occurrence 10% in 50 years); (b) Collapse Prevention (CP) performance level in case of very rare earthquakes (probability of occurrence 2% in 50 years). Two design spectrum-compatible synthetic earthquakes with a PGA of 0.4g and 0.6g are used as rare and very rare earthquakes, respectively. The target drift ratio was considered to be equal to 1.5% and 2.5% for the Life Safety (LS) and the Collapse Prevention (CP) performance levels as suggested by ATC-40 [9] and SEAOC Vision 2000 [10]. Figure 7 compares the inter-storey drift distribution of IBC-2009 and near optimum design frames subjected to the rare and very rare design earthquakes. It is shown that the final design satisfies both performance objectives with a rather more uniform inter-storey drift distribution compared to IBC-2009 design frame.

Table 6 shows the steel reinforcement ratio and the total required reinforcement weight for the near optimum design frame for this performance scenario. The results indicate that, for the same performance objectives, using the proposed design method resulted in a 32% reduction in longitudinal reinforcement weight compared to code-based designed structures.

Table 6. Steel reinforcement ratio and total required longitudinal reinforcement weight for near optimum 10-storey frame subjected to multiple performance objectives.

	Reinforcement Ratio (Near Optimum)		
Storey	**Exterior Columns**	**Interior Columns**	**Beams**
1	1.19%	1.14%	0.39%
2	1.21%	1.11%	0.37%
3	1.11%	1.12%	0.61%
4	1.00%	1.06%	0.63%
5	1.00%	1.00%	0.67%
6	1.00%	1.00%	0.57%
7	1.24%	1.26%	0.62%
8	1.18%	1.19%	0.77%
9	1.00%	1.00%	0.74%
10	1.00%	1.00%	0.92%
Total Reinforcement Weight	12555 (kg)		

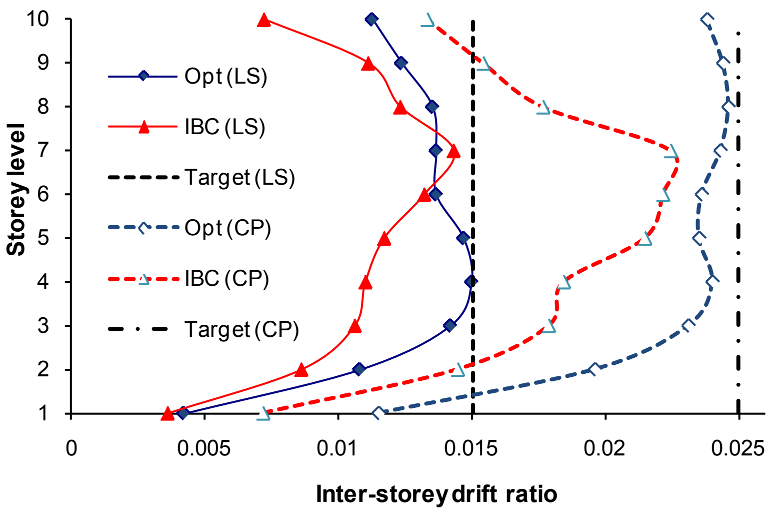

Figure 7. Inter-storey drift distribution of IBC-2009 and near optimum design frames subjected to rare (LS) and very rare (CP) design earthquakes.

OPTIMUM SEISMIC DESIGN FOR CODE DESIGN SPECTRUM

The proposed design method can be efficiently used to obtain an efficient design for a code design spectrum. In this case, a synthetic earthquake that is compatible with the design spectrum should be utilized in the proposed performance-based design process. To investigate the adequacy of the proposed method in this case, 5, 10 and 15-storey frames are designed with the synthetic earthquake compatible with IBC-2009 design spectrum. Subsequently, the near optimum and code-based design frames are subjected to a group of 10 synthetic spectrum-compatible earthquakes and the global damage index is calculated for each RC frame. The response spectra of the utilized synthetic earthquakes are compared with the IBC-2009 [18] design spectrum in Figure 8. Figure 9 compares the global damage index of the near optimum and IBC-2009 frames subjected to the 10 synthetic earthquakes. The results indicate that, for the same structural weight, 5, 10 and 15 storey RC frames designed with the synthetic spectrum-compatible earthquake experience on average 30% less global damage compared to IBC-2009 design frames under a group of code design compatible earthquakes.

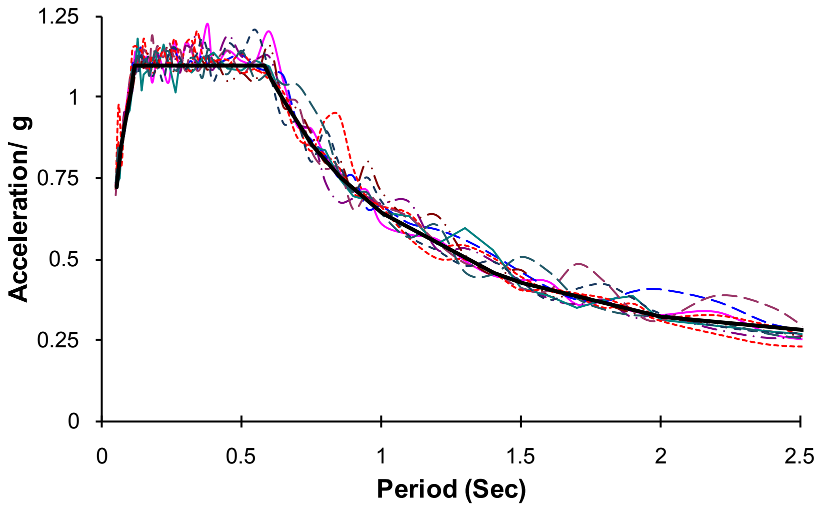

Figure 8. IBC-2009 design spectrum and response spectra of 10 synthetic earthquakes.

OPTIMUM SEISMIC DESIGN FOR A GROUP OF EARTHQUAKES

Based on the work presented in the previous sections, it was found that there is a specific distribution of reinforcement for each design spectrum that leads to a more efficient seismic design. Therefore, a remaining question that needs to be answered is whether this method can be utilized when structures are designed using a group of natural earthquake records. Previous studies by Hajirasouliha and Moghaddam [17] showed that, for shear building models, a better design loading pattern could be found by averaging the optimum patterns corresponding to a number of earthquakes representing a design spectrum. In this study a similar concept is adopted.

A new synthetic earthquake was generated using the SIMQKE program to represent the mean ground acceleration spectra of the four selected earthquakes (EQ 1 to 4). This synthetic earthquake (SEQ_{ave}) provides a close approximation to the mean spectrum of the design earthquakes as shown in Figure 10. To generate SEQ_{ave}, the mean ground acceleration spectra of the four design earthquakes were converted to a mean velocity spectrum as this was required as input by SIMQKE program.

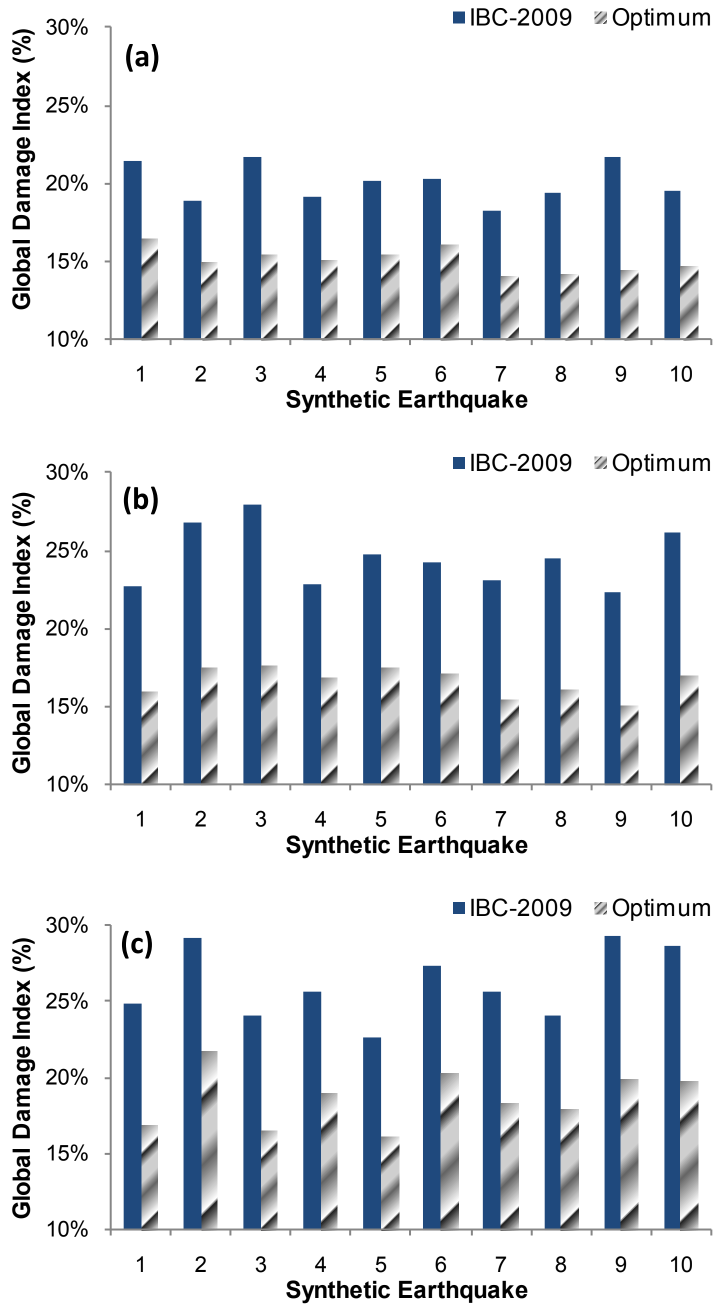

Figure 9. The global damage index of IBC-2009 and near optimum RC frames for a group of 10 synthetic spectrum-compatible earthquakes, (a): 5 storey model; (b): 10 storey model; (c): 15 storey model.

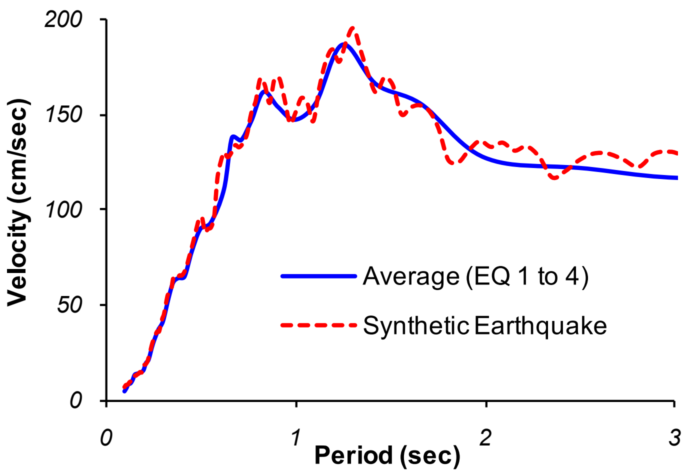

Figure 10. Comparison between synthetic earthquake velocity spectrum with the average spectrum.

Using the average synthetic earthquake, the proposed design algorithm was used for more efficient seismic design of the 5-storey frame to obtain a target drift ratio of 1.5%. The required longitudinal reinforcement weight in this case is around 9.1 ton that is 5% less than the conventional RC frame. Figure 11 compares the inter-storey drift distributions of IBC-2009 and near optimum design models subjected to the average synthetic earthquake. While the IBC-2009 RC frame does not meet the target performance objective, it is shown that the inter-storey drift ratio of the near optimum designed structure is less than 1.5% for all storeys. This shows that the final design frame has both less longitudinal reinforcement and better seismic performance compared to the IBC-2009 model.

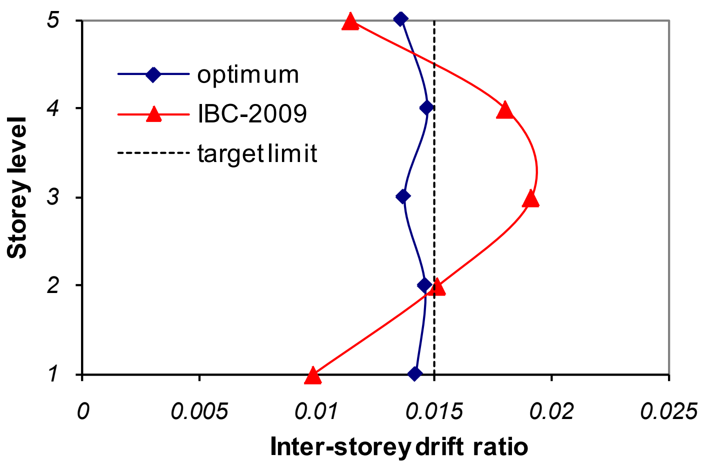

Figure 11. Inter-storey drift distributions of IBC-2009 and near optimum design models subjected to the average synthetic earthquake, 5-storey frame.

To investigate the efficiency of the method for real seismic excitations, the near optimum 5-storey structure designed with the average synthetic earthquake and the IBC-2009 designed structure were subjected to the four selected earthquakes (EQ 1 to 4). Maximum inter-storey drifts are shown in Table 7. The results confirm that the near optimum RC frame always experienced more uniform inter-storey drift distribution (i.e. less COV) and less maximum inter-storey drift compared to the conventionally design frame.

Table 7. Maximum inter-storey drifts for IBC-2009 and near optimum design model (based on the average synthetic earthquake) in four different seismic excitations.

Maximum inter-storey drift ratio (%)								
Storey	Kobe		Landers		Northridge		Cape Mendocino	
	IBC	Opt	IBC	Opt	IBC	Opt	IBC	Opt
5	1.77	2.31	0.53	0.62	1.20	1.83	1.00	1.70
4	2.93	2.82	0.65	0.64	1.59	1.22	1.86	1.52
3	3.66	2.80	0.71	0.62	1.83	1.31	2.81	1.92
2	2.92	2.63	0.68	0.67	1.41	1.36	2.49	2.45
1	1.44	2.36	0.32	0.55	0.58	0.95	1.29	2.12
COV	0.36	0.09	0.28	0.07	0.36	0.24	0.41	0.19

The average and envelop of maximum inter-storey drifts for the IBC-2009 and near optimum design frames in the four selected earthquakes (EQ 1 to 4) are calculated and compared in Figure 12. The results indicate that maximum and average inter-storey drifts experienced by 5-storey frame designed with the SEQ_{ave} are more uniform and around 20% less compared to the code-based designed frame. It is concluded that to improve the seismic performance of an RC frame for a group of earthquakes, it can be designed based on the average seismic excitation. It should be mentioned that the proposed design approach is general and can be used for any set of similar earthquakes representing a design spectrum.

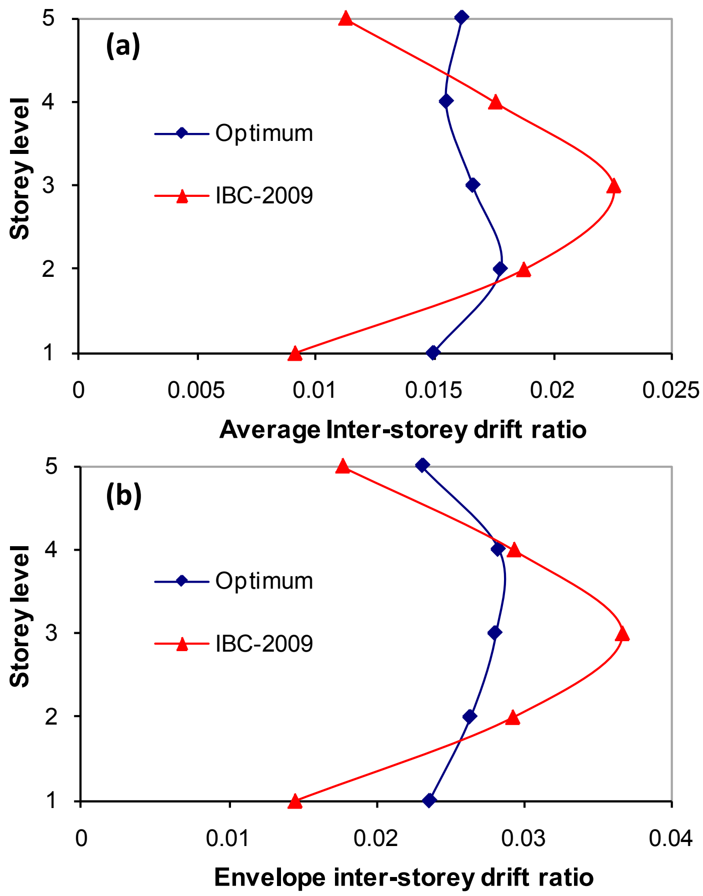

Figure 12. (a) Average and (b) envelope of inter-storey drift distributions for IBC-2009 and near optimum design models subjected to four selected earthquakes (EQ1 to 4).

Based on the above discussions, the proposed design method seems to be reliable and should prove useful in practical performance-based seismic design phase and in improving the efficiency and performance of RC buildings in seismic regions. However, the efficiency of the method for high-rise buildings should be further investigated.

CONLUSIONS

In this study a more efficient performance-based design method is proposed for the seismic design of RC structures. Based on the results, the following conclusions can be drawn:

- The concept of uniform distribution of deformation demands can be used efficiently to find better distribution of longitudinal reinforcement for RC structures subjected to gravity loads and seismic excitations.
- To achieve a specific performance level, the proposed design method could result in more than 50% reduction in the required longitudinal reinforcement weight.
- The results indicate that, for the same structural weight, a near optimum design RC frame may experience up to 30% less global damage compared to a similar code-based design frame.
- The proposed design method can be used for multiple performance objectives. It is shown that the proposed method leads to a structure with 33% less longitudinal reinforcement, while satisfying both Life Safety and Collapse Prevention performance objectives.
- Using an average synthetic earthquake in the proposed design process leads to a better seismic performance. It is shown that RC frames designed with the average synthetic earthquake always exhibit less global damage and more uniform inter-storey drift distribution.

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