Steel and Composite Structures, *Vol. 20, No. 2 (2016) 317-335*
DOI: http://dx.doi.org/10.12989/scs.2016.20.2.317

Effects of uncertainties on seismic behaviour of optimum designed braced steel frames

Iman Hajirasouliha [*1], Kypros Pilakoutas [1a] and Reza K. Mohammadi [2b]

[1] *Department of Civil & Structural Engineering, The University of Sheffield, Sheffield, UK*
[2] *Civil Engineering Department, K.N. Toosi University of Technology, Tehran, Iran*

(*Received August 12, 2015, Revised October 27, 2015, Accepted November 03, 2015*)

Abstract. Concentrically braced steel frames (CBFs) can be optimised during the seismic design process by using lateral loading distributions derived from the concept of uniform damage distribution. However, it is not known how such structures are affected by uncertainties. This study aims to quantify and manage the effects of structural and ground-motion uncertainty on the seismic performance of optimum and conventionally designed CBFs. Extensive nonlinear dynamic analyses are performed on 5, 10 and 15-storey frames to investigate the effects of storey shear-strength and damping ratio uncertainties by using the Monte Carlo simulation method. For typical uncertainties in conventional steel frames, optimum design frames always exhibit considerably less inter-storey drift and cumulative damage compared to frames designed based on IBC-2012. However, it is noted that optimum structures are in general more sensitive to the random variation of storey shear-strength. It is shown that up to 50% variation in damping ratio does not affect the seismic performance of the optimum design frames compared to their code-based counterparts. Finally, the results indicate that the ground-motion uncertainty can be efficiently managed by optimizing CBFs based on the average of a set of synthetic earthquakes representing a design spectrum. Compared to code-based design structures, CBFs designed with the proposed average patterns exhibit up to 54% less maximum inter-storey drift and 73% less cumulative damage under design earthquakes. It is concluded that the optimisation procedure presented is reliable and should improve the seismic performance of CBFs.

Keywords: Monte Carlo simulation; optimum seismic design; concentrically braced frame; seismic performance; non-linear behaviour

1. Introduction

Concentrically braced steel frames (CBFs) are one of the popular lateral-load resisting systems used in seismic areas. The preliminary design of CBFs is commonly based on the equivalent static force approach, in which the dynamic inertial forces due to seismic vibrations are represented by equivalent static forces. The height-wise distribution of these static forces is implicitly based on the fundamental elastic vibration modes (Hart 2000). Considering that structures do not remain elastic during severe earthquakes, the current capacity design approach in general does not lead to

*Corresponding author, Senior Lecturer, E-mail: i.hajirasouliha@sheffield.ac.uk
[a] Professor, E-mail: k.pilakoutas@sheffield.ac.uk
[b] Associate Professor, E-mail: rkarami@kntu.ac.ir

http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

a uniform distribution of ductility demands (Chopra 2012, Priestley *et al*. 2007) and, therefore, optimum use of structural materials (Karami *et al*. 2004, Moghaddam and Hajirasouliha 2006).

The seismic behaviour of code-based design steel frames has been extensively investigated both theoretically and experimentally (Lee and Foutch 2002, Yousuf and Bagchi 2009, Koboevic *et al*. 2012, McCrum and Broderick 2013, Jazany *et al*. 2013, Hsiao *et al*. 2014). It was found that, in general, CBFs that comply with new code requirements satisfy the collapse prevention and immediate occupancy performance levels, but they may exhibit extensive damage during strong earthquakes. Moreover, the seismic performance in the nonlinear response range is considerably more sensitive to variations in mechanical properties of structural elements and characteristics of future earthquake excitations (Haukaas and Kiureghian 2003, Kwon and Elnashai 2006). Kazantzi *et al*. (2014) investigated the effects of strength and ductility uncertainty and showed that ignoring the model parameter uncertainties may lead to un-conservative estimates of fragility for local damage-states. While structural uncertainties can affect the seismic performance of both optimum and code-designed structurers, they generally have a greater impact on the optimum design structures, since all structural elements are fully exploited and there is no extra capacity to deal with the effect of variations in structural properties. Therefore, quantifying the effect of uncertainties and understanding the reliability of design solutions are important issues in seismic design and optimization of steel structures.

Several studies are available on single and multi-objective reliability-based optimization of different structural systems (e.g., Fu and Frangopol 1990, Beck *et al*. 1999, Papadrakakis *et al*. 2005, Liu *et al*. 2005, Lagaros *et al*. 2008, Zacharenaki *et al*. 2013). The purpose of these studies was to find the optimal values of a set of design parameters to minimize different objective functions such as total structural weight, total cost, and element and system failure probabilities. However, most of these methods assume an elastic behaviour or use equivalent static forces for the seismic design of their structures. Moghaddam *et al*. (2005) and Karami and Sharghi (2014) developed a practical optimization method for seismic design of steel braced frames based on the concept of uniform damage distribution. While such methodologies can take into account the non-linear dynamic behaviour of the structures, they are based on a single design earthquake. Therefore, the impact of system and ground-motion uncertainty on the performance of optimum solutions should be investigated before such methods are widely accepted.

The focus of the present study is to quantify the effects of uncertainty in structural properties and design earthquake ground motion on the seismic performance of both optimum and conventionally designed CBFs. A set of 5, 10 and 15 storey CBFs are examined under different seismic excitations and the effects of variation in mechanical properties of structural elements, damping ratio and seismic excitation are investigated using the Monte Carlo simulation method. Based on the results of this study, a practical method is developed for optimum seismic design of CBFs for a group of earthquakes representing a specific design spectrum.

The paper starts out by explaining the developed FE models and the key performance-based design criteria for CBFs, followed by a description of the adopted optimisation method based on the concept of uniform damage distribution. Subsequently, the effects of uncertainties in storey shear strength and damping ratio on the seismic response of optimum and conventionally designed frames are investigated using the Monte Carlo simulation method. Finally, a simple method is proposed to manage the ground-motion uncertainty in seismic design of CBFs, and the efficiency of the method is demonstrated by using several design examples.

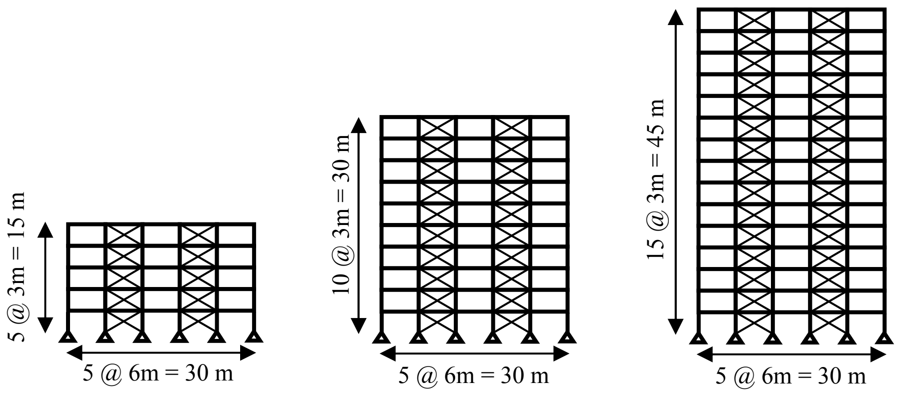


Fig. 1 Typical geometry of concentric braced frames

2. Analytical models

Series of 5, 10 and 15-storey concentrically braced steel frames (CBFs), with a typical geometry shown in Fig. 1, are used in the present study. The buildings are assumed to be located on soil class D of IBC-2012 (and ASCE 7-05), with design spectral response accelerations at short period and 1-sec period equal to 1.1g and 0.64 g, respectively. Ordinary CBFs were designed to support gravity loads and lateral loads in accordance with the minimum requirements of ANSI/AISC 360-5 and ANSI/AISC 341-05. Simple beam to column connections were used such that no moment is transmitted from beams to supporting columns. It should be noted that, in practice, the presence of gusset-plates in pinned beam-to-column connections does provide some limited moment-resistance, which is considered to be negligible in this study as such moment is also ignored in structural design.

The live load and dead load values for floors were 2.5 kN/m^2 and 4.5 kN/m^2, respectively, and the top storey was considered to be 25% lighter than the others. IPB (wide flange I-section), IPE (medium flange I-section) and UNP (U-Channel) sections, according to DIN-1025, were chosen for columns, beams and bracings, respectively. To model rigid diaphragms, all nodes at the same floor were constrained together in the horizontal direction. Although the brace elements in CBFs can generally provide adequate lateral stiffness to meet the code drift requirements, the code-based designed structures were checked to ensure they meet the maximum drift limitations.

To predict the seismic response of the CBFs, nonlinear time-history analyses were carried out using computer program DRAIN-2DX (Prakash *et al.* 1992). The Rayleigh damping model with a constant damping ratio of 0.05 was assigned to the first mode and the mode at which the cumulative mass participation exceeded 95%. A non-linear beam-column fibre element model, which allows the formation of axial-moment (*P-M*) plastic hinges near its ends, was employed to model steel columns. The brace elements and their gusset-plate connections were designed to prevent fracture under the design seismic loads. The post-buckling behaviour of the brace members was taken into account by utilizing the hysteretic model suggested by Jain *et al.* (1980) for axially loaded steel members. In this model, the axial compressive strength of bracing elements is reduced to consider the influence of buckling on their hysteretic behaviour and energy dissipation capacity. This hysteresis model, in general, predicts well the inelastic behaviour observed in experimental investigations (Dicleli and Calik 2008). However, more sophisticated analytical models can be used to simulate the non-linear behaviour and fracture of the brace elements (e.g., Broderick *et al*. 2008).

Fifteen strong ground motion records with similar characteristics, including six components of Imperial Valley 1979 and nine components of Northridge 1994, were selected to investigate the seismic performance of CBFs under real earthquake excitations. Table 1 shows the characteristics of the selected records. All of these excitations correspond to soil class D of IBC-2012 (selected design spectrum) and were recorded in low to moderate distances from the epicentre (less than 45 km) with rather high local magnitudes (i.e.,, $M > 6.7$). Since this exercise is only for comparison purposes, the earthquakes were used directly without being normalized. In addition, to investigate the seismic performance of the designed CBFs under IBC-2012 design spectrum, a set of ten spectrum-compatible earthquakes were artificially generated using SIMQKE program (Vanmarcke *et al*. 1999). It is shown in Fig. 2 that, on average, these earthquakes provide a close approximation

Table 1 Selected ground motion records

EQ No.	Earthquake	Station	*Ms*	PGA (g)
1	Imperial Valley 1979	H-E04140	6.9	0.49
2	Imperial Valley 1979	H-E04230	6.9	0.36
3	Imperial Valley 1979	H-E05140	6.9	0.52
4	Imperial Valley 1979	H-E06230	6.9	0.44
5	Imperial Valley 1979	H-E08140	6.9	0.45
6	Imperial Valley 1979	H-EDA360	6.9	0.48
7	Northridge 1994	CNP196	6.7	0.42
8	Northridge 1994	JEN022	6.7	0.42
9	Northridge 1994	JEN292	6.7	0.59
10	Northridge 1994	NWH360	6.7	0.59
11	Northridge 1994	RRS228	6.7	0.84
12	Northridge 1994	RRS318	6.7	0.47
13	Northridge 1994	SCE288	6.7	0.49
14	Northridge 1994	SCS052	6.7	0.61
15	Northridge 1994	STC180	6.7	0.48

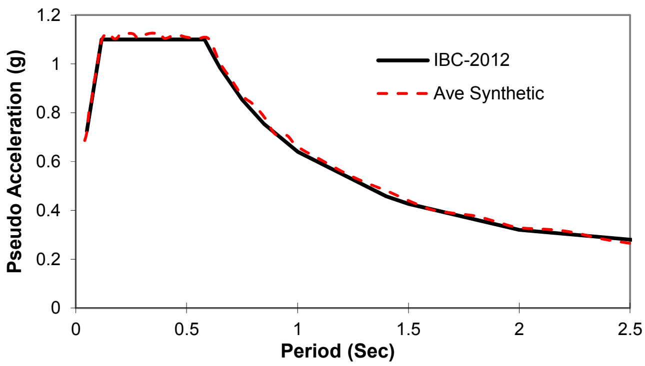


Fig. 2 IBC design spectrum and average response spectra of 10 synthetic earthquakes

to the design response spectra of IBC-2012 (soil class D), which was used for the seismic design of the CBFs in this study.

3. Performance-based design criteria

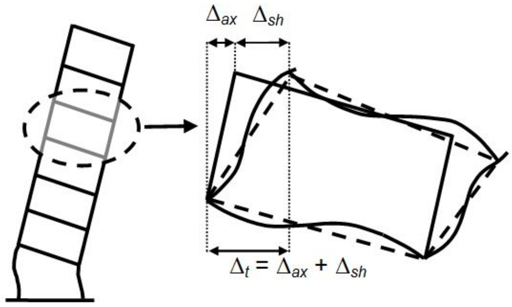
In modern performance-based seismic design guidelines (such as ASCE/SEI 41-13), acceptance criteria are expressed in terms of different performance parameters such as lateral roof displacement, inter-storey drift, plastic deformation and plastic hinge rotation. In this study, maximum inter-storey drift is considered as the failure performance criterion to evaluate the efficiency of CBFs under design earthquakes. This response parameter is simple to determine and is widely used to define the level of damage to the structural and non-structural elements. However, in concentrically braced frames, the axial deformation of columns results in additional lateral inter-storey drifts, which do not directly contribute to the structural damage at the storey level (Bertero *et al*. 1991). Considering the 2-D frame shown in Fig. 3(a), the total inter-storey drift (Δ_t) in each storey is a combination of the shear deformation (Δ_{sh}) due to shear flexibility of the storey, and the flexural deformation (Δ_{ax}) due to axial flexibility of the lower columns. Hence, inter-storey drift can be expressed as

$$\Delta_t = \Delta_{sh} + \Delta_{ax} \tag{1}$$

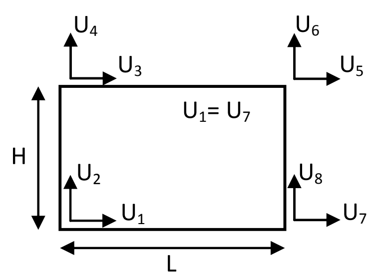
As it is shown in Fig. 3(a), Δ_{ax} is due to the rigid body motion of each storey and does not directly contribute to the structural damage imposed by earthquake excitations, although it may affect the lateral stability due to P-Δ effects (Bertero *et al*. 1991). Previous studies on steel braced frames showed that the flexural deformation (Δ_{ax}) can be up to 30% of the total inter-storey drift in the top storeys of high-rise buildings (Moghaddam *et al*. 2005). For a single panel, shear deformation, which causes damage to the structure, can be calculated using the following approximate equation

$$\Delta_{sh} = \Delta_t + \frac{H}{2L}\left(U_6 + U_8 - U_2 - U_4\right) \tag{2}$$

where, U6, U8, U2 and U4 are vertical displacements, as shown in Fig. 3(b), and H and L are height of the storey and span length, respectively. The axial deformation of beams is neglected in Eq. (2). For multi-span models, the maximum value of the shear drift in different panels at the same level is considered as the shear inter-storey drift.



(a) Shear inter-storey drift



(b) Displacement components

Fig. 3 Definition of shear inter-storey drift

4. Seismic performance evaluation

To investigate the efficiency of the utilized optimization technique to reduce global structural damage, a damage index based on the classical low-cycle fatigue approach is adopted (Krawinkler and Zohrei 1984, Baik *et al*. 1988). The storey inelastic shear deformation is chosen as the basic damage quantity, and the cumulative damage index after N excursions of plastic deformation is calculated by

$$D_i = \sum_{j=1}^{N} \left(\frac{\Delta\delta_{pj}}{\delta_u} \right)^c \tag{3}$$

where D_i is the cumulative damage index at i^{th} storey, ranging from 0 for undamaged to 1 for severely damaged storeys, N is the number of plastic excursions, $\Delta\delta_{pj}$ is the plastic deformation of i^{th} storey in j^{th} excursion, and δ_u is the nominal ultimate deformation. The power factor c is a parameter that accounts for the effect of magnitude of plastic deformation, which is taken to be 1.5 as suggested by Krawinkler and Zohrei (1984). To evaluate the level of damage exhibited by the whole structure, the global damage index is obtained as a weighted average of the damage indices at storey levels, with the weighting function being the energy dissipated at each storey

$$D_g = \frac{\sum_{i=1}^{n} D_i W_{pi}}{\sum_{i=1}^{n} W_{pi}} \tag{4}$$

where D_g is the global damage index, W_{pi} is the energy dissipated at i^{th} storey, D_i is the damage index at i^{th} storey, and n is the number of storeys.

To prevent severe damage and failure in some storeys, the optimised design should aim to provide a uniform damage distribution under earthquake actions. Karami *et al*. (2004) and Hajirasouliha and Moghaddam (2009) showed that, in general, the seismic performance of structures can be improved by redistributing material from strong to weak parts of a structure until a uniform distribution of damage (or distribution) is achieved. Therefore, it can be assumed that a status of uniform distribution of structural damage is a direct consequence of the optimum use of material. This criterion can be also used to assess how much a design solution is close to the optimum answer. In this study, by considering the shear storey-drift as a damage index, the following equation is used to evaluate the efficiency of different design solutions

$$Opt_{eff} = [1 - \mathrm{COV}_{\Delta sh}] \times 100 \tag{5}$$

where Opt_{eff} is the optimum closeness factor and $COV_{\Delta sh}$ is the Coefficient of Variation (COV) of maximum shear inter-storey drifts, which is calculated by the following formula

$$COV_{\Delta sh} = \frac{\sqrt{\dfrac{\sum_{i=1}^{n} [(\Delta_{sh})_i - (\Delta_{sh})_{ave}]}{n-1}}}{(\Delta_{sh})_{ave}} \tag{6}$$

$(\Delta_{sh})_{ave}$ is the average of maximum shear inter-storey drifts and n is the number of storeys. While the optimum closeness factor Opt_{eff} for an optimum structure tends to 100%, this ratio will decrease linearly with increasing the $COV_{\Delta sh}$. It should be mentioned that Opt_{eff} for structures with $COV_{\Delta sh} > 1$ is considered to be zero, which implies that these structures are far from optimum.

5. Optimum seismic design of braced steel frames

The objective of the optimization in this study is to obtain the best distribution of structural materials to exhibit minimum structural damage under a design earthquake. Since the maximum shear inter-storey drift, Max(Δ_{sh}) is considered as the performance-based design parameter, the objective function f can be formulated as

$$\text{Minimize:} \quad f(x) = Max(\Delta_{sh})_i \qquad (i = 1, 2, .., n) \tag{7}$$

where x is the design variable vector; $(\Delta_{sh})_i$ is the shear inter-storey drift of the i^{th} storey subjected to the design seismic excitation; and n is the number of storeys. For better comparison between the seismic behaviour of optimal and conventionally designed models, the total structural weight is kept constant during the optimization process. Therefore, the structural elements are chosen to satisfy the following design constraint

$$\text{Subject to:} \quad \sum_{i=1}^{n} (W_S)_i = W_0 \qquad (i = 1, 2, .., n) \tag{8}$$

$(W_s)_i$ is the structural weight of the i^{th} storey and W_0 is the total structural weight of the IBC-2012 design model.

In this study, a practical optimization method introduced by Moghaddam *et al*. (2005) and Hajirasouliha *et al*. (2012) is further developed for the optimum performance-based seismic design of CBFs. This method is based on the concept of uniform distribution of damage, where the distribution of structural properties is modified based on the response of equivalent shear-building models under a design earthquake. To avoid high computational costs and complexity, non-linear dynamic analyses are performed on a modified shear-building model that can account for both shear and bending displacements. Previous research by Hajirasouliha and Doostan (2010) demonstrated that this equivalent model can accurately estimate different performance parameters of braced steel frames, such as roof displacement, inter-storey drift and cumulative damage. The design variables in the optimization process are lateral stiffness and strength of storeys, which are directly linked to the size of structural elements. The optimization steps are summarized as follows:

(1) The initial structure is designed for gravity and seismic loads based on any modern design codes, e.g., IBC-2012.
(2) Pushover analysis is performed on the designed frame to obtain the non-linear properties (i.e., stiffness, strength and hardening coefficient) of the equivalent modified shear-building model at each storey (Hajirasouliha and Doostan 2010).
(3) The equivalent shear building is subjected to the design seismic excitation, and maximum shear inter-storey drifts $(\Delta_{sh})_i$ are obtained for all stories. The Opt_{eff} factor is then calculated based on Eq. (5). If Opt_{eff} is greater than the target threshold value (e.g., 95%), the uniform damage distribution is achieved and the structure is considered to be practically optimum. Otherwise, the optimization algorithm proceeds in iterations.

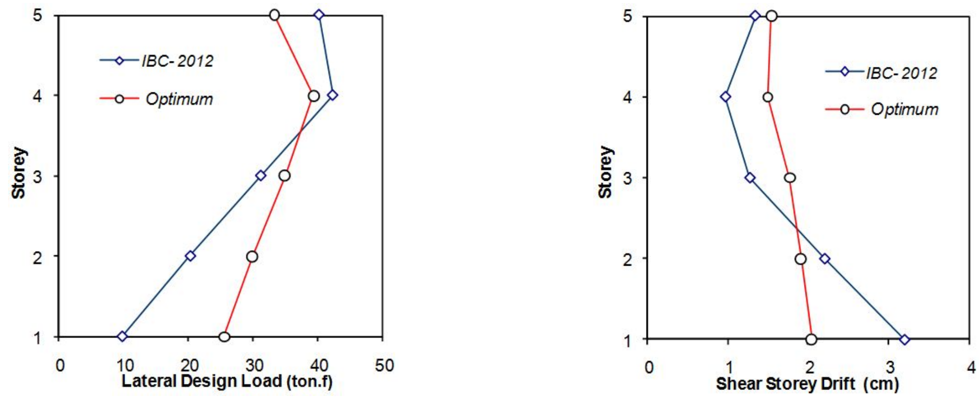
(4) Based on the distribution of maximum shear inter-storey drifts, the following equation is used for re-distribution of inefficiently used material in the structure

$$(S_i)_{j+1} = \left[\frac{(\Delta_{sh})_i}{\Delta_{sh(ave)}} \right]^{\alpha} (S_i)_j \tag{9}$$

where $(S_i)_n$ is the lateral shear strength of the i^{th} storey at j^{th} iteration; $(\Delta_{sh})_i$ and $(\Delta_{sh})_{ave}$ are shear inter-storey drift of the i^{th} storey and the average of shear inter-storey drifts of all stories, respectively. The power factor α is used as a convergence parameter, which is selected to be 0.2. The optimization procedure is then repeated from step 3 until the Opt_{eff} factor reaches the target value under the design earthquake. The seismic performance of such a structure is considered optimum as the material capacity is fully exploited, which indicates that the dissipation of seismic energy in each structural element is maximized.

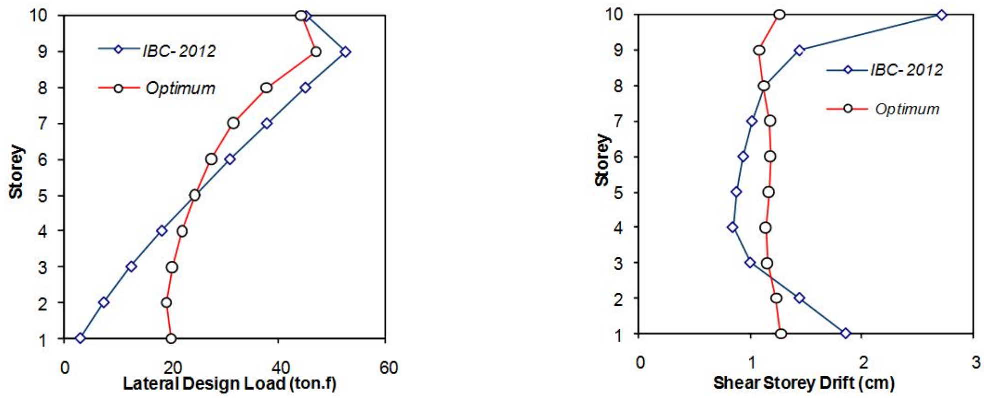
(5) Based on the shear strength distribution of the optimum structure, a new lateral seismic design load pattern is obtained. Using a simple iterative process, the optimum design load pattern is scaled to maintain the total structural weight W_0 (Moghaddam and Hajirasouliha 2008). Finally, the design of the initial structure is revised based on the new lateral load pattern to determine optimum structural sections.

In the current study, the optimum design of the 5, 10 and 15-storey braced frames (shown in Fig. 1) were obtained for the 15 strong ground motion records listed in Table 1. Subsequently, the seismic performance of the optimum and IBC-2012 designed frames were evaluated under each seismic excitation. The results indicate that, for similar structural weight, optimum structures always exhibit lower maximum shear inter-storey drifts (with more uniform distribution) compared to conventionally designed structures. For example, Fig. 4 compares the distribution of shear inter-storey drift and lateral seismic design loads for optimum and conventionally designed models subjected to Imperial Valley 1979 earthquake H-E04230 component. It is shown that improved seismic performance is achieved for all CBFs by using new load patterns different from the conventional IBC pattern. It should be mentioned that the optimum design load-patterns

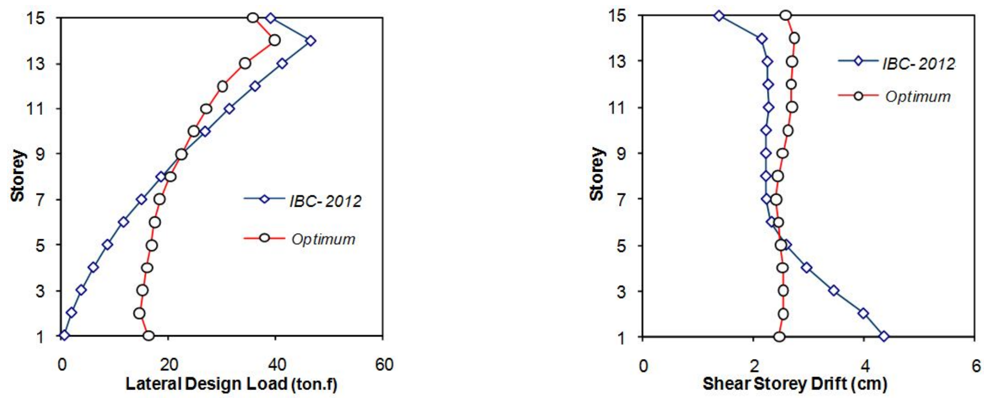


(a) 5-Storey concentrically braced frame

Fig. 4 Comparison of optimum models with conventionally designed models subjected to Imperial Valley 1979 earthquake (H-E04230)



(b) 10-Storey concentrically braced frame



(c) 15-Storey concentrically braced frame

Fig. 4 Continued

depend on the design earthquake and, therefore, may not lead to the optimum seismic performance under other seismic excitations. This is discussed in more detail in Section 7.

6. Effects of structural uncertainties

The main sources of structural uncertainty in building structures are due to the tolerances and variability in material properties, geometrical dimensions, damping characteristics as well as uncertainties associated with the definition of analytical models. In this section, the sensitivity of the optimum and conventionally designed CBFs to variations in storey shear strength and damping ratio is investigated by using the classical Monte Carlo simulation method.

6.1 Uncertainty in storey shear strength

In this study, any variation in storey shear strength is attributed to a combination of uncertainties in material properties, section size, imperfections and actual strength of connections. Most of these parameters can vary independently and, therefore, the storey shear strengths at different levels are assumed to be statistically independent. For each CBF considered, an ensemble of storey shear strengths is generated by using the following equation

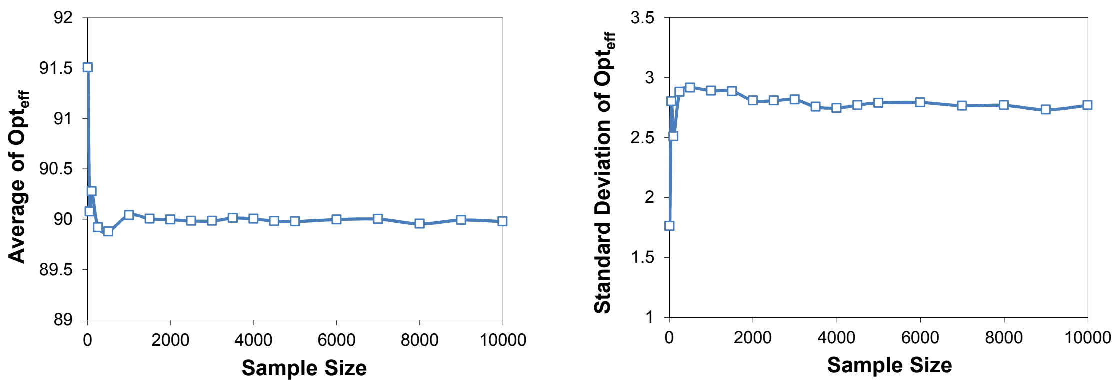
$$(Fs)_i = (Fs_0)_i + N_R \times \sigma_{Fs} = (Fs_0)_i \times (1 + N_R \times COV\ F_s) \tag{10}$$

where $(Fs)_i$ and $(Fs_0)_i$ are the randomised and the original shear strength of the i^{th} storey, respectively; N_R is a standard normal (or Gaussian) distributed random number; Δ_{Fs} and COV *Fs* are the expected Standard Deviation and Coefficient of Variation of storey shear strengths due to uncertainties in structural properties, respectively. By using Eq. (10), for each storey, the average of shear strengths over all samples is expected to be equal to the shear strength of the original building.

Using yield strength and section dimensional variability for typical building structures from Simoes da Silva *et al*. (2009), the expected *COV Fs* was found to be between 5-10%, which seems to be a realistic value for practical applications. However, to cover other modelling and construction uncertainties, *COV* Fs was varied from 1% to 20% in this study. It should be noted that the brace elements and the gusset-plate connections in this study were designed to prevent fracture under seismic loads and, therefore, the variability in the fracture capacity of braces was not taken into account. To investigate the effects of uncertainties on global seismic performance of CBFs, the global damage index D_g and the optimum closeness ratio Opt_{eff} were calculated for each set of frames.

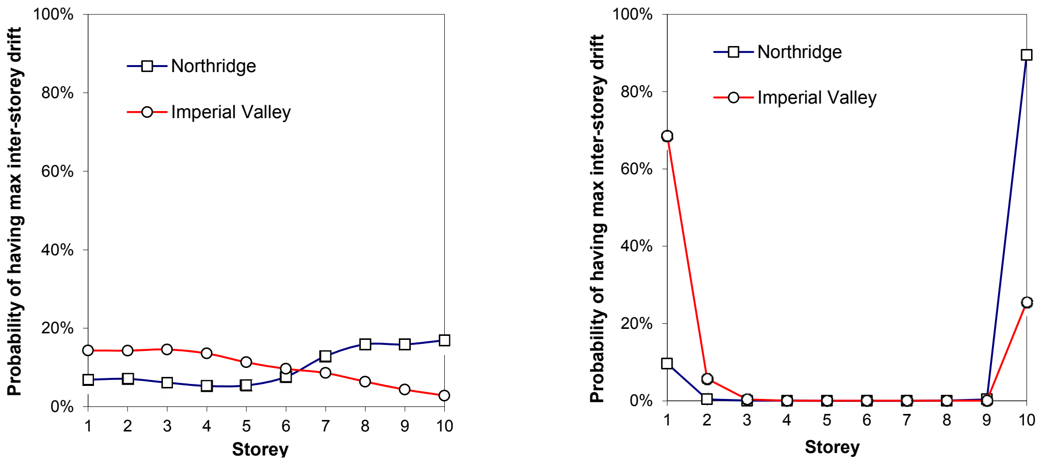
To ensure the reliability of the Monte Carlo estimations, the effect of sample size is investigated. Fig. 5 presents the average and the standard deviation of optimum closeness factor Opt_{eff} for a 10-storey optimum design model subjected to Northridge 1994 (NWH360) for various sample sizes. It is shown that a sample size of 5000 is reasonable (if not rather conservative) for an adequate estimation of the response statistics. Similar results were obtained for 5 and 15-storey structures. In this study, 5000 random models were generated for optimum and IBC-design structures at each uncertainty level (1,440,000 non-linear CBFs in total).

Fig. 6 shows a comparison between critical storeys (i.e., with maximum shear inter-storey drift) of the 10-storey (a) optimum; and (b) IBC-2012 design models, for the two earthquakes Northridge 1994 (NWH360) and Imperial Valley 1979 (H-E04230). The probability of being a critical storey is given by the percentage of the times that a particular storey exhibits the maximum



(a) Average of Opt_{eff} (b) Standard deviation of Opt_{eff}

Fig. 5 Effects of sample size on the response of 10-storey optimum design structure subjected to Northridge 1994 (NWH360), COV *Fs* = 5%



(a) Optimum design models (b) IBC-2012 design models

Fig. 6 Probability of being a critical storey, 10-storey structures subjected to Imperial Valley 1979 and Northridge 1994 earthquakes, *COV Fs* = 5%

shear inter-storey drift under the design earthquake. For the IBC-2012 models, it is shown in Fig. 6 that the critical storeys are always concentrated at the top and the bottom floors and, hence, the seismic capacity of the other floors is not fully exploited. The results indicate that the distribution of critical storeys for the optimum models is more uniform compared to the IBC-2012 design models. Therefore, based on the concept of uniform damage distribution, optimum CBFs are expected to exhibit better seismic performance under the design earthquakes compared to their code-based counterparts. This will be discussed in more detail in the following sections.

To investigate the impact of variations in storey shear strength on the seismic performance of CBFs, the optimum closeness factor Opt_{eff} was determined for both optimum and IBC-2012 design models subjected to the 15 strong ground motions listed in Table 1, using COV Fs of 1%, 3.33%, 5%, 7%, 10% and 20%. Fig. 7 compares the average and average minus standard deviation of Opt_{eff} for the optimum and IBC design 5, 10 and 15 storey models subjected to Northridge 1994 (NWH360) and Imperial Valley 1979 (H-E04230) earthquakes.

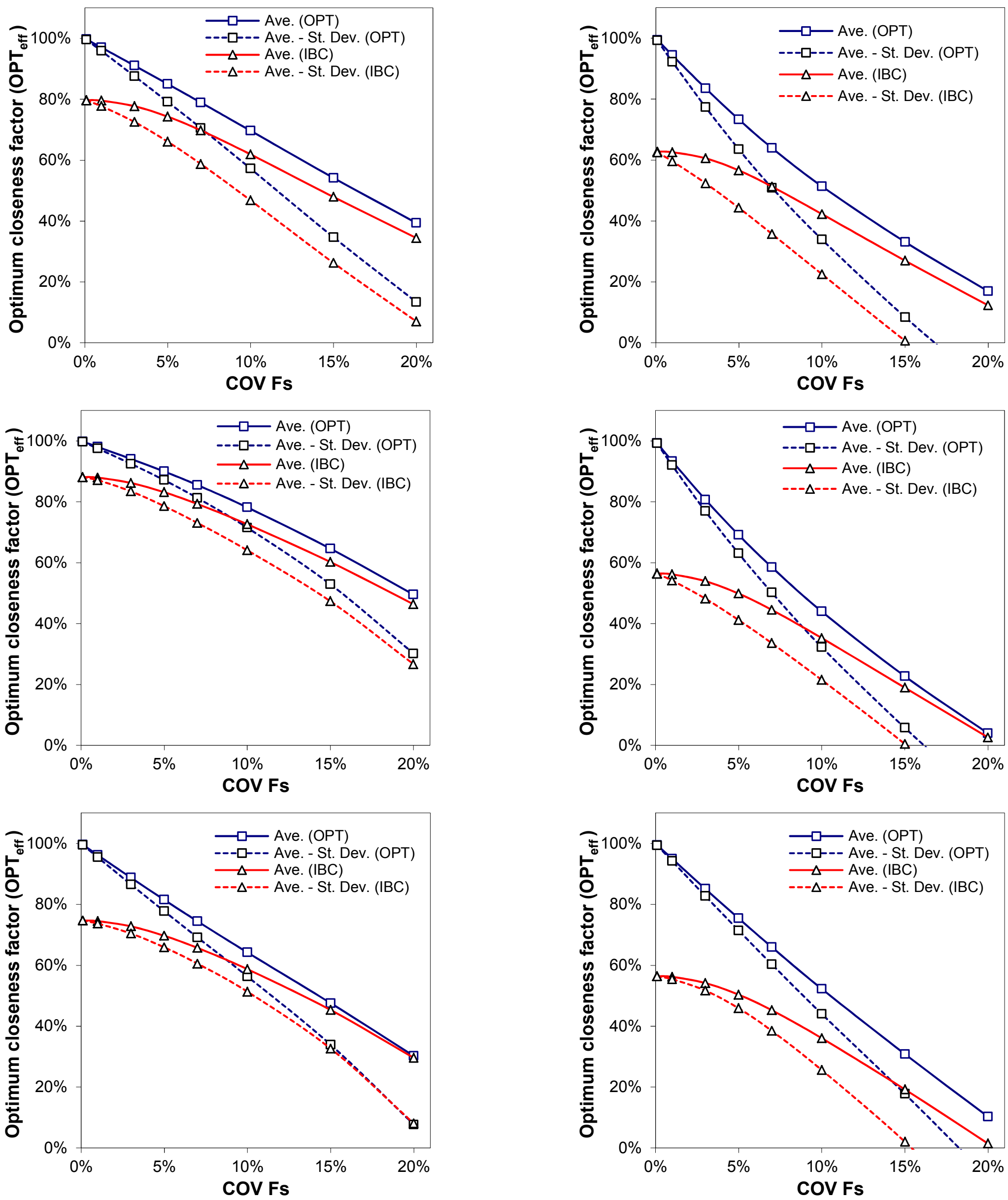
Although the efficiency of both optimum and IBC design structures decreases with increasing *COV Fs*, in general, optimum designed structures appear to be more sensitive to small variations in storey strengths. At extremely high *COV Fs* (e.g., 20%), there is no substantial difference between the seismic performance of optimum and IBC design structures. However, for practical applications where *COV Fs* is expected to be less than 10%, optimum design structures always exhibit more uniform shear inter-storey drift distribution and, therefore, considerably higher optimum closeness Opt_{eff}. In this study, the following equation is used to quantify the sensitivity of the optimum and code-based CBFs to small variations in storey-shear strength

$$\text{Sensitivity Factor:} \quad SF = \frac{\Delta Opt_{eff}}{\Delta COV\ F_s} \tag{10}$$

The sensitivity factor (SF) captures the drop from the optimum solution as the variability of storey shear increases. Table 2 shows The average sensitivity of the optimum and IBC design 5, 10

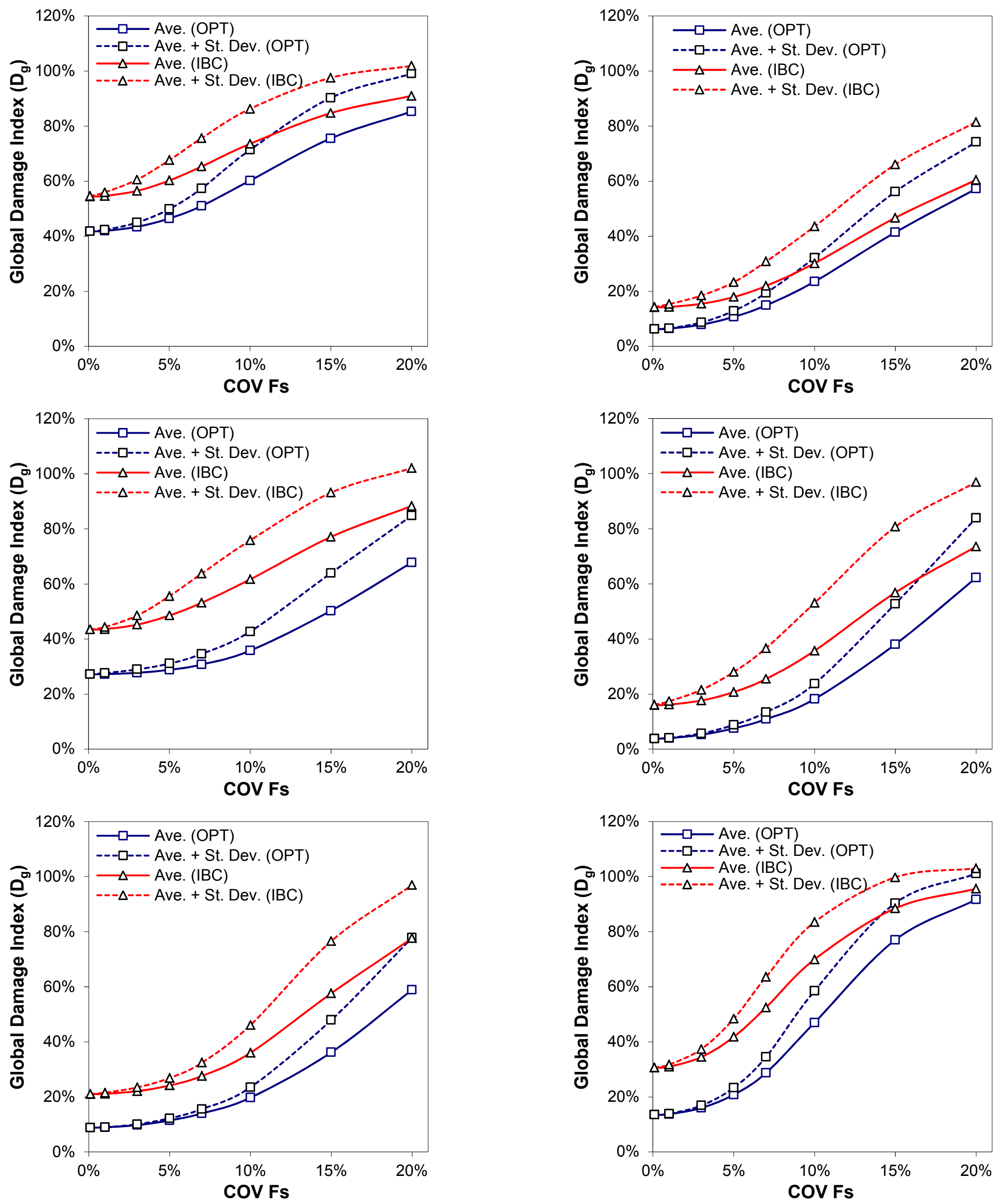
Table 2 Average sensitivity factor (SF) of optimum and IBC design CBFs

CBF	Northridge 1994 (NWH360)		Imperial Valley 1979 (H-E04230)	
	IBC-2012	Optimum	IBC-2012	Optimum
5- Storey	1.89	3.02	2.13	4.56
10- Storey	1.72	2.34	2.26	5.27
15- Storey	1.83	3.55	2.25	4.64



(a) Northridge 1994 (NWH360) (b) Imperial Valley 1979 (H-E04230)

Fig. 7 Optimum closeness factor of optimum and IBC-2012 design frames with variation of COV Fs

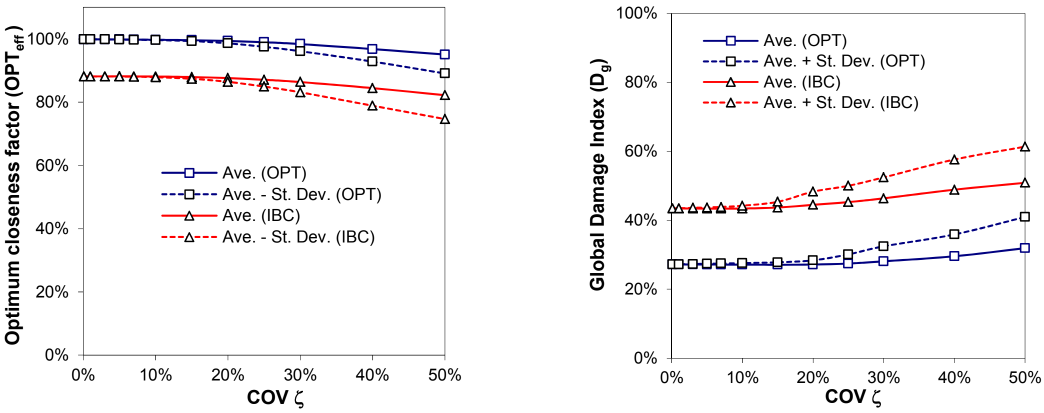


(a) Northridge 1994 (NWH360) (b) Imperial Valley 1979 (H-E04230)

Fig. 8 Global damage index (Dg) of optimum and IBC-2012 design frames with variation of *COV Fs*

and 15 storey models subjected to Northridge 1994 (NWH360) and Imperial Valley 1979 (H-E04230) earthquakes. The results indicate that the sensitivity factors of an optimum design CBF can be more than twice that of its code-based counterpart. Similar results were obtained by using the other earthquake records listed in Table 1.

The same investigation was repeated for the global damage index (*Dg*) and the results are



(a) Optimum closeness factor (Opt_{eff}) (b) Global damage index (Dg)

Fig. 9 Effect of random variation in damping ratio on seismic performance of optimum and IBC-2012 design structures, 10-storey frame subjected to Northridge 1994 earthquake

shown in Fig. 8. The results indicate that, for the same structural weight, the optimum structures experienced on average 40% less global damage compared to IBC design models. As expected, the global damage index of both optimum and code-based design models increases with increasing the *COV Fs*, but in this case *Dg* is rather insensitive to small variations in storey shear strengths (i.e., *COV Fs* ≤ 5%). However, as in the previous case, very high variability in storey shear strength results in excessive damage for both optimum and IBC-2012 designs. For normal variabilities (i.e., *COV Fs* between 5-10%), the optimum design structures exhibited at least 20% less global damage compared to their code-based counterparts; which means the adopted optimization method always led to a better seismic performance under the design earthquakes.

6.2 Uncertainty in damping ratio

Damping is one of the major sources of uncertainty in building structures. To investigate the effect of variation in damping ratio on the seismic performance of optimum and IBC design models, an ensemble of damping ratios ξ was generated for each CBF by using the following equation

$$\xi = \xi_0 \times (1 + N_R \times COV\xi) \qquad (11)$$

where ξ and ξ_0 are the randomised and the original damping ratio, respectively; N_R is a standard normal (or Gaussian) distributed random number; and $COV\ \zeta$ is the expected Coefficient of Variation of damping ratios. In this study, the original damping ratio ξ_0 was considered to be 5%. Fig. 9 shows the changes in optimum closeness factor and global damage index due to variations in damping ratio. It is shown that the effect of variability in damping on the seismic performance of CBFs is similar for optimum and IBC design models. The results indicate that, on average, up to 50% random changes in damping ratio (i.e., $COV\ \zeta < 50\%$) do not considerably affect the optimum closeness factor and the global damage index of the CBFs. This shows that, in general, uncertainty in damping ratio does not affect the efficiency of the optimisation method.

7. Uncertainties in earthquake ground motion

The seismic ground motion is the main source of uncertainty in the seismic design of structures. There is a concern that this may influence the efficiency of the optimum structures that are designed based on a single earthquake event. To manage the uncertainty in the design seismic loads, previous studies by Hajirasouliha and Pilakoutas (2012) suggested that shear-building models could be designed by averaging the optimum patterns corresponding to a set of real or synthetic earthquakes representing a design spectrum. This concept is adopted for the first time in this study for optimum seismic design of CBFs.

7.1 Optimum design based on a design spectrum

To design CBFs based on IBC-2012 design spectrum, ten synthetic spectrum-compatible earthquakes corresponding to soil class D (see Section 2) were considered. For each synthetic ground motion record, the optimum design load patterns (ratio of storey lateral design force to base shear force) were obtained. Fig. 10 compares the average of the ten optimum load patterns and the IBC-2012 load pattern for the 10-storey frame.

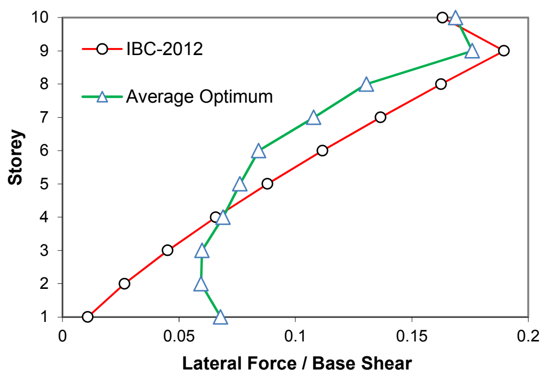
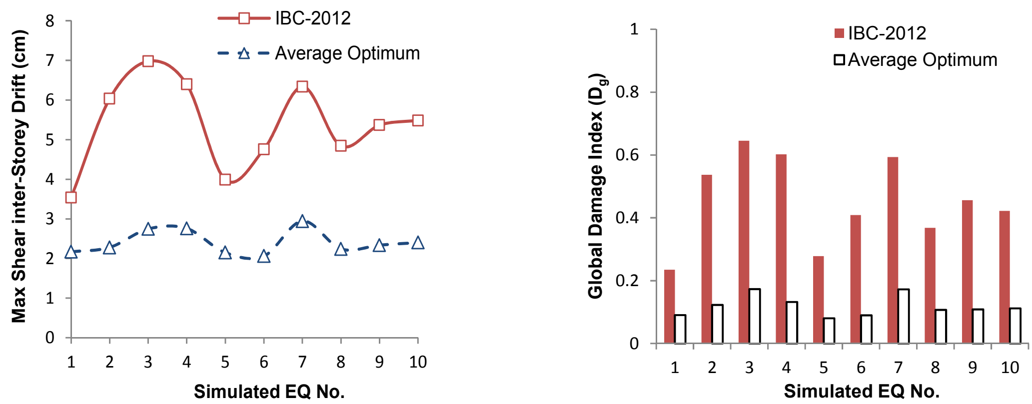


Fig. 10 Comparison between average of optimum design load patterns and IBC-2012 design load



(a) Maximum shear inter-storey drift

(b) Global damage index (*Dg*)

Fig. 11 Seismic performance of 10-storey CBFs designed with the average optimum and IBC-2012 load patterns under 10 synthetic spectrum-compatible earthquakes representing the design spectrum

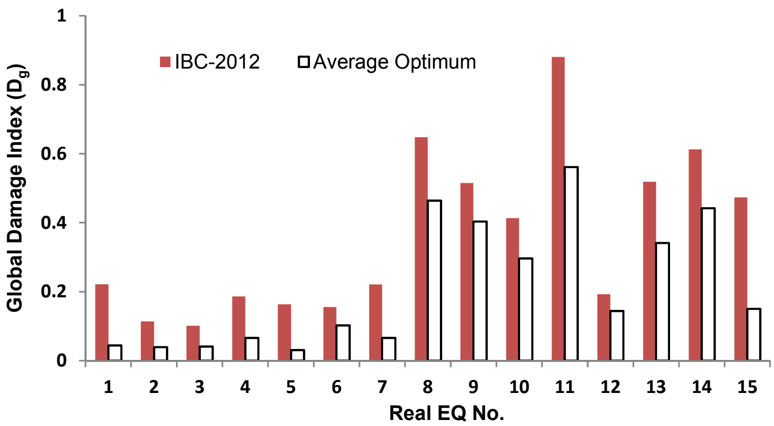


Fig. 12 Comparison between global damage index (*Dg*) of CBFs designed with IBC-2012 and average optimum load patterns in 15 real earthquakes listed in Table 1

To account for seismic load uncertainty, the CBFs were designed with the average of optimum load patterns. The new frames were then subjected to each of the ten synthetic earthquakes, and the maximum shear inter-storey drift and the global damage index, *Dg*, were determined. Fig. 11 compares the resulting seismic performance for the 10-storey braced frames under each synthetic earthquake. The results indicate that structures designed with the average optimum pattern experienced on average 54% less maximum shear inter-storey drift and 73% less cumulative damage compared to the similar code-based design structures.

7.2 Efficiency of the optimum solutions for real earthquakes

To investigate the efficiency of the adopted optimisation method in practical applications, the CBFs designed with the average of optimum load patterns (corresponding to the synthetic spectrum-compatible earthquakes) were subjected to the 15 real seismic excitations listed in Table 1. As mentioned before, these seismic excitations are recorded on a similar soil type as the design spectrum (soil class D of IBC-2012). The results presented in Fig. 12 show that the CBFs designed with the average load pattern result in considerably improved seismic performance compared to similar code-based design frames, exhibiting at least 20% less cumulative damage during real seismic excitations. This shows that seismic load uncertainty can be efficiently managed by using the average of optimum load patterns for a group of synthetic earthquakes representing the design spectrum.

8. Conclusions

In this study, a practical optimisation method based on the concept of uniform distribution of damage was adapted to obtain the optimum seismic design of 5, 10 and 15-storey concentrically braced frames (CBFs). By using the Monte Carlo simulation method, the effects of uncertainty in the storey shear strength, damping ratio and design earthquake were investigated. Based on the results presented in this paper, the following conclusions can be drawn:

- For typical uncertainties in conventional steel frames, optimum design frames always exhibit more uniform inter-storey drift distribution. However, the efficiency of both optimum and IBC design structures decreases with increasing Coefficient of Variation of storey shear strengths, and optimum designed structures are in general more sensitive (increase of average SF by a factor of 2) to small variations in the storey strengths. This means that the optimum solutions are affected twice as much by increased variability in storey shear.

- Although increasing the Coefficient of Variation of storey shear strengths (*COV Fs*) increases the global damage index of both optimum and IBC design CBFs, the efficiency of the optimum design solutions may significantly reduce in the case of high structural variability (i.e., *COV Fs* > 15%). However, for typical buildings with expected structural variabilities between 5-10%, the optimum design structures always exhibit at least 20% less global damage compared to their code-based counterparts.
- Up to 50% random changes in damping ratio (i.e., *COV* ζ = 50%) do not considerably affect the seismic performance of optimum design CBFs.
- It is shown that seismic load uncertainty can be efficiently managed in CBFs by using the average of optimum load patterns for a set of synthetic earthquakes representing a specific design spectrum. The results indicate that the CBFs designed with the average load pattern exhibited up to 54% less maximum shear inter-storey drift and 73% less cumulative damage compared to code-based design counterparts. This conclusion was further confirmed by using real seismic excitations.

References

ANSI/AISC 341-05 (2005), Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Chicago, IL, USA.

ANSI/AISC 360-05 (2005), Specification for Structural Steel Buildings, American Institute of Steel Construction, Inc., Chicago, IL, USA.

ASCE 7-05 (2006), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE), Reston, VA, USA.

ASCE/SEI 41-13 (2014), Seismic Rehabilitation of Existing Buildings, (1st Edition), American Society of Civil Engineers.

Baik, S.W., Lee, D.G. and Krawinkler, H. (1988), "A simplified model for seismic response prediction of steel frame structures", *Proceedings of the 9th World Conference on Earthquake Engineering*, Volume 5, Tokyo-Kyoto, Japan, August.

Beck, J.L., Chan, E., Irfanoglu, A. and Papadimitriou, C. (1999), "Multi-criteria optimal structural design under uncertainty", *Earthq. Eng. Struct. Dyn.*, **28**(7), 741-761.

Bertero, V.V., Anderson, J.C., Krawinkler, H. and Miranda, E. (1991), Design guidelines for ductility and drift limits; Report No. UCB/EERC-91/15, University of California, Earthquake Eng Center, Berkeley, CA, USA.

Broderick, B.M., Elghazouli, A.Y. and Goggins, J. (2008), "Earthquake testing and response analysis of concentrically-braced sub-frames", *J. Construct. Steel Res.*, **64**(9), 997-1007.

Chopra, A.K. (2012), *Dynamics of Structures*, (4th ed.), Prentice Hall Inc., London, UK.

Dicleli, M. and Calik, E.E. (2008), "Physical theory hysteretic model for steel braces", *J. Struct. Eng. ASCE*, **134** (7), 1215-1228.

DIN 1025 (1995), Hot rolled I and H sections: Dimensions, mass and static parameters, DIN Deutsches Institut Fur Normung EV, Berlin, Germany.

Fu, G. and Frangopol, D.M. (1990), "Reliability-based vector optimization of structural systems", *J. Struct. Eng. ASCE*, **116**(8), 2141-61.

Hajirasouliha, I. and Doostan, A. (2010), "A simplified model for seismic response prediction of concentrically braced frames", *Adv. Eng. Software*, **41**(3), 497-505.
Hajirasouliha, I. and Moghaddam, H. (2009), "New lateral force distribution for seismic design of structures", *J. Struct. Eng. ASCE*, **135**(8), 906-915.
Hajirasouliha, I. and Pilakoutas, K. (2012), "General Seismic Load Distribution for Optimum Performance-Based Design of Shear-Buildings", *J. Earthq. Eng.*, **16**(4), 443-462.
Hajirasouliha, I., Asadi, P. and Pilakoutas, K. (2012), "An efficient performance-based seismic design method for reinforced concrete frames", *Earthq. Eng. Struct. Dyn.*, **41**(4), 663-679.
Hart, G.C. (2000), "Earthquake forces for the lateral force code", *Struct. Des. Tall Build.*, **9**(1), 49-64.
Haukaas, T. and Kiureghian, A.D. (2003), Finite element reliability and sensitivity methods for performance-based engineering, Report No. PEER 2003/14, Pacific Earthquake Eng Research Center, University of California, Berkeley, CA, USA.
Hsiao, P.C., Lehman, D.E., Berman, J.W., Roeder, C.W. and Powel, J. (2014), "Seismic vulnerability of older braced frames", *J. Perform. Construct. Facil. ASCE*, **28**(1), 108-120.
IBC (2012), International Building Code, International Code Council, Country Club Hills, USA.
Jain, A.K., Goel, S.C. and Hanson, R.D. (1980), "Hysteretic cycles of axially loaded steel members", *J. Struct. Div. ASCE*, **106**(8), 1777-1795.
Jazany, R.A., Hajirasouliha, I. and Farshchi, H. (2013), "Influence of masonry infill on the seismic performance of concentrically braced frames", *J. Construct. Steel Res.*, **88**, 150-163.
Karami Mohammad, R. and Sharghi, H. (2014), "On the optimum performance-based design of eccentrically braced frames", *Steel Compos. Struct., Int. J.*, **16**(4), 357-374.
Karami Mohammadi, R., El Naggar, M.H. and Moghaddam, H. (2004), "Optimum strength distribution for seismic resistant shear buildings", *Int. J. Solid. Struct.*, **41**(22-23), 6597-6612.
Kazantzi, A.K., Vamvatsikos, D. and Lignos, D.G. (2014), "Seismic performance of a steel moment-resisting frame subject to strength and ductility uncertainty", *Eng. Struct.*, **78**, 69-77.
Koboevic, S.M., Rozon, J. and Tremblay, R. (2012), "Seismic performance of low-to-moderate height eccentrically braced steel frames designed for North American seismic conditions", *J. Struct. Eng. ASCE*, **138**(12), 1465-1476.
Krawinkler, H. and Zohrei, M. (1984), "Cumulative damage in steel structures subjected to earthquake ground motions", *Comput. Struct.*, **16**(1-4), 531-41.
Kwon, O.S. and Elnashai, A. (2006), "The effect of material and ground motion uncertainty on the seismic vulnerability curves of RC structure", *Eng. Struct.*, **28**(2), 289-303.
Lagaros, N.D., Garavelas, A.T. and Papadrakakis, M. (2008), "Innovative seismic design optimization with reliability constraints", *Comput. Method. Appl. Mech. Engrg.*, **198**(1), 28-41.
Lee, K. and Foutch, D.A. (2002), "Performance evaluation of new steel frame buildings for seismic loads", *Earthq. Eng. Struct. Dyn.*, **31**(3), 653-670.
Liu, M., Burns, S.A. and Wen, Y.K. (2005), "Multiobjective optimization for performance-based seismic design of steel moment frame structures", *Earthq. Eng. Struct. Dyn.*, **34**(3), 289-306.
McCrum, D.P. and Broderick, B.M. (2013), "An experimental and numerical investigation into the seismic performance of a multi-storey concentrically braced plan irregular structure", *Bull. Earthq. Eng.*, **11**(6), 2363-2385.
Moghaddam, H. and Hajirasouliha, I. (2006), "Toward more rational criteria for determination of design earthquake forces", *Int. J. Solid. Struct.*, **43**(9), 2631-2645**.**
Moghaddam, H. and Hajirasouliha, I. (2008), "Optimum strength distribution for seismic design of tall buildings", *Struct. Des. Tall Special Build.*, **17**(2), 331-349.
Moghaddam, H., Hajirasouliha, I. and Doostan, A. (2005), "Optimum seismic design of concentrically steel braced frames: Concepts and design procedures", *J. Construct. Steel Res.*, **61**(2), 151-166.
Papadrakakis, M., Lagaros, N.D. and Plevris, V. (2005), "Design optimization of steel structures considering uncertainties", *Eng. Struct.*, **27**(9), 1408-1418.
Prakash, V., Powell, G.H., and Filippou, F.C. (1992), DRAIN-2DX: Base program user guide; UCB/ SEMM - 92/29, Earthquake Engineering Research Centre, University of California, Berkeley, CA, USA.

Priestley, M.J.N., Calvi, M.C. and Kowalsky, M.J. (2007), *Displacement-based Seismic Design of Structures*, IUSS Press, Pavia, Italy.

Simoes da Silva, L., Rebelo, C., Nethercot, D., Marques, L., Simoes, R. and Vila Real P.M.M. (2009), "Statistical evaluation of the lateral-torsional buckling resistance of steel I-beams, Part 2: Variability of steel properties", *J. Construct. Steel Res.*, **65**(4), 832-849.

Vanmarke, E.H., Fenton, G.A. and Heredia-Zavoni, E. (1999), SIMQKE-II, Conditioned earthquake ground motion simulator: User's manual; Version 2.1, Pacific Earthquake Engineering Research (PEER) Center, University of California, Berkeley, CA, USA.

Yousuf, M. and Bagchi, A. (2009), "Seismic design and performance evaluation of steel-frame buildings designed using the 2005 National Building code of Canada", *Can. J. Civil Eng.*, **36**(2), 280-294.

Zacharenaki, A., Fragiadakis, M. and Papadrakakis, M. (2013), "Reliability-based optimum seismic design of structures using simplified performance estimation methods", *Eng. Struct.*, **52**(1), 707-717.

DL