Article:

Hajirasouliha, I. and Pilakoutas, K. (2012) General Seismic Load Distribution for Optimum Performance-Based Design of Shear-Buildings. Journal of Earthquake Engineering, 16 (4). 443 - 462. ISSN 1363-2469

https://doi.org/10.1080/13632469.2012.654897

Hajirasouliha I, Pilakoutas K (2012) General seismic load distribution for optimum performance-based design of shear-buildings. Journal of Earthquake Engineering, 16(4), 443-462.

General seismic load distribution for optimum performance-based design of shear-buildings

Iman Hajirasouliha [1]

Department of Civil Engineering, The University of Nottingham, Nottingham, UK

Kypros *Pilakoutas*

Department of Civil & Structural Engineering, The University of Sheffield, Sheffield, UK

Abstract

An optimisation method based on uniform damage distribution is used to find optimum design load distribution for seismic design of regular and irregular shear-buildings to achieve minimum structural damage. By using 75 synthetic spectrum-compatible earthquakes, optimum design load distributions are obtained for different performance targets, dynamic characteristics and site soil classifications. For the same structural weight, optimum designed buildings experience up to 40% less global damage compared to code-based designed buildings. A new general load distribution equation is presented for optimum performance-based seismic design of structures which leads to a more efficient use of structural materials and better seismic performance.

Key words: Optimisation; Performance-based seismic design; Irregular structures; Lateral forces; Site soil classification; Ductility.

[1] Address correspondence to Iman Hajirasouliha, Department of Civil Engineering, University of Nottingham, University Park, Nottingham NG7 2RD, UK; E-mail: i.hajirasouliha@nottingham.ac.uk, Tel: +44 (0) 115 951 3922 Fax: +44 (0) 115 951 3898

1. Introduction

The preliminary design of most buildings is normally based on equivalent static forces obtained from seismic design guidelines and codes of practice. The height-wise distribution of these static forces is implicitly based on the first-mode dynamic response of elastic structures [Hart, 2000; Chopra, 2001]. As structures exceed their elastic limits in severe earthquakes, the use of inertia forces corresponding to the elastic modes may not lead to the uniform distribution of ductility demands [Chopra, 2001]. The seismic behaviour of different code-based designed structural systems have been extensively investigated over the last two decades [Anderson et al., 1991; Gilmore and Bertero, 1993; Martinelli et al., 2000; Lee and Foutch, 2002; Goulet et al., 2007]. The results of these studies showed that, in general, buildings designed based on new seismic design guidelines satisfy the collapse prevention and immediate occupancy performance levels. However, the lateral load distribution used by current seismic design guidelines does not always lead to the optimum use of structural materials [Chopra, 2001; Moghaddam and Hajirasouliha, 2006].

In an attempt to find optimum distribution of structural properties, Takewaki (1996, 1997) developed an analytical method to find stiffness (and strength) distribution that leads to a constant storey-ductility demand for a shear-building structure subjected to a given design spectrum. This method is based on an elastic equivalent linearization technique, and the results showed that for tall buildings it does not lead to a uniform ductility demand distribution when the structure is subjected to a time-history excitation. Gantes et al. (2000) used the Euler–Bernoulli beam theory to find optimum bending and shear distribution of multi-storey steel frames to obtain uniform height-wise drift distribution. Although their proposed method is simple and practical, it is not capable of considering the non-linear behaviour of structures.

Lee and Goel (2001) and Chao et al. (2007) analyzed a series of steel moment and braced frames subjected to various earthquake excitations. They showed that in general there is a discrepancy between the earthquake induced shear forces and the forces determined by assuming code-based design load distribution patterns. Based on the results of their studies, they suggested a new lateral force distribution for seismic loads to address the influence of increasing higher mode effects in the inelastic state. However, the effects of ground motion characteristics and the degree of nonlinearity were not considered in their suggested load distribution.

Moghaddam and Hajirasouliha (2006) and Hajirasouliha and Moghaddam (2009) developed an effective optimisation method to find optimum lateral load distribution for seismic design of regular shear-building structures to obtain uniform storey ductility. They showed that, for the same target storey-ductility demand, structures designed with the average of optimum load patterns for a set of earthquakes with similar characteristics, have relatively lower structural weight compared to those designed conventionally. This lead to the following equation for preliminary design of height-wise regular shear-buildings based on the results of twenty earthquake excitations recorded on soft rock:

$$\phi_i = \frac{K_i}{\sum_{j=1}^{n} K_j}, \tag{1}$$

$$K_i = (a_i T + b_i)\mu_t^{(\frac{c_i T + d_i}{100})}$$

where ϕ_i is the ratio of optimum design force at i[th] storey to the base shear for a regular structure with fundamental period of T and maximum ductility demand of μ_t . a_i, b_i, c_i, and d_i are constant coefficients at i[th] storey that should be calculated for each set of design earthquakes.

The above load pattern is a function of structural performance level (i.e. storey ductility), and therefore, is suitable for performance-based seismic design of structures. However, the load pattern adopted cannot be used directly in practical design of structures, as the utilized seismic records were not compatible with modern building code design spectra (such as Eurocode 8 and IBC-2009), and the effects of height-wise irregularity and site soil profile were not considered in the above equation. This may lead to structures with unacceptable seismic performance under design spectrum-compatible earthquakes, and hence, needs further development.

In this study, the above mentioned optimisation method is further developed to optimize both regular and height-wise irregular structures to exhibit minimum structural damage under a design spectrum. By using 200 shear-building models subjected to 75 synthetic earthquakes compatible with

IBC-2009 design spectra, the effects of using different damage criteria, height-wise irregularity and site soil classification are investigated. Based on the results of this study, a more realistic lateral design load distribution is proposed and its efficiency is assessed by using a design example.

2. Modelling and assumptions

In spite of some drawbacks, shear-building models have been widely used to study the seismic response of multi-storey buildings [Diaz et al., 1994] because of simplicity and low computational effort that enables a wide range of parametric studies. In shear-building models, each floor is assumed to be a lumped mass that is connected by perfect elastic-plastic springs which only have shear deformations when subjected to lateral forces as shown in Fig. 1. All parameters required to define a shear-building model corresponding to a full-frame model can be determined by performing a single pushover analysis [Hajirasouliha and Doostan, 2010]. The shear-building model is capable of considering both non-linear behaviour and higher mode effects for the first few effective modes, and therefore, can represent well the actual behaviour of several types of multi-storey buildings [Diaz et al., 1994]. In shear-building models, the strength of each floor is obtained from the corresponding storey shear force, and therefore, the height-wise distribution of storey strength can be easily converted to the height-wise distribution of lateral forces. This makes shear-buildings a very suitable model for calculating the optimum seismic design load pattern for multi-storey structures with different dynamic characteristics and performance targets. In the present study, 200 regular and irregular 5, 10 and 15-storey shear-building models with fundamental period ranging from 0.1 sec to 3 sec, and maximum ductility demand equal to 1, 1.5, 2, 3, 4, 5, 6 and 8 are utilized. Prior studies by Hajirasouliha and Moghaddam (2009) showed that, for a specific fundamental period and target ductility demand, optimum load pattern can be considered independent of number of stories. Therefore, 5, 10 and 10 storey shear-building models with different fundamental periods can be representative of a wider range of structural systems. The range of the fundamental period considered in this study is wider than that of real structures to cover special cases. The Rayleigh damping model with a constant damping ratio of 0.05 is assigned to the first mode and to the first mode at which the cumulative mass participation exceeds 95%. To predict the seismic response of the shear-building models, nonlinear time-history analyses were carried out using computer program DRAIN-2DX [Parakash et al., 1992].

To investigate the effects of different soil profiles on the optimum design load distributions, five sets of spectrum-compatible synthetic earthquakes were generated using the SIMQKE program [Vanmarke, 1976] to represent the elastic design response spectra of IBC-2009 (and ASCE 7-05) soil types A, B, C, D and E (Table 1). These design response spectra are assumed to be an envelope of the many possible ground motions that could occur at the site. To include the ground motion variability, each set of synthetic earthquakes consists of 15 generated seismic excitations, with a PGA of 0.4g. It is shown in Fig. 2 that the average acceleration response spectrum of each set of synthetic earthquakes compares well with its corresponding IBC-2009 design spectrum.

3. Optimum seismic load distribution for a design earthquake

In this study, the optimisation target is to obtain a seismic design load that leads to minimum structural damage (i.e. optimum distribution of structural material) using a fixed amount of structural material. In shear-building structures, any increase in structural material is normally accompanied by an increase in storey strength. Therefore, total structural weight could be considered proportional to the sum of all storey shear strengths. Consequently, the storey shear strength can be considered as a design variable to optimise the seismic behaviour of shear-building structures.

3.1. Optimisation methodology

During strong earthquakes the deformation demand (that is corresponding to structural and non-structural damage) in code-based designed structures is not expected to be uniform [Chopra, 2001]. As a result, in some parts of the structures the maximum level of seismic capacity is not necessarily utilized. If the strength of underused elements is decreased incrementally, for a ductile structure, it is expected to eventually obtain a status of uniform damage distribution. In such a case, the dissipation of seismic energy in each structural element is maximized and the material capacity is fully exploited. Therefore, in general, it can be assumed that a status of uniform distribution of structural damage is a direct consequence of the optimum use of material. The optimisation of a non-linear structure subjected to a dynamic excitation is a complex problem; however, the use of the concept of uniform damage distribution simplifies the mathematics of the optimisation algorithm to a large extent. In the present study, in an attempt to reach uniform damage distribution through the structure, the following optimisation procedure is adopted:

1- The initial structure is designed for seismic loads based on a design guidelines, such as IBC-2009. The distribution of storey shear strength along the structure is then determined.

2- A model of the structure is subjected to the design seismic excitation, and a suitable local damage index (such as storey ductility, inter-storey drift and cumulative damage) is calculated for all stories.

3- The Coefficient of Variation (COV) of damage indices of all stories is calculated. If this COV is small enough (e.g. less than 0.1), the structure is considered to be practically optimum. Otherwise, the optimisation algorithm proceeds to iterations.

4- During the iterations, the distribution of storey shear strength is modified. The shear strength is reduced in the stories with lower-than-average damage index and increased in the stories which experienced higher-than-average damage. To obtain convergence in numerical calculations, this alteration needs to be applied incrementally using the following equation:

$$(S_i)_{n+1} = \left[\frac{DI_i}{DI_{ave}} \right]^{\alpha} (S_i)_n \tag{2}$$

where $(S_i)_n$ is the shear strength of the i^{th} storey at n^{th} iteration, DI_i and DI_{ave} are damage index for the i^{th} storey and average of damage indices for all stories, respectively. α is convergence parameter ranging from 0 to 1. Analyses carried out in this study on different models and seismic excitations indicated that an acceptable convergence is usually obtained by using α values of 0.1 to 0.2. The results presented in this paper are based on α value of 0.15.

5- The shear strength of all stories are scaled such that the sum of storey shear strengths (and structural weight) remains unchanged. The optimisation procedure is then repeated from step 2 until the COV of damage indices become small enough. The final solution is considered to be practically optimum. Analyses carried out by the authors showed that the optimum distribution of storey shear strengths is independent of the seismic load distribution used for initial design.

The concept of uniform damage distribution can also be used to find the optimum distribution of storey shear strengths for a specific performance target (DI_{target}). In this case, the equation (2) in the optimisation process should be replaced by the following equation, and there is no need to scale the sum of storey shear strengths in the step 5.

$$(S_i)_{n+1} = \left[\frac{DI_i}{DI_{target}} \right]^{\alpha} (S_i)_n \tag{3}$$

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 proposed method can optimise the design using different types of performance parameters as discussed in the following sections.

3.2. Minimum storey ductility

Storey ductility has been widely used to assess the level of damage in non-linear shear-building structures (Chopra 2001). In this section, storey ductility is considered as the damage index in the optimisation process (Equation 2). To show the efficiency of the proposed method, the above optimisation algorithm is used for the optimum design of a 10-storey shear-building with fundamental period of 1.1 sec subjected to a ground motion recorded at the Canoga Park station in the Northridge earthquake 1994 (CNP196).

Based on the concept of uniform damage distribution, the proposed optimisation method is expected to lead to a structure with minimum storey ductility (selected response parameter). Fig. 3 shows the variation of maximum storey ductility and COV of storey ductility demands from IBC-2009 to optimum designed model. It is shown that decreasing the COV was always accompanied by reduction of maximum storey ductility, and the proposed method practically converged to the optimum solution in less than 7 steps without any fluctuation.

Fig. 4-a compares the storey ductility distribution of IBC-2009 and optimum designed model (minimum storey ductility). It is shown that the proposed optimisation method resulted in a design with an almost perfectly uniform storey ductility distribution. The results indicate that, for the same structural weight, the optimum designed structure experienced 52% less maximum storey ductility (i.e. less structural damage) compared to the conventionally designed structure.

As it is mentioned before, the final height-wise distribution of storey strength in a shear-building model can be easily converted to the height-wise distribution of design lateral forces. Such pattern may be regarded as the optimum distribution pattern for seismic design forces (Opt_{eq}). The lateral force distribution of IBC-2009 and optimum designed models are compared in Fig. 4-b. The results indicate that to improve the seismic performance under this specific earthquake, the above mentioned model should be designed based on an equivalent lateral load distribution different from one used by conventional code patterns. However, this optimum load distribution may not be suitable for other cases as it depends on the characteristics of the structure and seismic excitation.

3.3. Minimum cumulative damage

To investigate the effect of selected damage criteria on the optimum design load pattern, the damage index proposed by Baik et al. (1988) based on the classical low-cycle fatigue approach is used in the optimisation process (Equation 2). The inter-storey inelastic deformation is chosen as the basic damage quantity, and the cumulative damage index after *N* excursions of plastic deformation is calculated as:

$$DI_i = \sum_{j=1}^{N} \left(\frac{\Delta\delta_{pj}}{\delta_y} \right)^c \tag{4}$$

where DI_i is the cumulative damage index at i^{th} storey, ranging from 0 for undamaged to 1 for severely damaged stories, *N* is the number of plastic excursions, $\Delta\delta_{pj}$ is the plastic deformation of i^{th}

storey in j^{th} excursion, δ_y is the nominal yield deformation, and c is a parameter that accounts for the effect of plastic deformation magnitude which is taken to be 1.5 [Krawinkler and Zohrei, 1984].

To assess the damage experienced by the full structure, the global damage index is obtained as a weighted average of the damage indices at the storey levels, with the energy dissipated being the weighting function.

$$DI_g = \frac{\sum_{i=1}^{n} DI_i W_{pi}}{\sum_{i=1}^{n} W_{pi}} \tag{5}$$

where DI_g is the global damage index, W_{pi} is the energy dissipated at i^{th} storey, DI_i is the damage index at i^{th} storey, and n is the number of stories.

The previous 10-storey example was solved by considering the cumulative damage (Equation 4) as the local damage index. As shown in Fig. 5, the optimum designed structure in this case exhibits minimum cumulative damage during the design earthquake. Based on the results, for the same structural weight, 10-storey buildings optimised for minimum storey-ductility and cumulative damage experience on average 40% less global damage index as compared to the IBC-2009 designed structure.

The results shown in Figures 4 and 5 indicate that, in general, changing the damage assessment criteria does not have a major effect on the optimum design load distribution, as well as the maximum storey-ductility demand and the cumulative damage of optimum designed structures. This conclusion has been confirmed by analysis of different structures and ground motion records.

4. Optimum seismic design load distribution for building code design spectra

Based on the work presented in the previous sections, it was found that for every building there is a specific optimum load distribution that leads to optimum seismic performance during the design earthquake (Opt_{eq}). This optimum pattern depends on the characteristics of the design earthquake, and therefore, varies from one earthquake to another. However, there is no guarantee that the structure will experience seismic events with the exact characteristics of the design ground motion. Therefore, for practical applications, appropriate design load distributions should be developed for typical building code design spectra.

Using the proposed optimization algorithm, the optimum load distribution patterns for the 200 shear-building models presented earlier were calculated for the five sets of selected synthetic earthquakes representing different soil types (15,000 optimum load patterns). The average of the optimum load patterns for each set of synthetic records was then used to design new shear-buildings (Opt_{ave}). For each seismic excitation, the required structural weight to obtain a target storey-ductility demand was determined for 60,000 shear-buildings designed with: a) optimum load pattern corresponding to the design earthquake (Opt_{eq}); b) IBC 2009 design; c) Hajirasouliha and Moghaddam (2009) load pattern; and d) the average of optimum load patterns (Opt_{ave}). As expected, the results of this study showed that, for the same storey-ductility demand, structures designed with the optimum load patterns corresponding to the design earthquake (Opt_{eq}) always have less structural weight (optimum structural weight) compared to the other structures. However, these optimum load patterns are specific to the particular design earthquake, and therefore, are not appropriate for general design purposes which rely on a design spectrum.

To compare the adequacy of different design load patterns, Fig. 6 compares the ratio of required to optimum structural weight (based on Opt_{eq}) for structures with fundamental period of 0.5 and 1 sec and maximum ductility demands of 1 to 8. This figure is based on the average weights required for each of the 15 synthetic earthquakes representing soil type C. It is shown that in the elastic range of response (i.e. μ_t=1), the total structural weight for models designed based on IBC-2009 load distribution is on average around 8% above the optimum value. Therefore, it is confirmed that using conventional loading patterns leads to acceptable designs for elastic structures. However, the efficiency of the code load distribution deteriorates increasingly in the non-linear range of behaviour.

In the low ductility range (i.e. μ_t<3), Hajirasouliha and Moghaddam (2009) load pattern leads to structures with less structural weight compared to IBC-2009 designed models. However, this load pattern gets worse in the high ductility range as it can result in structural weights up to 80% more than the optimum values. This is attributed to the fact that Hajirasouliha and Moghaddam (2009) load pattern was mainly developed based on a limited number of seismic records rather than a group of design spectrum-compatible earthquakes.

Structures designed with the average of optimum load distributions for a set of spectrum-compatible earthquakes (Opt_{ave}) always have less (up to 37%) structural weights compared to IBC designed structures. The results indicate that the average of the optimum load distributions can be used for seismic design of buildings in a wide range of target ductility demands (i.e. different performance targets). However, calculation of the average load patterns requires a lot of computational effort, and therefore, for practical design purposes it is necessary to develop a simple method to estimate the average of optimum load patterns for different structures and performance targets.

The results of this study show that the general form of Equation (1) can be adopted to represent the average of optimum load patterns corresponding to different building code design spectra (Opt_{ave}). For this purpose, the constant coefficients a_i, b_i, c_i, and d_i in Equation (1) should be calculated based on the average of the results for a set of synthetic spectrum-compatible earthquakes representing a specific design spectrum. For example, Table 2 shows the constant coefficients corresponding to the design response spectrum of IBC-2009 soil type C. These coefficients can be obtained at each level of the structure by interpolating the values given in Table 2. Fig. 6 shows that structures designed with the modified coefficients (Table 2) always require less structural weight compared to similar structures designed with IBC-2009 and Hajirasouliha and Moghaddam (2009) load patterns. The results also indicate that structures designed with the proposed equation behave very similar to those designed with the average of optimum load patterns (Opt_{ave}).

Fig. 7 compares the new general load distributions (calculated by using Equation 1 and modified coefficients given in Table 2) and the corresponding load distributions obtained from nonlinear dynamic analysis. The results of the proposed equation compare very well with the analytical results, and the equation works well for different periods and ductility demands.

5. Efficiency of the proposed design load pattern

The adequacy of different design load patterns can be assessed by evaluating their correlation with the average of optimum load patterns corresponding to the typical building code design spectra. For this purpose, the following efficiency factor is defined in this study:

$$EF = \frac{\sqrt{\sum_{i=1}^{n} [(\phi_i)_{design} - (\phi_i)_{ave}]^2}}{n} \tag{6}$$

where n is the number of stories, and $(\phi_i)_{design}$ and $(\phi_i)_{\mathrm{ave}}$ are the scaled lateral load pattern at i^{th} storey calculated based on the selected design load pattern and the average of optimum load patterns, respectively.

Fig. 8 compares the EF factor for structures designed with IBC-2009, Hajirasouliha and Moghaddam (2009) load pattern, and the general load pattern proposed in this study. This figure shows the average of the results for ten 10-storey structures with fundamental periods between 0.1 to 3 sec. It is shown that the general load pattern has better agreement with the average of the optimum load patterns compared to IBC-2009 and Hajirasouliha and Moghaddam (2009) load patterns. The efficiency of the proposed equation is further assessed by using a design example in the upcoming sections.

6. Effect of site soil profile on optimum design load distribution

To investigate the effect of site soil classification on the optimum seismic design load distribution, five sets of 15 synthetic earthquakes were considered as introduced in section 2. For each synthetic ground motion record, the optimum design load distribution was derived for the 200 shear-building models with different fundamental periods and target ductility demands. Using the suggested formula for optimum design load distributions, the constant coefficients a_i, b_i, c_i, and d_i were determined for each group of synthetic earthquakes representing a site soil classification, and compared in Fig. 9.

The results indicate that the optimum load distributions for structures with similar fundamental period and maximum ductility demand sited on soil profiles type A, B, C and D are practically identical. However, the optimum load distributions for soft soil profiles (type E) are slightly different. Therefore, for practical applications, it is suggested to provide two sets of coefficients a_i, b_i, c_i, and d_i for hard rock to stiff soil profiles and for soft soil.

7. Design load distribution for height-wise irregular structures

To investigate the effect of height-wise irregularity on the design load distribution, six 10-storey shear-buildings with different mass distribution patterns were considered as shown in Fig. 10. Using the proposed optimisation algorithm, the buildings were designed to have a fundamental period of 1 sec and uniform storey ductility of 4 when subjected to the Northridge earthquake of 1994 (CNP196). For each height-wise mass distribution, there is a specific load distribution that leads to a uniform storey ductility demand. Using the proposed optimisation method, the optimum seismic design load distribution for the irregular shear-building models (type A to F) were calculated as shown in Fig. 11 .

To evaluate the efficiency of the proposed optimum design load distributions, the shear-building models shown in Fig. 10 were also designed with the IBC-2009 load distribution using the same structural weight as the optimum designed models. Storey ductility distribution of IBC-2009 and optimum designed models are compared in Fig. 12. The results indicate that, for the same structural weight, optimum designed models experience less maximum storey ductility (up to 55% less), and therefore, less structural damage during the design earthquake (Northridge earthquake of 1994).

Lateral seismic design load in most of design guidelines is considered to be proportional to the storey weight (UBC-97, Eurocode 8, IBC-2009 and ASCE 7-05). In this study, a similar concept is used to normalize the optimum design load distribution of height-wise irregular structures. Fig. 13 compares the optimum design load distributions of different height-wise irregular structures (shown in Fig. 10) after being normalized to the relative storey weight. Despite the big difference between height-wise mass distributions of the examined shear-building structures, the results indicate that the normalized optimum design loads are almost identical for buildings with similar fundamental period and maximum ductility demand. The small difference between the normalized optimum design load distributions may be due to the effect of higher modes.

By knowing the optimum load distribution for a height-wise regular structure, the optimum load distribution for an irregular structure with similar fundamental period and maximum ductility demand can be calculated by using the following equation:

$$\phi_i' = \frac{w_i \phi_i}{\sum_{j=1}^{n} w_j \phi_j} \tag{7}$$

where ϕ_i and ϕ'_i are the ratio of optimum design force at i^th storey to the base shear (load distribution pattern) of regular and irregular structures, respectively; w_i is the weight of the i^th storey; and n is the number of stories. To calculate the optimum load distribution for an irregular shear-building, first the optimum load distribution for a regular building with similar fundamental period and maximum ductility demand should be calculated by using Equation (1) with appropriate coefficients for site soil classification (shown in Fig. 9). Subsequently, Equation (7) should be used to convert the optimum design load distribution for the equivalent irregular structure.

8. Verification using an irregular shear-building design example

The efficiency of the general load pattern proposed in this study is demonstrated through the seismic design of an irregular five-storey shear-building shown in Fig. 14-a. The building is assumed to be located on a soil type B of IBC-2009 with fundamental period of 0.6 sec and maximum storey ductility demand of 6. The proposed design load pattern was calculated by using Equations (1) and (6) as explained in the previous section. Two shear buildings were designed based on the IBC-2009 and the proposed load pattern (shown in Fig. 14-b), and subjected to 15 synthetic earthquakes representing the IBC-2009 soil type B design spectrum. For each seismic excitation, the required structural weight (i.e. sum of storey shear strengths) was calculated to obtain target ductility demand of 6. The average and 95^th percentile (average plus 1.65 times the standard deviation) of the required storey shear strengths were 4570 kN and 6290 kN for the IBC-2009 structure, and 3660 kN and 4710 kN for the structure designed based on the proposed load pattern, respectively. Therefore, for the same maximum ductility demand, the shear-building designed by the proposed load pattern requires considerably less (up to 34% less) structural material.

The average and envelope of maximum storey ductility demands for the IBC-2009 and the structure designed by the proposed load pattern are compared in Fig. 15. It is shown that by using the proposed load pattern, the seismic capacity of the designed structure was fully utilized as the maximum ductility demand of all stories reached the target ductility of 6 at least at one earthquake. The results indicate that, in general, the proposed load pattern leads to shear-buildings with a more uniform storey ductility demand.

9. Verification using concentrically braced frames

The efficiency of the proposed load pattern is further examined for the seismic design of three concentrically braced steel frames of 5, 10 and 15 stories (shown in Fig. 16). The buildings were assumed to be located on a soil type C of IBC-2009 (and ASCE 7-05) category, with the design spectral response acceleration at short and 1-sec periods equal to 1.1g and 0.64g, respectively. Ordinary concentrically braced frames (OCBF) were designed to support gravity 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. In all models, the top storey was considered to be 25% lighter than the rest. 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 eliminate the effect of discrete section sizes, auxiliary sections were artificially developed by assuming a continuous variation of section properties based on DIN-1025. In the code based designed models, once the members were seized, the entire design was checked for the code drift limitations and refined to meet the code requirements when necessary. A beam-column element which allows for the formation of P-M hinges near its ends was employed to model the columns. The post-buckling behaviour of brace members was taken into account by utilizing the hysteretic model suggested by Jain et al. (1980). In this section, shear inter-storey drift is considered as the main performance parameter to assess the level of structural and non-structural damage as suggested by Bertero et al. (1991) and Moghaddam et al. (2005).

The 5, 10 and 15 storey concentrically braced frames were designed based on IBC-2009 and the proposed load patterns (Figures 17-a to 19-a) and subjected to 15 synthetic earthquakes representing the IBC-2009 soil type C design spectrum. The proposed design load pattern for each model was scaled to obtain the same structural weight as the IBC-2009 frames. The average and envelope of maximum shear inter-storey drifts for the IBC-2009 and the structures designed by the proposed load pattern are compared in Figures 17-b to 19-b. The results indicate that the efficiency of the proposed load pattern for seismic design of a concentrically braced frame is less than for shear-building models as the distribution of shear storey drift is not fully uniform. However, for the same structural weights, concentrically braced frames designed with the proposed load pattern always undergo lower shear inter-storey drifts (up to 20%) under design earthquakes, and therefore, exhibit an overall better seismic performance compared to IBC-2009 designed models.

10. Application of the proposed design load pattern

In performance-based design methods, different multiple limit states (e.g. service event, rare event, very rare event) are usually considered. The optimum design for a specific limit state does not guarantee the optimum behaviour in other conditions. In this case, it is usually accepted to use the very rare event as the governing criterion for the initial design, and then check the design for other limit states.

The results of this study indicate that the general loading pattern proposed in this paper is efficient for structural systems that exhibit shear-building like behaviour, such as buckling-restrained braced frames and moment resisting frames with high beam-to-column stiffness ratio. The efficiency of the proposed load pattern reduces slightly for conventional concentrically braced frames, since the seismic behaviour of the frames is significantly influenced by the slenderness of the brace elements (Karavasilis et al. 2007). However, the proposed load pattern can still improve the seismic performance of the designed frames, and should prove useful in the conceptual design phase.

Initial studies show that the proposed loading pattern cannot be directly applied to some structural systems such as structural walls, as they behave substantially different from shear-building type of structures. Further research is required to extend the proposed load pattern to different structural systems and different performance targets.

11. Conclusions

A method based on the concept of uniform damage distribution is adopted for optimum seismic design of regular and irregular structures subjected to a design seismic excitation. It is shown that, for the same structural weight, structures designed with the optimum load distribution experience up to 50% less maximum storey ductility and 40% less global damage compared to code-based designed structures.

It is shown that optimum design load distribution, storey ductility demand and global damage index for buildings optimised for minimum storey ductility and minimum cumulative damage are relatively similar.

For a set of synthetic earthquakes representing a typical building code design spectrum, optimum seismic design load distributions were determined. It is shown that structures designed with the average of optimum load distributions have up to 37% less structural weight compared to similar conventionally designed structures.

The results indicate that the optimum load distributions for structures with similar fundamental period and maximum ductility demand sited on IBC-2009 soil profiles type A, B, C and D (hard rock to stiff soil) are nearly identical. However, the optimum load distributions for soft soil profiles (type E) are slightly different.

Based on the results of this study, a general load distribution is introduced for seismic design of height-wise regular and irregular structures that is a function of soil type, fundamental period of the structure and maximum ductility demand. It is shown that using the proposed loading pattern leads to a more efficient use of structural materials, and therefore, better seismic performance for shear-building like structures and concentrically braced frames. Further work is required to extend the proposed design load pattern to other types of structural systems and performance targets.

References

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

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

Anderson, J. C., Miranda, E. and Bertero, V. V. [1991] "Evaluation of the seismic performance of a thirty-story RC building" UCB/EERC-91/16, Berkeley: Earthquake Engineering Research Centre, University of California.

ASCE 7-05 [2006] Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, Virginia.

Baik, S. W., Lee, D. G. and Krawinkler, H. [1988] "A simplified model for seismic response prediction of steel frame structures," In: The 9th WCEE, Vol. 5, Japan.

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.

CEN Eurocode 8 [2004] design provisions of structures for earthquake resistance. European Committee for Standardization. Brussels.

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

Chopra, A. K. [2001] Dynamics of structures: theory and applications to earthquake engineering. 2nd Edition, Prentice-Hall: Englewood Cliffs, NJ.

Diaz, O., Mendoza, E. and Esteva, L. [1994] "Seismic ductility demands predicted by alternate models of building frames," Earthquake Spectra 10(3), 465-487.

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

Gantes, C. J., Vayas, I., Spiliopoulos, A. and Pouangare, C. C. [2000] "Optimum Bending and Shear Stiffness Distribution for Performance Based Design of Rigid and Braced Multi-Story Steel Frames," In: 3rd International Conference STESSA 2000- Behaviour of Steel Structures in Seismic Areas, F.M. Mazzolani and R. Tremblay (Editors), Montreal, Canada, 585-592.

Gilmore, T. A. and Bertero, V. V. [1993] "Seismic performance of a 30-story building located on soft soil and designed according to UBC 1991," UCB/EERC-93/04, Berkeley, EERC, University of California.

Goulet, C. A., Haselton, C. B., Reiser, J. M., Beck, J. L., Deierlein, G. G., Porter, K. A. and Stewart, J. P. [2007] "Evaluation of the seismic performance of a code-conforming reinforced-concrete frame building-from seismic hazard to collapse safety and economic losses," Earthquake Eng Struc 36(13), 1973-1997.

Hajirasouliha, I. and Doostan, A. [2010] "Simplified model for seismic response prediction of concentrically braced frames," Advances in Engineering Software 41 (3), 497-505.

Hajirasouliha, I. and Moghaddam, H. [2009] "A new lateral force distribution for seismic design of structures," ASCE J. Structural Eng 135 (8), 906-915.

Hart, G. C. [2000] "Earthquake forces for the lateral force code," The Structural Design of Tall Buildings 9(1), 49-64.

IBC-2009 [2009] 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 Structural Division ASCE, 106, 1777-1795.

Karavasilis, T.L., Bazeos, N. and Beskos, D.E. [2007] "Estimation of seismic drift and ductility demands in planar regular X-braced steel frames," Earthquake Engineering and Structural Dynamics 2007 36 (15), 2273-2289.

Krawinkler, H. and Zohrei, M. [1984] "Cumulative damage in steel structures subjected to earthquake ground motions," Computers & Structures 16 (1-4), 531-541.

Lee, K. and Foutch, D. A. [2002] "Performance evaluation of new steel frame buildings for seismic loads," Earthquake Eng Struc 31(3), 653-670.

Lee, S. S. and Goel, S. C. [2001] "Performance based seismic design of structures using target drift and yield mechanism," In: U.S Japan Seminar on Advanced Stability and Seismicity Concept for Performance Based Design of Steel and Composite Structures, Kyoto, Japan.

Martinelli, L., Perotti, F. and Bozzi, A. [2000] "Seismic design and response of a 14-story concentrically braced steel building," in: Behavior of Steel Structures in Seismic Areas. Mazzolani F and Tremblay R (eds), Balkema, Rotterdam, 327-335.

Moghaddam, H. and Hajirasouliha, I. [2006] "Toward more rational criteria for determination of design earthquake forces," Int J Solids Struct 43(9), 2631-2645.

Moghaddam, H., Hajirasouliha, I. and Doostan, A. [2005] "Optimum seismic design of concentrically braced steel frames: concepts and design procedures," J Constr Steel Res 61 (2), 151-166.

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.

Takewaki, I. [1996] "Design-oriented approximate bound of inelastic responses of a structure under seismic loading," Comput Struct 61 (3), 431-440.

Takewaki, I. [1997] "Design-oriented ductility bound of a plane frame under seismic loading," J Vib Control 3 (4), 411-434.

Uniform Building Code, UBC [1997] International Conference of Building Officials, vol. 2.

Vanmarke, E.H. [1976] SIMQKE: A Program for Artificial Motion Generation, User’s Manual and Documentation. Department of Civil Engineering, Massachusetts Institute of Technology.

Table 1- Site soil classifications according to IBC-2009

Site class	Soil profile name	Soil shear wave velocity
A	Hard rock	> 1500 m/s
B	Rock	760 to 1500 m/s
C	Very dense soil and soft rock	370 to 760 m/s
D	Stiff soil profile	180 to 370 m/s
E	Soft soil profile	< 180 m/s

Table 2- Modified coefficients for Equation (1) as a function of relative height (site class C)

Relative Height	a	b	c	d
0	6.14	20.15	6.89	62.35
0.1	3.17	32.81	6.40	45.75
0.2	0.24	45.50	5.91	29.19
0.3	-1.92	58.78	5.03	16.09
0.4	-2.86	71.75	2.63	7.89
0.5	-4.33	87.18	0.85	0.90
0.6	-5.71	104.33	-0.33	-5.23
0.7	-5.79	122.37	-1.76	-8.52
0.8	-2.95	141.16	-3.20	-10.23
0.9	4.79	160.50	-4.70	-10.46
1	21.96	184.07	-6.84	-8.61

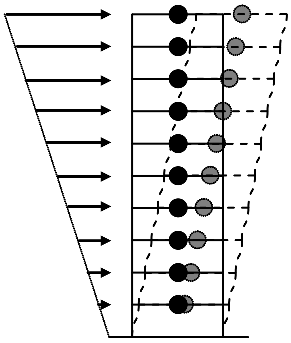

Fig. 1- Typical shear-building model

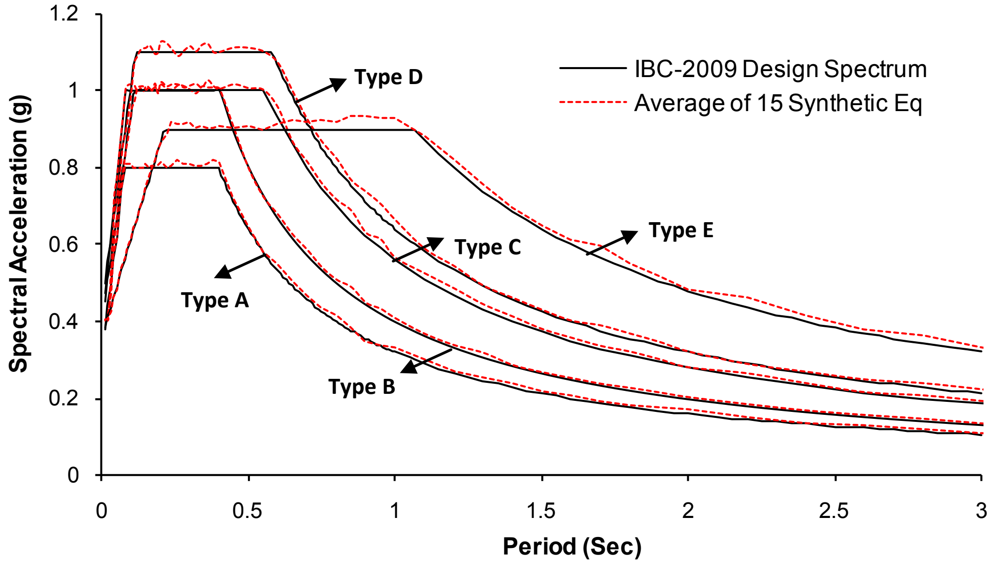

Fig. 2- Comparison between IBC-2009 design spectrum and average response spectra of 15 synthetic earthquakes

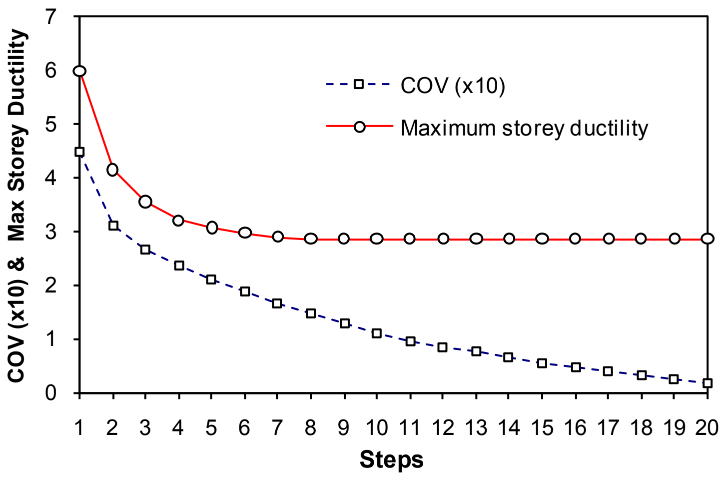

Fig. 3- Variation of maximum storey ductility and COV of storey ductility demands from IBC-2009 to optimum model

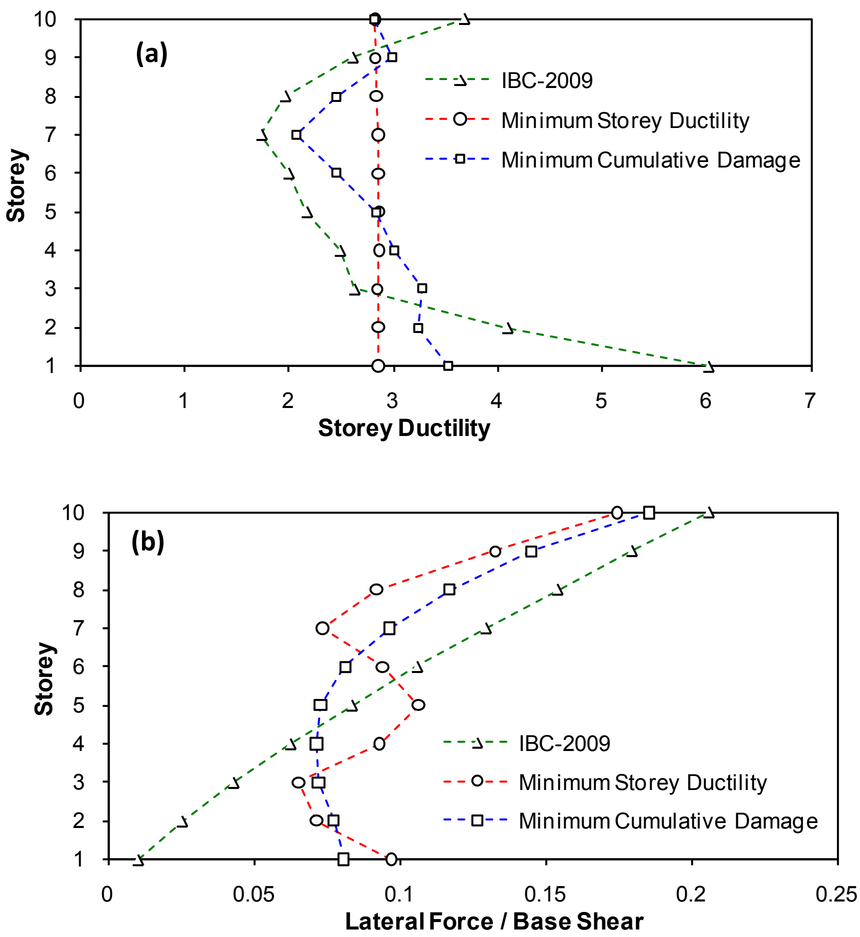

Fig. 4- (a) Storey ductility, and (b) lateral force distribution of IBC-2009 model and optimum models designed for minimum storey ductility and cumulative damage

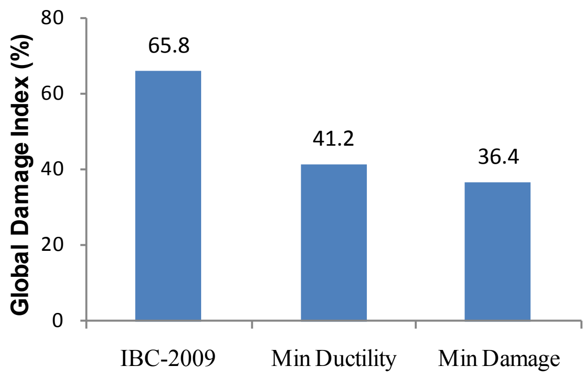

Fig. 5- Global damage index for IBC-2009 model and optimum models designed for minimum storey ductility and cumulative damage

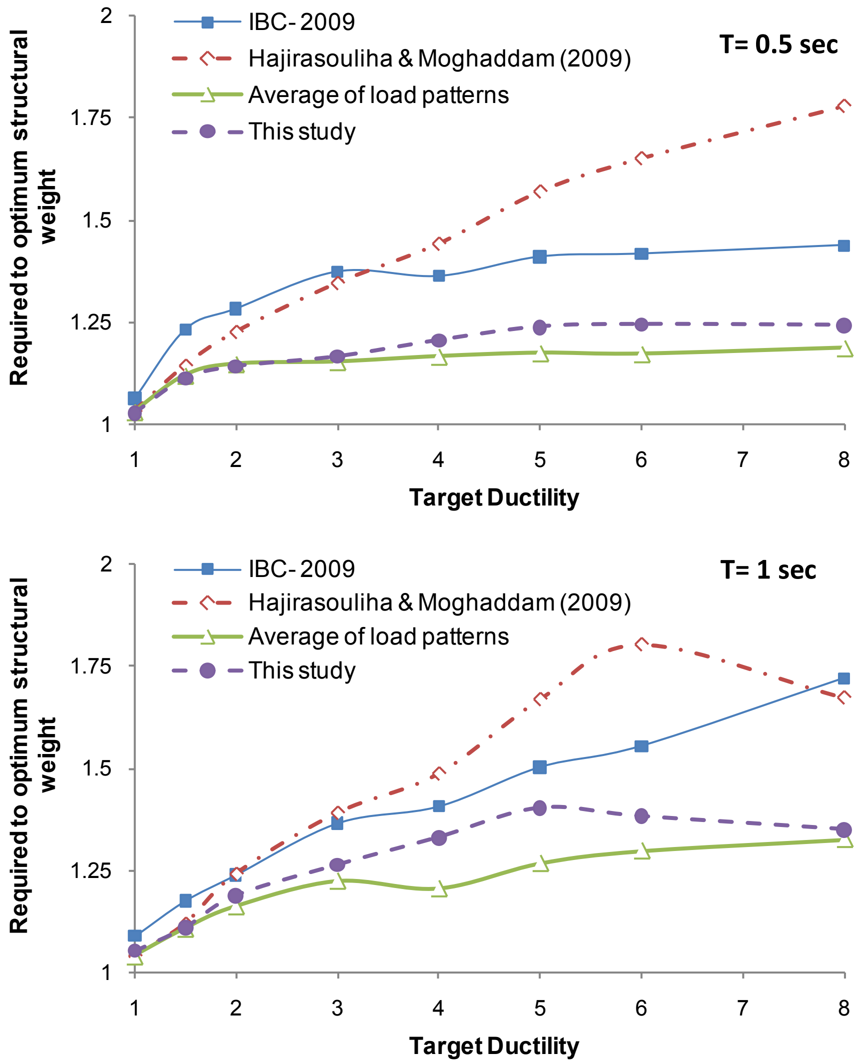

Fig. 6- The ratio of required to optimum structural weight for structures designed with IBC-2009, Hajirasouliha and Moghaddam (2009), average of optimum load patterns (Opt_{ave}), and the general load pattern proposed in this study, average of 15 synthetic earthquakes (soil type C)

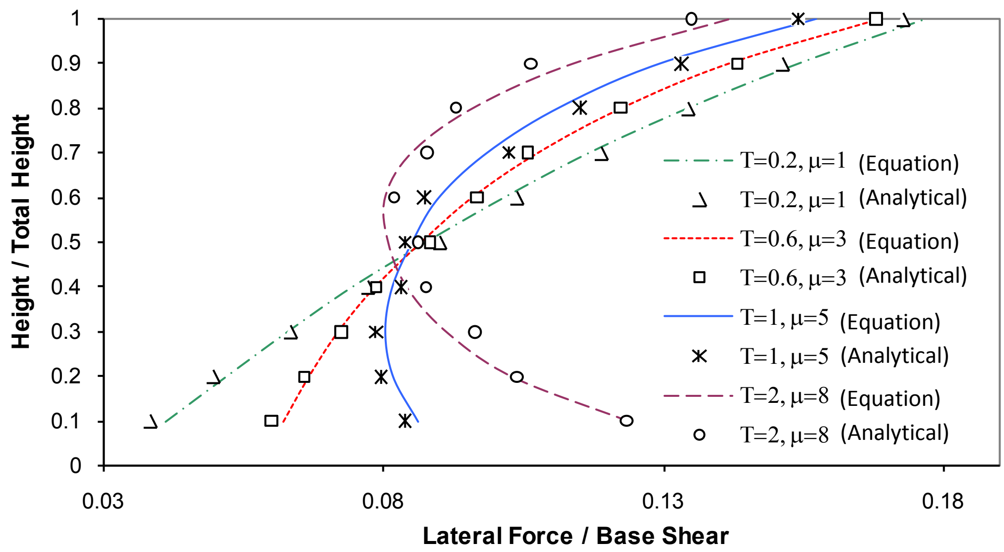

Fig. 7- Correlation between the proposed equation and analytical results

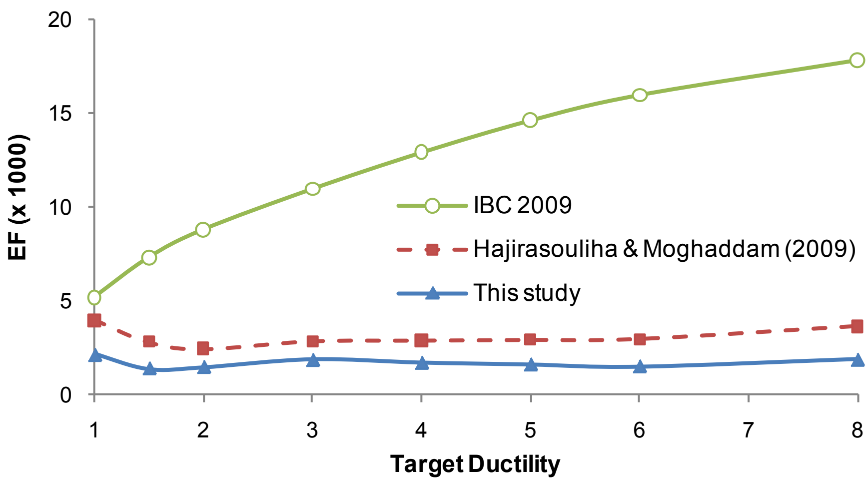

Fig. 8- Efficiency Factor (EF) for structures designed with IBC-2009, Hajirasouliha and Moghaddam (2009), and the general load pattern proposed in this study

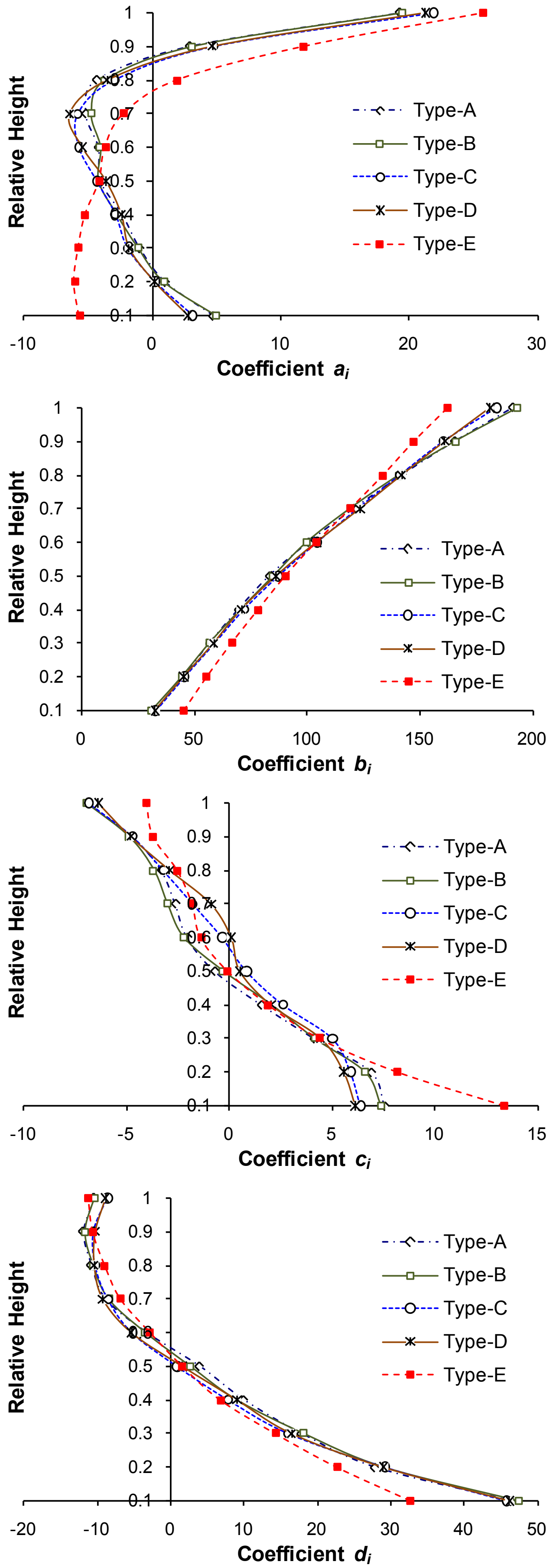

Fig. 9- Constant coefficients a_i, b_i, c_i, and d_i (Equation 1) for different site soil classifications

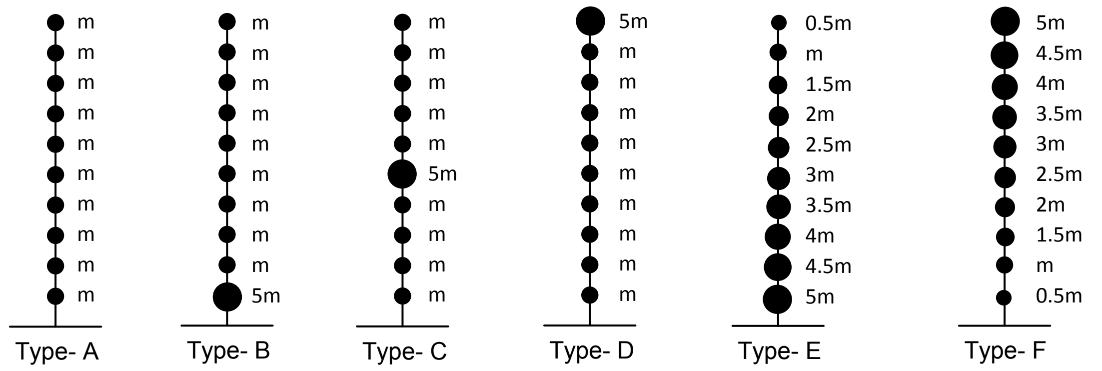

Fig. 10- Shear-building models with different height-wise mass distribution

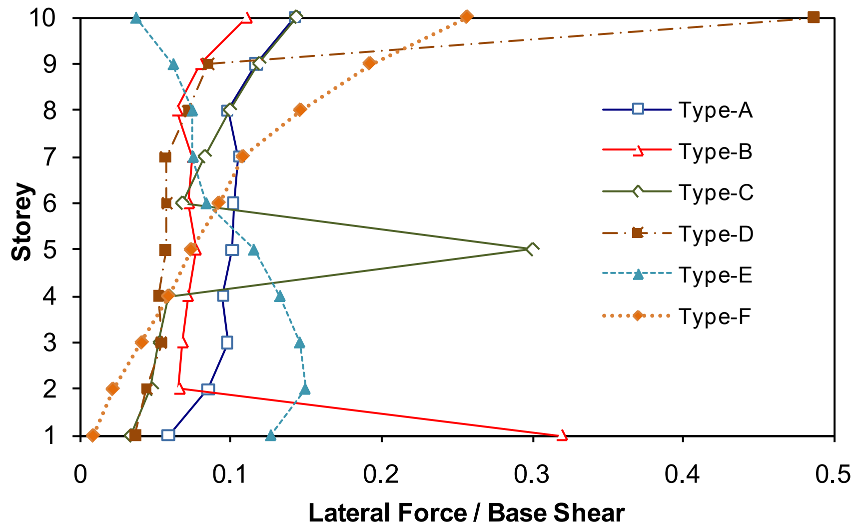

Fig. 11- Optimum seismic design load distribution for shear-buildings with different height-wise mass distribution

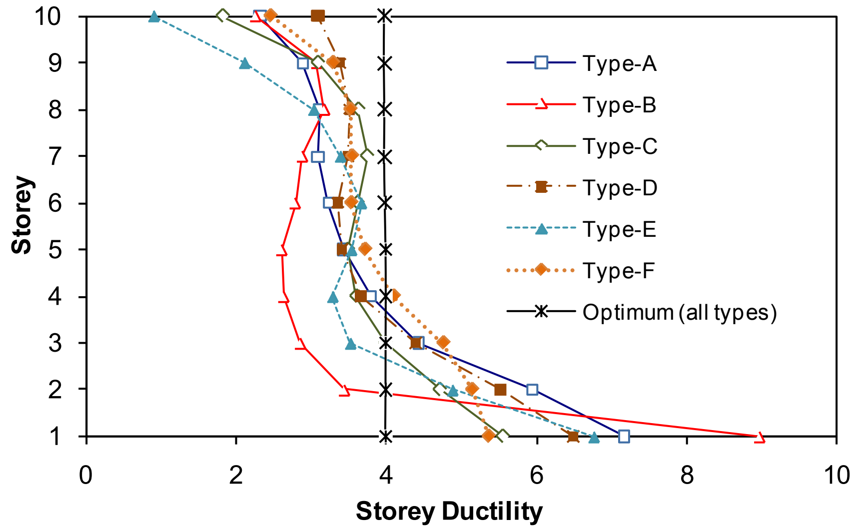

Fig. 12- Storey ductility distribution of IBC-2009 and optimum designed buildings having different height-wise mass distribution

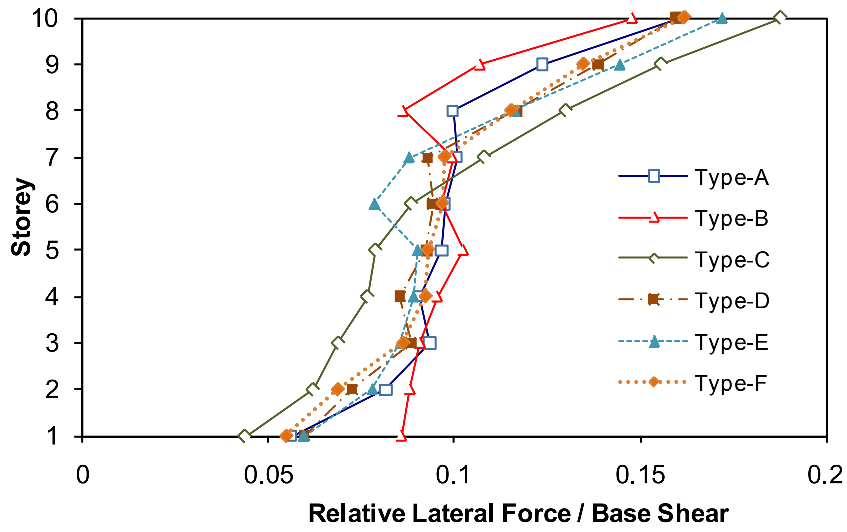

Fig. 13- Normalized optimum seismic design load distribution for shear-buildings with different height-wise mass distributions

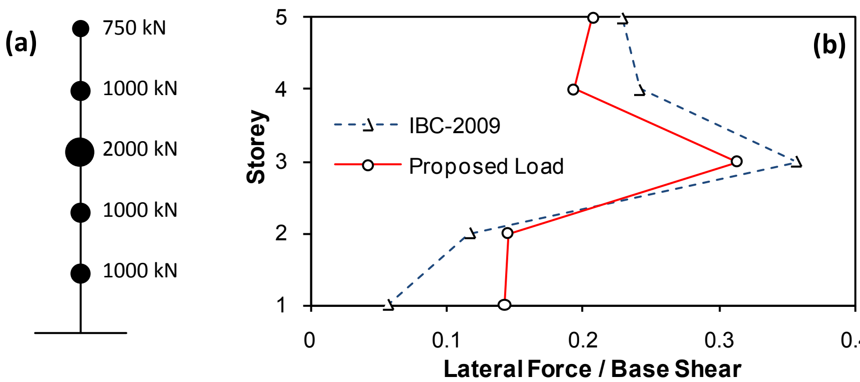

Fig. 14- (a) Storey weight, and (b) Comparison between IBC-2009 and the proposed load pattern

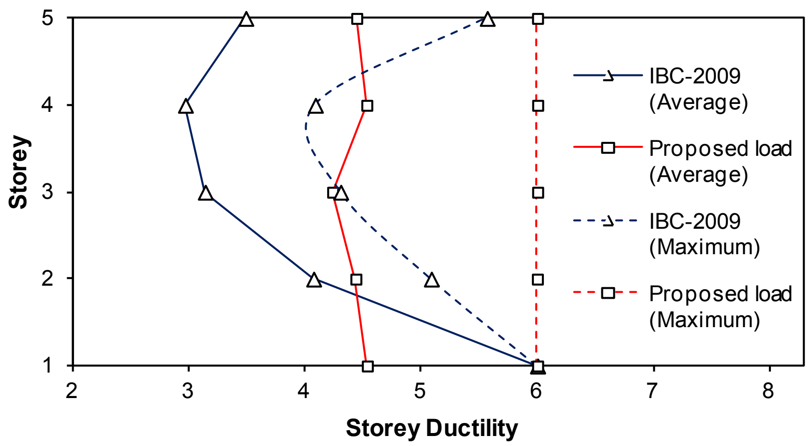

Fig. 15- Maximum and average of storey ductility demands for 15 synthetic earthquakes representing IBC-2009 design spectrum

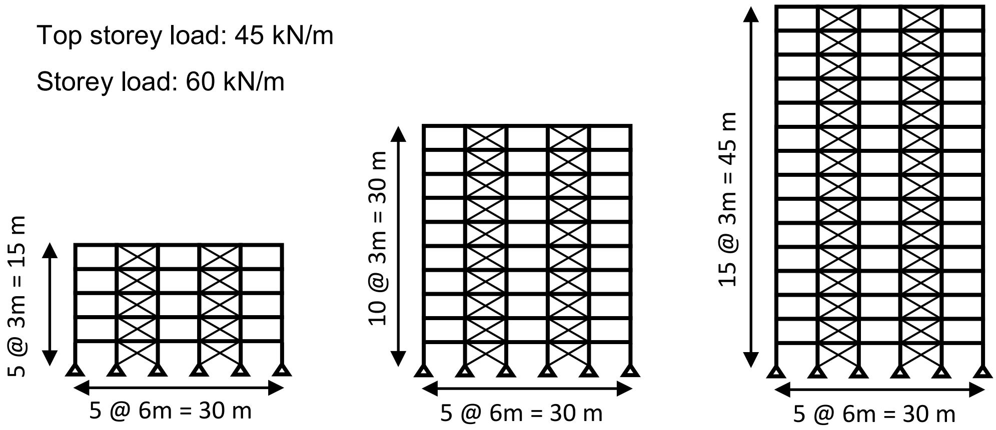

Fig. 16- Typical geometry of concentrically braced frames

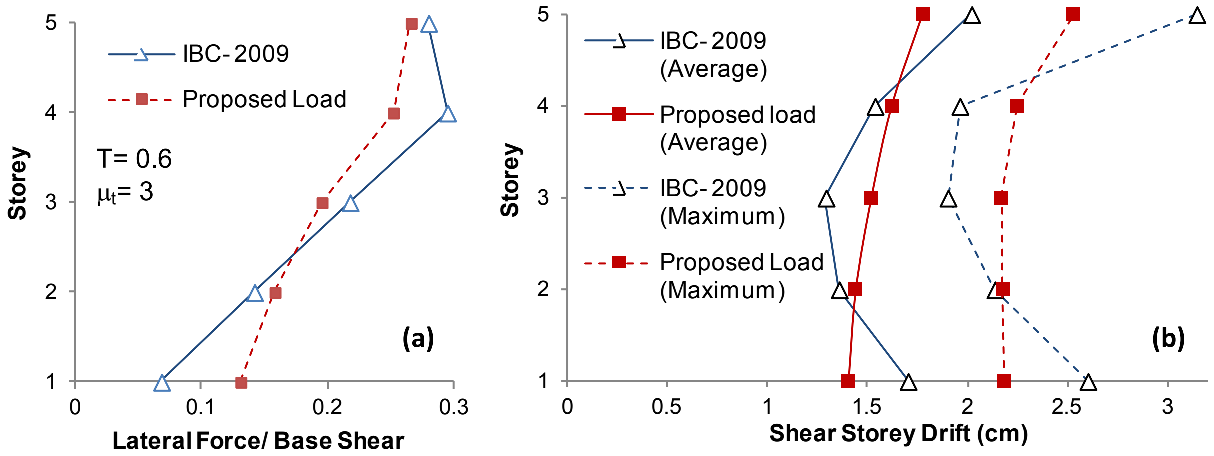

Fig. 17- (a) IBC-2009 and optimum seismic design load distribution, and (b) Maximum and average of shear storey drifts for the 5-storey concentrically braced frame

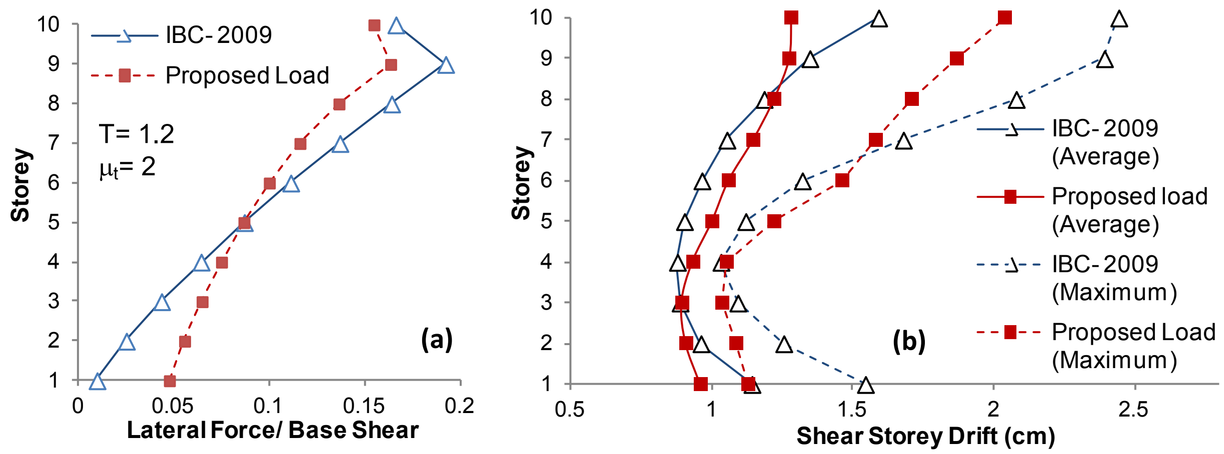

Fig. 18- (a) IBC-2009 and optimum seismic design load distribution, and (b) Maximum and average of shear storey drifts for the 10-storey concentrically braced frame

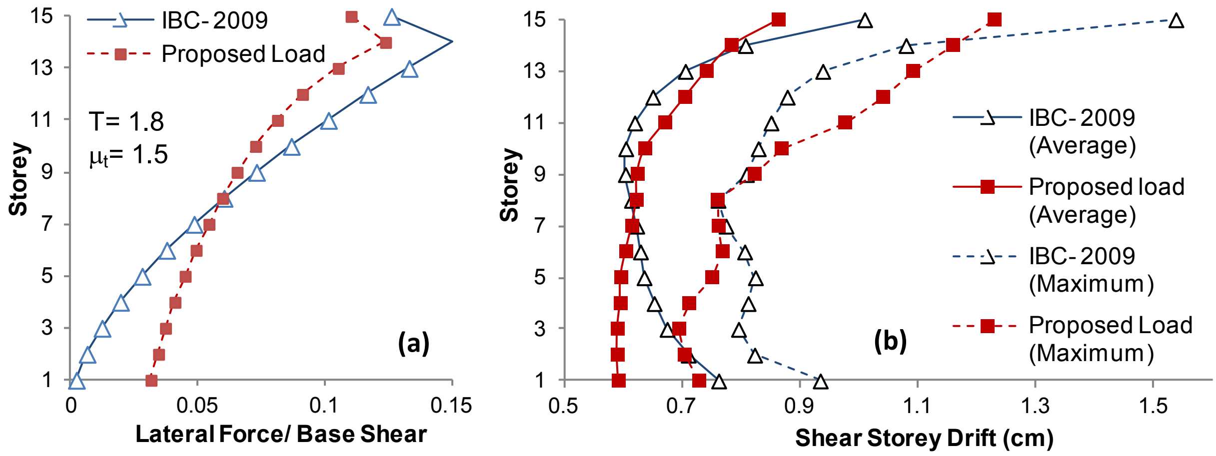

Fig. 19- (a) IBC-2009 and optimum seismic design load distribution, and (b) Maximum and average of shear storey drifts for the 15-storey concentrically braced frame