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Parametric study on energy demands for steel special concentrically braced frames

Selçuk Doğru [*1], Bora Akşar [1a], Bülent Akbaş [1b] and Jay Shen [2b]

[1] *Department of Civil Engineering, Gebze Technical University, Kocaeli, Turkey*
[2] *Department of Civil, Construction and Environmental Engineering, Iowa State University, USA*

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Abstract. Structures are designed in such a way that they behave in a nonlinear manner when subject to strong ground motions. Energy concepts have been widely used to evaluate the structural performance for the last few decades. Energy based design can be expressed as the balance of energy input and the energy dissipation capacity of the structure. New research is needed for multi degree of freedom systems (MDOFs)-real structures- within the framework of the energy based design methodology. In this paper, energy parameters are evaluated for low-, medium- and high-rise steel special concentrically braced frames (SCBFs) in terms of total energy input and hysteretic energy. Nonlinear dynamic time history analyses are carried out to assess the variation of energy terms along the height of the frames. A seismic energy demand spectrum is developed and hysteretic energy distributions within the frames are presented.

Keywords: special concentrically braced frame; energy spectrum; energy demands; energy based design; energy dissipation

1. Introduction

Forced-based design has been widely used to consider the seismic effects on structures. The structural members are checked based on structural analysis of the frame under the equivalent static lateral load combined with gravity loads. Final design of the structure assumes nonlinear behavior of the structure under strong ground motions. Energy based design parameters have long been considered as reliable to estimate damage and evaluate nonlinear behavior. Energy input to the structure depends on the ground motions characteristics as well as structural properties. The properties of the strong ground motions (frequency content, duration, amplitude, etc.) and the properties of the structural (natural frequency, mass, etc) are effective on energy input to the structure. Capacity of the structure can be defined as the energy dissipation capacity and obtained by summing the energy dissipation capacities of the individual components. Energy dissipation capacity of a structural component is equal to the area enclosed by the force-deformation relationship under cyclic loadings. Structural design is based on the balance between the energy input and energy dissipation capacity of the structure. Structures are expected to dissipate all the energy input to the structure. Part of this energy is stored as elastic strain energy and kinetic energy, the rest is dissipated through damping and nonlinear behavior.

Energy based design was first proposed by Housner (1956). Akiyama (1985) further developed Housner's (1956) method and developed an earthquake-resistant design method to be applied from one-story to multi-story buildings. Uang and Bertero (1988) analyzed the energy equations and studied to estimated energy demands for SDOF systems. Fajfar *et al*. (1991) studied the energy input into SDOF systems and proposed a formula to predict the maximum energy input. Fajfar and Vidic (1994) proposed inelastic design spectra considering hysteretic energy and input energy. Shen and Akbas (1999) predicted the energy input in low-, medium-, and high-rise steel moment resisting frames (SMRFs) and proposed an energy-based earthquake resisting design method for SMRFs. In their study, they made comparisons of the energy input between the formulas proposed for mainly SDOF systems and concluded that there was a clear difference between the SDOF systems and MDOF systems in terms of energy input. Shen *et al*. (2000) investigated the distribution of energy terms in MDOF systems. They provided the distribution of hysteretic energy in regular steel moment frames and emphasized the difference in energy terms between SDOF and MDOF systems. In another study by the same authors (Akbas *et al*. 2001), a practical design approach based on hysteretic energy was proposed. Akbas and Shen (2002) proposed an Energy Spectrum based on the nonlinear dynamic time history analysis on SDOF systems within the period range of 0-3.0 sec subject to seven different EQGMs.

Benavent-Climent and Zahran (2010) studied the energy input into existing reinforced concrete frames. Bojórquez *et al*. (2010) introduced an energy-based damage index for

∗Corresponding author, Ph.D. Candidate,
E-mail: dogrusel@gmail.com
[a] Ph.D. Candidate
[b] Ph.D., Professor
[c] Ph.D., Associate Professor

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steel buildings accounting explicitly for the effects of cumulative plastic deformation demands. Guan *et al*. (2011) carried out the variation of hysteretic energy on a R/C frame subject to multiple ground motions. Zhang and Wang (2012) carried out a study on a high-rise steel building to investigate the effects of variability for hysteretic energy. Habibi *et al*. (2013) proposed a multi-mode energy-based design method for seismic retrofitting with passive energy dissipation systems. Paolacci (2015) performed an analytical study on developing an energy-based design for structures with viscoelastic dampers. Manoukas *et al*. (2014) suggested a design methodology based on performance-based design principles on energy-equivalent of SDOF systems. Bojórquez *et al*. (2015) provided new parameters for predicting energy demands in structures.

All of the above studies are carried out on regular frames. CBFs are among the most cost-effective systems to resist lateral loads in steel buildings. Contrary to their advantages, they perform the worst among all available steel framing systems in inelastic range due to buckling of the braces. Inelastic energy (hysteretic energy) is expected to be dissipated through braces. However, they have such a limited energy dissipation capacity when braces buckle. Thus, energy terms derived from previous studies might not be applicable for CBFs. A comprehensive study is strongly needed to understand the variation of energy terms in concentrically braced frames (CBFs) when subject to strong ground motions.

In this paper, energy parameters -total energy input and hysteretic energy- are evaluated to investigate the variation of energy terms in CBFs. To suggest an energy based design for a certain type of framing systems, the main steps to be investigated are (1) to predict energy input to structure; (2) to estimate total inelastic dissipated energy (hysteretic energy); (3) to identify the distribution hysteretic energy throughout the frame; (4) to compare the seismic energy demand and energy capacity of the structural members. Structural analyses are needed for the first three steps which represent demand side of the basic design equation. The last step requires the estimate of energy dissipation capacity of the braces and can only be obtained from experimental studies. The current study mainly focuses on the demand parameters to develop an energy demand spectrum for CBFs. For this purpose, CBFs with different bracing configuration (inverted V and two-story X) are designed. An ensemble of ground motions are selected so that the seismic response of each of the four frames would range from moderate to severe and the seismic energy demand would be evaluated based on the response of the frames. Two sets of ground motions corresponding to 10% and 2% probability of exceedance in 50-year time period which correspond to the life safety performance level (LS) and collapse prevention performance level (CP), respectively are used in nonlinear dynamic time history analyses.

2. Energy-based design approach

An earthquake resistant design methodology based on the energy concepts may be expressed by equating the energy input into a structure due to design earthquake (demand) to the energy absorption capacity of the structure (capacity). The demand should be equal or smaller than the supply for a proper design. To derive energy equations, either “relative energy formulation” or “absolute energy formulation” is used. Although the two methods are equivalent, absolute energy formulation is based on defining the input energy as the work done by the total base shear on the absolute ground displacement while relative energy formulation is based on defining the input energy as work done on a fixed-based system by an equivalent seismic lateral force. Relative energy equations represent the energy values obtained from the motion of the system relative to the base. Uang and Bertero (1988) showed that relative and absolute input energies are very similar for wide period range (0.3 sec to 5.0 sec). Considering that the relative displacements and velocities cause internal forces in a structure, defining the energy values in terms of the relative motion becomes more realistic (Chopra 2010). Besides that, the energy dissipated in viscous damping or yielding depends only on the relative motion.

The equation of motion for a viscous damped SDOF is

$$m\ddot{u}_t + c\dot{u} + f_s = 0 \tag{1}$$

where m is the mass; c is the viscous damping coefficient, f_s is the restoring force (for a linear elastic system $f_s = k_u$, k = stiffness), u is the relative displacement of the mass relative to the ground, u_g is the earthquake ground motion displacement, u_t is the total displacement of the mass ($u + u_g$). So, Eq. (1) for a SDOF system subjected to a horizontal earthquake ground motion can be rewritten as

$$m\ddot{u} + c\dot{u} + f_s(u,\dot{u}) = -m\ddot{u}_g(t) \tag{2}$$

It is necessary to derive the relative energy equations to develop reliable energy-based design methods. The input energy into an inelastic SDOF system due to an EQGM is dissipated by both viscous damping and yielding. The following energy terms can be defined by integrating the equation of motion (Eq. (2)) as follows (Uang and Bertero1988, Chopra 2010)

$$\int_0^u m\ddot{u}(t)du + \int_0^u c\dot{u}(t)du + \int_0^u f_s(u,\dot{u})du = -\int_0^u m\ddot{u}_g(t)du \tag{3}$$

The terms on left side of Eq. (3) represents the kinetic energy (E_K), damping energy (E_D) and absorbed energy (E_A), respectively. Based on the defined energy quantities, the energy response of a nonlinear system can be written as

$$E_K(t) + E_D(t) + E_A(t) = E_I(t) \tag{4}$$

The right side of Eq. (3) represents the total relative energy input, $E_I(t)$, to the structure and is defined as the work done by the effective seismic force over the structural deformation.

$$E_I(t) = -\int_0^u m\ddot{u}_g(t)du \tag{5}$$

The absorbed energy (E_A) as expressed in Eq. (6)

consists of elastic strain energy (E_E) and hysteretic energy (E_H).

$$E_A(t) = \int_0^u f_S(u,\dot{u})du = E_E(t) + E_H(t) \tag{6}$$

$$E_E(t) = \frac{[f_S(t)]^2}{2k} \tag{7}$$

$$E_H(t) = \int_0^u f_S(u,\dot{u})du - E_e(t) \tag{8}$$

where k is the initial stiffness of the system. The hysteretic energy, $E_H(t)$ includes the inelastic deformation of structural members and is directly related to the cyclic deformation capacity of structural components. E_H is equal to zero if the structure remains elastic. The relative energy balance equation can be rewritten in the following form

$$E_K(t) + E_D(t) + E_E(t) + E_H(t) = E_I(t) \tag{9}$$

If Eq. (9) were considered as a design equation (capacity ≥ demand), the four terms on the left-hand side of Eq. (9), could be considered as energy response of the structure (capacity) and the term on the right-hand side as energy input (demand). The demand terms in Eq. (9) can be separated into two parts: recoverable and unrecoverable components. $E_K(t)$ and $E_E(t)$ are the recoverable components while $E_D(t)$ and $E_H(t)$ are the unrecoverable energies dissipated during the EQGM. Energy parameters such as total energy input and hysteretic energy is very important to understand the energy concept. Considering the inelastic behavior of the structure, unrecoverable part of the total input energy, E_I, is distributed through hysteretic energy, E_H, and viscous damping energy, E_D. The hysteretic energy is related to yielding of the structural members, i.e., damage in the structure occurs as a result of yielding. The quantities in Eq. (9) at the end of an EQGM can be determined and the distribution of hysteretic energy throughout the structure can be evaluated for a given structure and ground motions.

3. Evaluation of the seismic energy demands in SCBFs

3.1 Design of the structures

Inverted V and two-story X- type steel braced frames with 4-, 8-, 12- and 20-stories, representing typical low -, medium- and high-rise steel buildings were designed based on the seismic design requirements for SCBFs in accordance with ASCE 7-10 , AISC 360-10, AISC 341-10. The plan dimensions of buildings are 45 m × 45 m with span length of 9.0 m (five equal spans) for all stories in two orthogonal directions as shown in Fig. 1. The floor plans of the buildings are symmetrical. The columns are assumed to be simply connected to the ground. The braces are also simply connected to the beams and beam-column connections. The structural system for each building consists of seismically resistant perimeter inverted V-type and two-story X- type braced frames whereas the interior

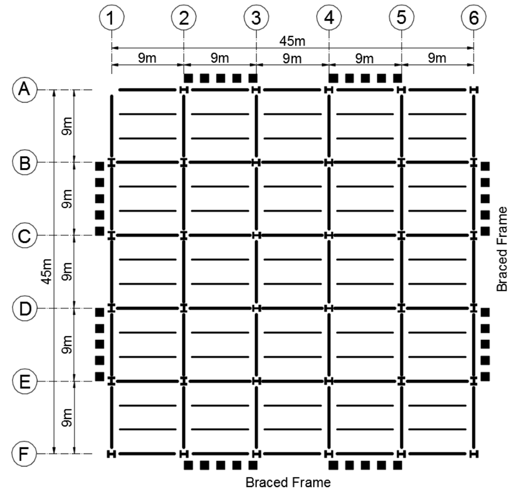


Fig. 1 Plan of the 4-, 8-, 12- and 20- story buildings

frames are designed for gravity loads only.

The typical story height for the frames is 4.0 m except for the 8-, 12-, and 20-story frames where the first story height is 5.0 m. For the 8-, 12-, and 20-story building, concrete foundation walls and surrounding soil are assumed to prevent any significant horizontal displacement of the structure at the ground level, i.e., the seismic base is assumed to be at the ground level. The linear analysis of buildings is applied by using Load and Resistance Factor Design (LRFD) methodology in accordance with AISC 360-10 (2010). Dead loads including self-weight of the members and live load used in the study are 5.0 kN/m^2 and 2.4 kN/m^2 except at the roof level, where they are taken as 4.0 kN/m^2 and 1.4 kN/m^2, respectively. European wide flange profiles with steel grades S355 are preferred for

Table 1 Earthquake ground motion characteristics from PEER database

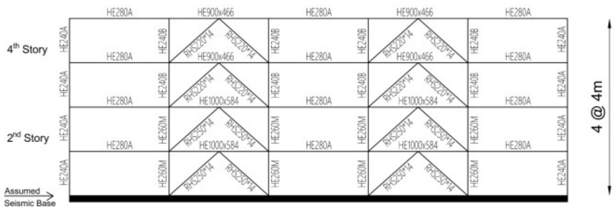
	NGA*	Record	Scale Factor	Duration (sec)	PGA (g)
GM 1(Design)	265	Victoria	1.453	24.54	0.937
GM 2(Design)	289	İrpinia	2.095	35.22	0.264
GM 3(Design)	587	New Zealand	1.560	49.38	0.375
GM 4 (Design)	881	Landers	1.898	56.07	0.423
GM 5 (Design)	1119	Kobe	0.600	40.99	0.418
GM 6 (Design)	4132	Park Field	2.088	21.20	0.766
GM 7 (Design)	8166	Düzce	1.768	35.02	0.700
GM 8 (MCE)	265	Victoria	2.179	24.54	1.407
GM 9 (MCE)	289	İrpinia	3.143	35.22	0.397
GM 10 (MCE)	587	New Zealand	2.340	49.38	0.563
GM 11 (MCE)	881	Landers	2.848	56.07	0.635
GM 12 (MCE)	1119	Kobe	3.132	40.99	0.628
GM 13 (MCE)	4132	Park Field	2.652	21.20	1.150
GM 14 (MCE)	8166	Düzce	0.900	35.02	1.051

* NGA: Sequential number in PEER Strong Ground Motion Database

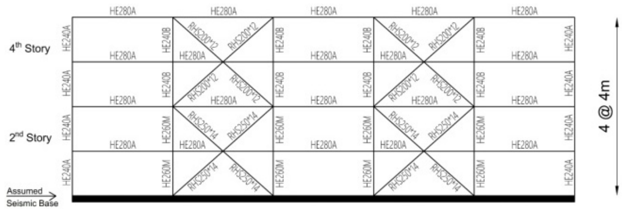
design. Hollow structural sections are used for braces with steel grade of S275. The member sizes of the frames are given in Fig. 2. Redundancy factor, ρ = 1.3, is assigned to the seismic force-resisting system (ASCE 7-10 2010).

Two sets of ground motion records are selected from PEER Strong Ground Motion Database corresponding to 10% and 2% probability of exceedance in 50 years. Each set consists of 7 ground motion records. Table 1 lists the characteristics of the selected ground motions. The response spectrum curves corresponding to design level and maximum considered earthquake level (MCE) are developed as specified in ASCE 7-10 and shown in Fig. 3. Shear wave velocity (V_s) of site class is assumed to vary between 300-770 m/s. The selected ground motions are then scaled to comply with the response spectrum (Fig. 3).

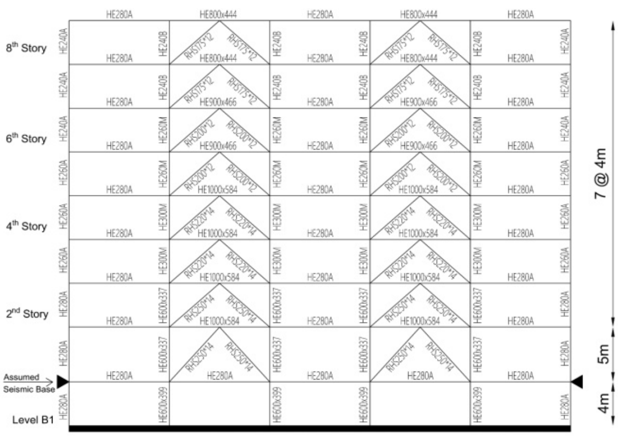
Response Spectrum Analysis are used in seismic design. Seismic design category D is taken for all structures in design. In cases where the combined response for the modal base shear (V_t) is less than 85% of the calculated base shear (V) using the equivalent lateral force procedure, the forces are increased by using coefficient ($0.85V/V_t$) in accordance with ASCE 7-10 (2010).



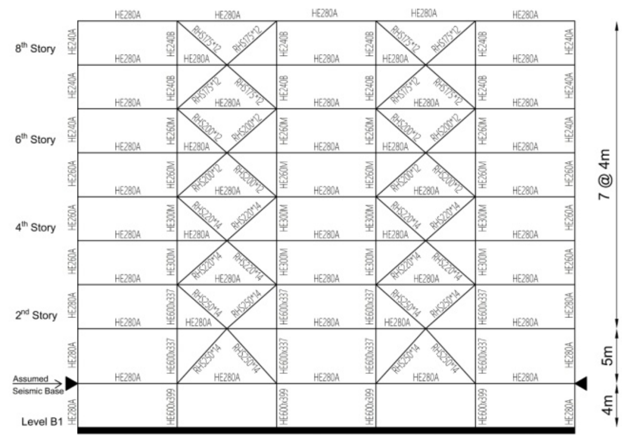
(a) 4-story inverted V-braced system



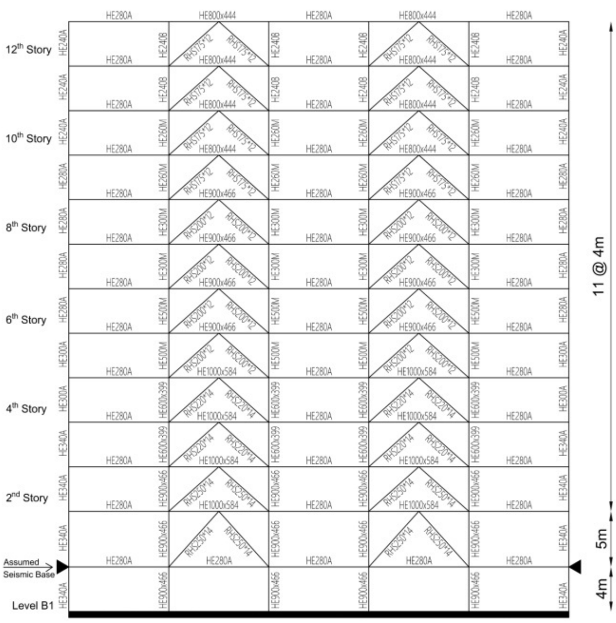
(b) 4-story X-braced system



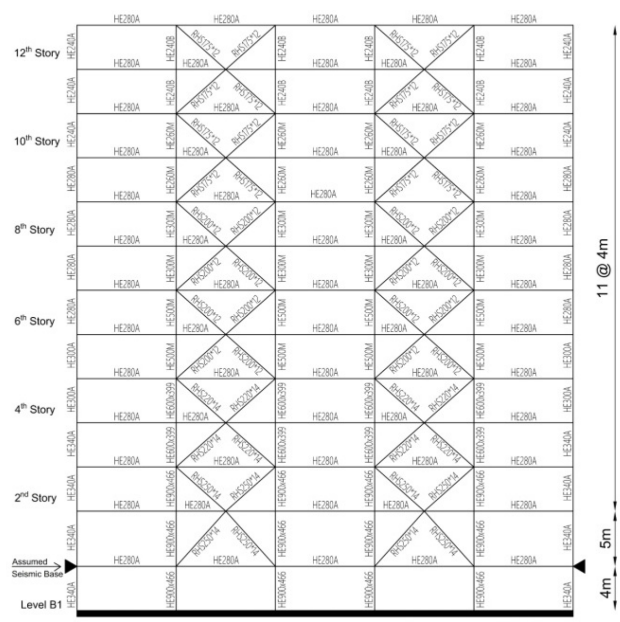
(c) 8-story inverted V-braced system



(d) 8-story X-braced system

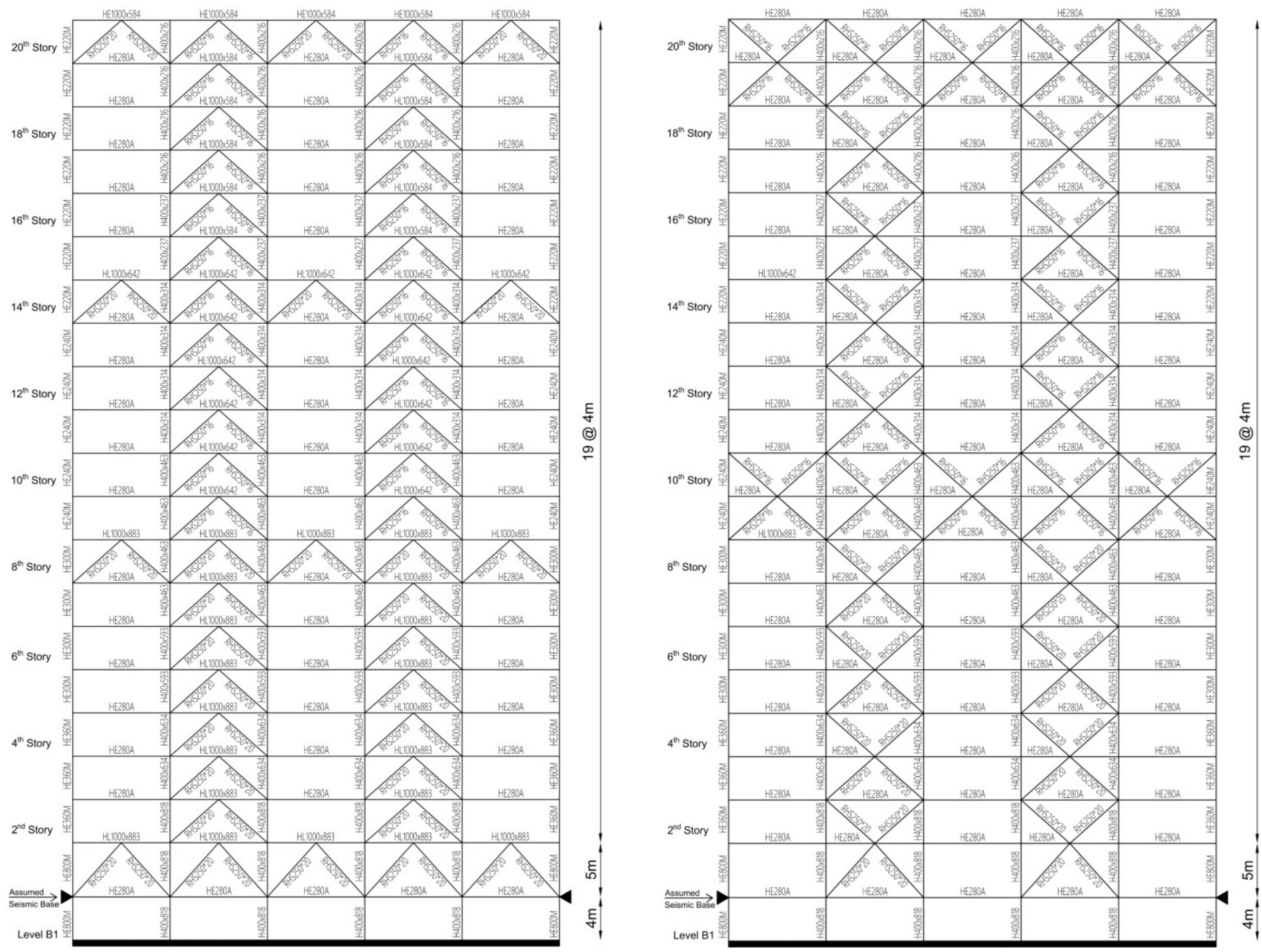


(e) 12-story inverted V-braced system



(f) 12-story X-braced system

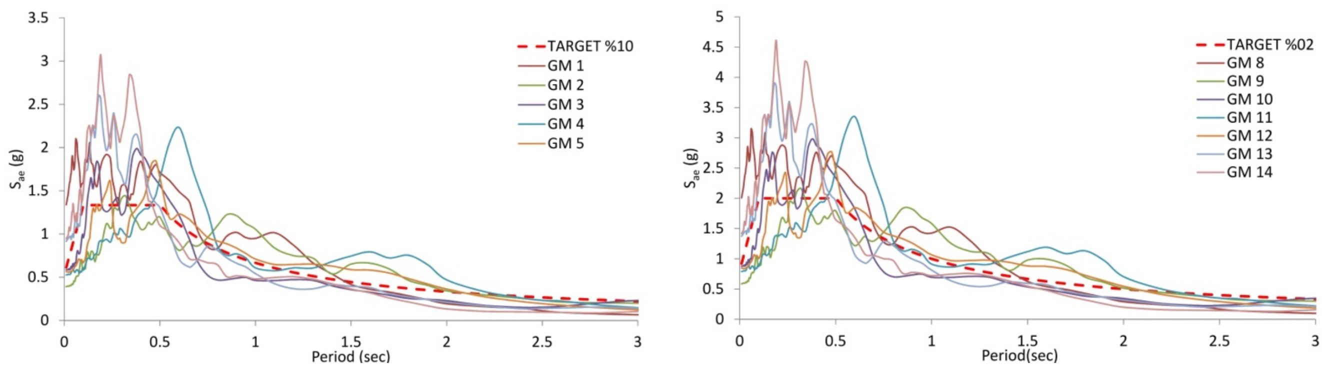
Fig. 2 Elevations and member sizes for the 4-, 8-, 12- and 20- story braced frames



(g) 20-story inverted V-braced system

(h) 20-story X-braced system

Fig. 2 Continued



(a) Design level

(b) MCE level

Fig. 3 Response Spectra of the scaled GMs selected from PEER database for Design Level (a) and MCE Level (b)

The calculated fundamental period of the structure (T) are checked with the approximate fundamental period (T_a) multiplied with coefficient of upper limit (C_u). Where the calculated fundamental period (T) exceeds C_uT_a, C_uT_a is used instead of T in accordance with ASCE 7-10 (2010). The design of the structural system is also checked by using the amplified seismic load effect including overstrength factor. The calculated fundamental periods of vibration, the total mass for the frames are given in Table 2. The floor system of the buildings is assumed to provide diaphragm action and to be rigid in the horizontal plane. In design of steel braced frames, the appropriate response modification coefficient (R = 6), overstrength factor (Ω_o = 2), and the deflection amplification factor (C_d = 5), are used in determining the base shear, element design forces, and design story drift where redundancy factor ρ is taken as 1.3.

The buildings are designed for a site where MCE spectral response acceleration parameters S_S is 2.0 g and S_1 is 1.0 g. Design earthquake spectral response acceleration parameters are taken as S_{DS} = 1.333 (g), S_{D1} = 0.666 (g). Long-period transition period, T_L is assumed to be 12.0 s. Story drift limitation is taken as 2% of story height as specified in ASCE7-10 (2010). The columns and braces are classified as highly ductile members, and beams satisfy the requirements for moderately ductile members according to AISC 341-10 (2010).

Table 2 Calculated fundamental periods and total mass for the SCBFs

Story	T (sec)		$T_{average}$ (sec)	Total mass (kN.sec^2/m)
	Inverted V	Two–story X		
4-Story	0.563	0.593	0.578	2173
8-Story	1.520	1.580	1.550	4478
12-Story	2.380	2.450	2.410	6875
20-Story	2.700	2.730	2.715	11420

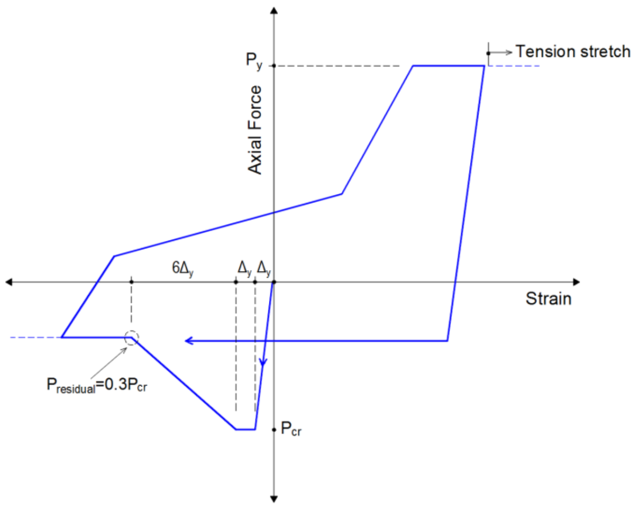


Fig. 4 Hysteresis loop of nonlinear steel brace element considering buckling

3.2 Design of the structures

Inelastic dynamic time history analyses are carried out to study the energy response parameters on the 4-, 8-, 12- and 20-story braced frames. The braced frames are subjected to two sets of ground motions (a total of 14 ground motions). PERFORM 3D (2011) – nonlinear time history analysis software- are used for numerical modeling and analyses. The inelastic axial behavior of a brace is modeled using a nonlinear steel bar element in PERFORM 3D. Hysteresis loop for axial behavior of braces is shown in Fig. 4. Nominal tensile strength, P_y, and nominal compressive strength (buckling strength), P_{cr}, are calculated in accordance with AISC 341-10 (2010). Residual compressive strength (post-buckled strength), $P_{residual}$ is taken as $0.3P_{cr}$ at axial deformation of $8\Delta_y$. Δ_y is the axial deformation at expected buckling load. The length of yield plateau is taken as Δ_y after buckling. Tension stretch effect that can occur during reloading in a cycle is considered by using a stretch factor of 0.05 (PERFORM-3D 2011). PERFORM 3D (2011) provides the dissipated energy for all elements by calculating the area under the hysteresis loop. The interaction between the axial force and bending moment is considered in columns. P-Δ effects are always included in the analyses. The selected ground motions corresponding to 10% (Design Level) probability of exceedance in 50 years are named as GM 1 to GM 7. These

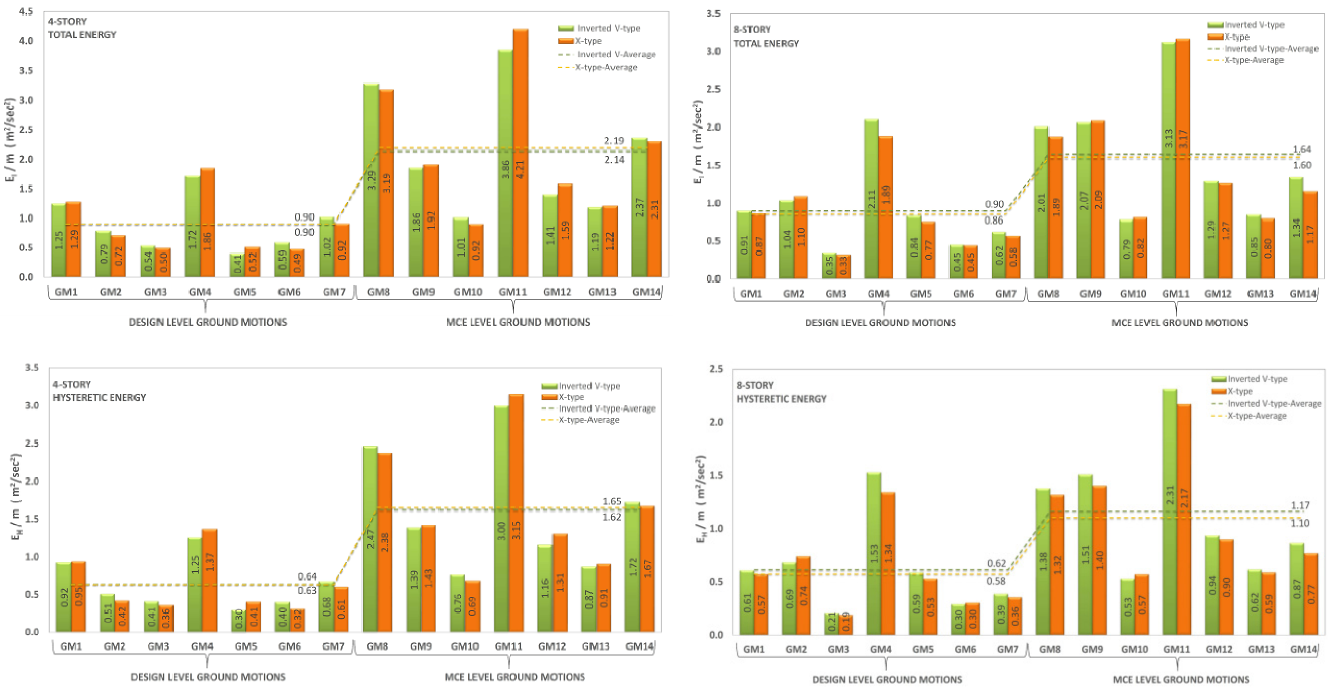


Fig. 5 Seismic energy parameters (E_I/m, E_H/m, E_H/E_I) for the 4-, 8-, 12- and 20- story frames

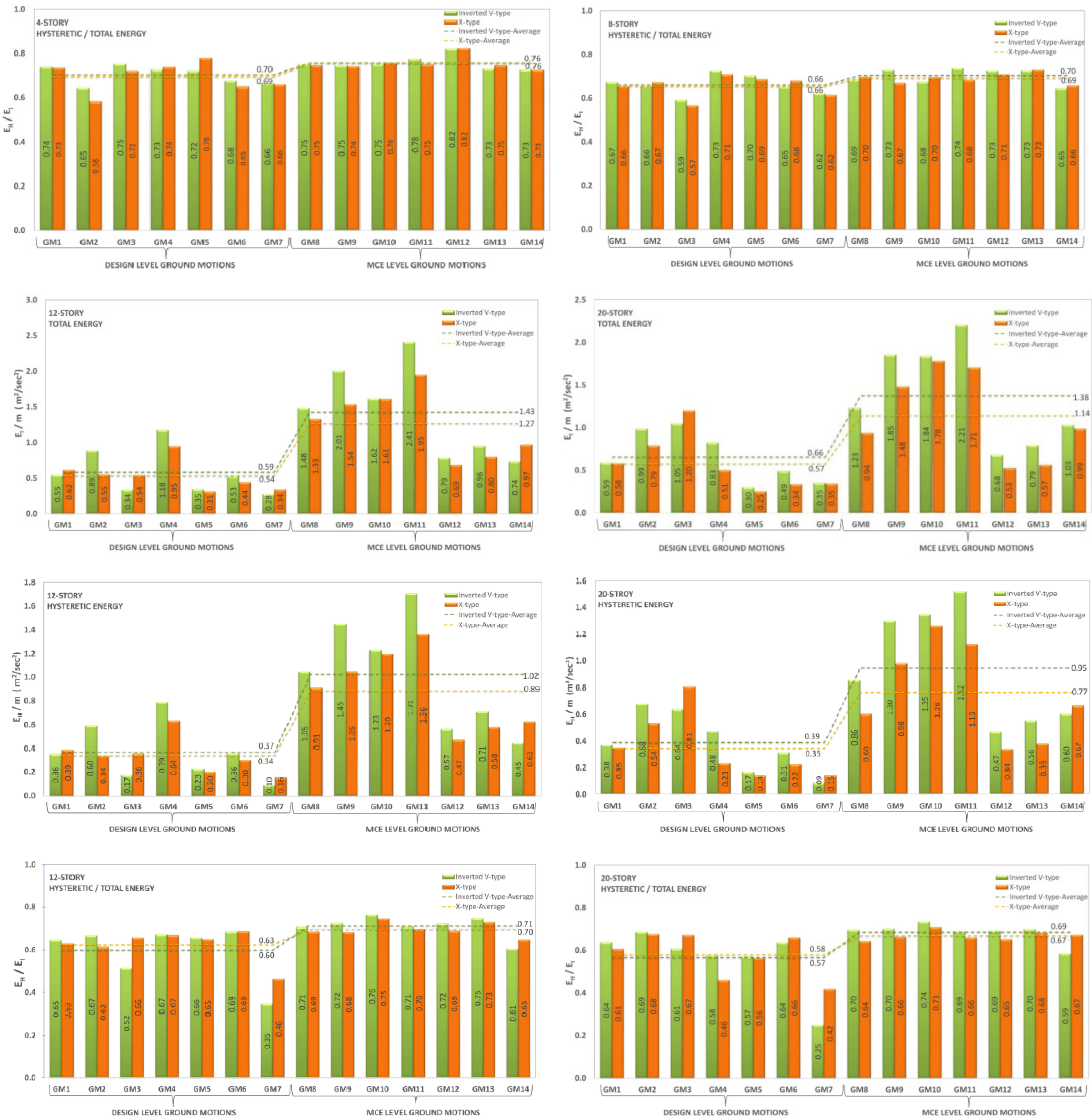
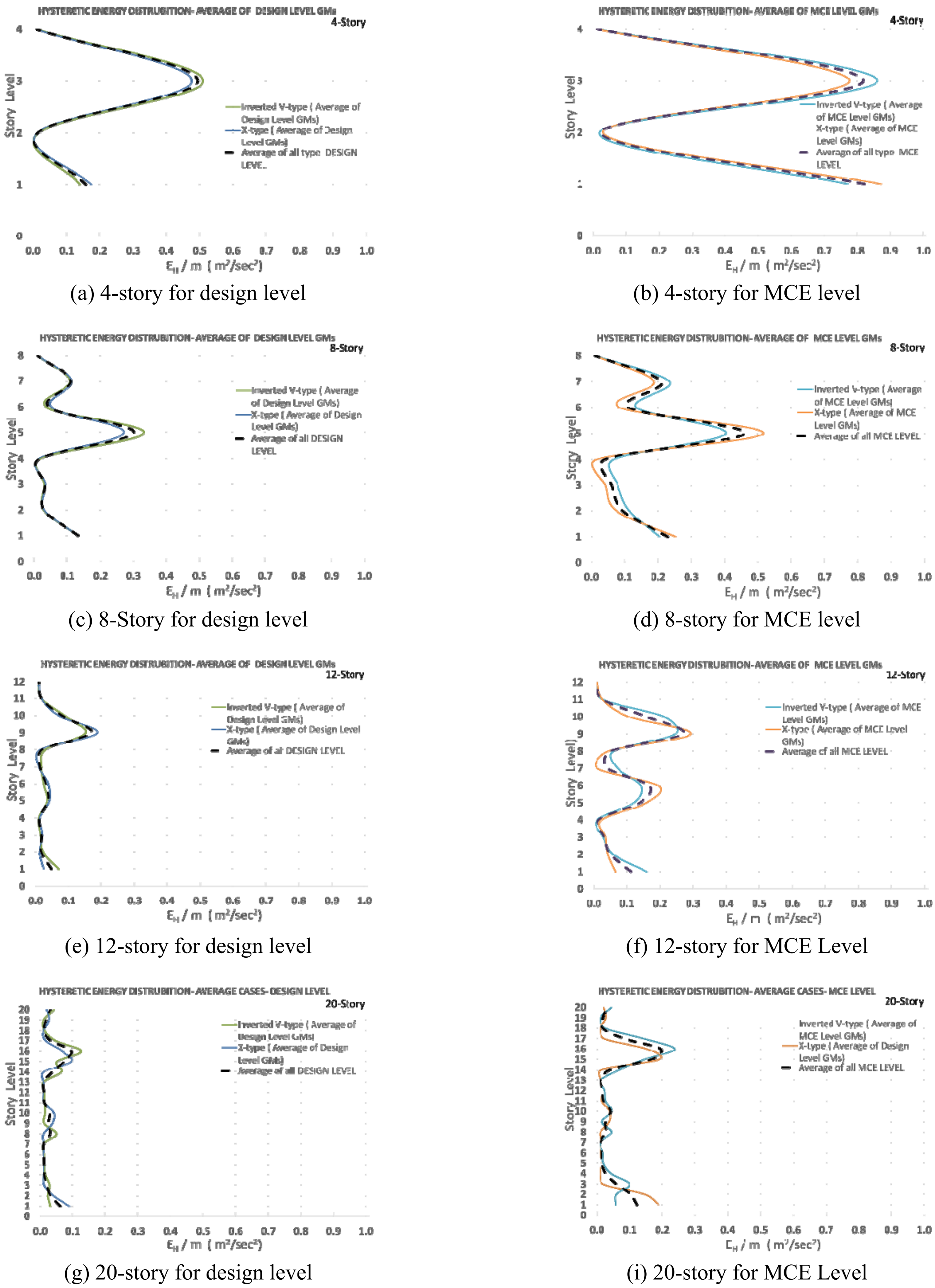


Fig. 5 Continued

ground motions are assumed to cause moderate structural damage. Similarly, the selected ground motions corresponding to 2% (Maximum Considered Earthquake Level-MCE) probability of exceedance in 50 years are designated as GM 8 to GM 14 which has a potential to cause heavy structural damage in the structure.

The seismic energy response parameters; total energy input (E_I), hysteretic energy (E_H) and the ratio of hysteretic energy to total energy input (E_H/E_I) normalized with respect to mass (m) in each bracing configuration are given in Fig. 5. These energy terms represent seismic energy demand. It is observed that the difference in seismic energy response parameters (E_I/m, E_H/m, E_H/E_I) is minor for Inverted *V* and two-story X- type steel braced frames subjected to Design Level and MCE Level ground motions. Thus, similar structural properties (fundamental period, mass) cause similar seismic energy demands. However, E_I/m and E_H/m are very sensitive to ground motions with different seismic intensity. Sensitivity in E_H/E_I is low which seems to be independent of seismic intensity. The maximum mean value for E_H/E_I for the 4-, 8-,12- and 20-story frames subjected to Design Level ground motions (GM1 to GM7) are 0.70, 0.66, 0.63 and 0.58, respectively, while they are 0.70, 0.70, 0.71 and 0.69, respectively, when subjected to MCE Level ground motions (GM8 to GM14). The E_I/m and E_H/m decrease when the number of stories increases. The greatest



(a) 4-story for design level
(b) 4-story for MCE level
(c) 8-Story for design level
(d) 8-story for MCE level
(e) 12-story for design level
(f) 12-story for MCE Level
(g) 20-story for design level
(i) 20-story for MCE Level

Fig. 6 Average of hysteretic energy (E_H/m) distribution along the 4-, 8-, 12- and 20- story frame

demand of E_I/m occurs for the 4- story frame. E_I/m for Design Level ground motions is in the range of 0.90 m^2/sec^2 - 0.57 m^2/sec^2 while it is in the range of 1.14 m^2/sec^2-2.19 m^2/sec^2 for MCE Level ground motions. Similarly, E_H/m for Design Level ground motions remains at 0.35 m^2/sec^2 to0.64 m^2/sec^2 while varies from 0.77 m^2/sec^2 to 1.65 m^2/sec^2 for MCE Level ground motions. The highest E_H/m demand occurs also for the 4-story frame which is a low-rise frame with more inelastic deformations per unit mass. On the other hand, E_H/m reduces when fundamental period increases, e.g. for medium and high-rise frames.

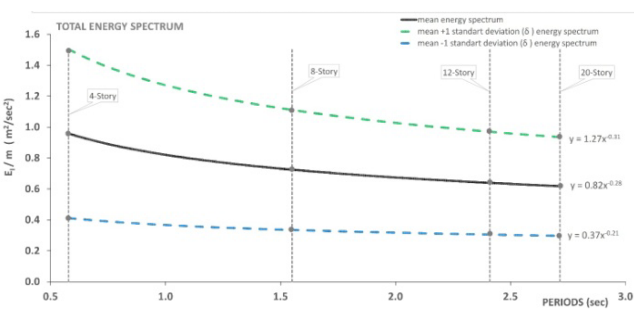
Hysteretic energy demand (E_H) for structural members in each story is computed by integrating the areas under the force-deformation hysteresis curves. The hysteretic energy distribution at each story is calculated by summing up the hysteretic energy demand, (E_H), for all structural members at the same story. Thus, the values of hysteretic energy distribution represent the inelastic dissipated energy at each story. The average hysteretic energy distributions for the 4-, 8-, 12- and 20- story braced frames with Inverted V and two-story X-type bracing configurations subjected to ground motions in Design Level and MCE Level are represented in Fig. 6. The hysteretic energy is mainly dissipated by the braces. The hysteretic energy dissipated by the columns is observed to be relatively very small. The hysteretic energy demand does not occur in the beams subjected to Design Level and MCE Level ground motions. Based on these results, it can be said that inelastic behavior of a braced steel frame is due to mostly the inelastic behavior of steel braces.

It is also observed that the shape of the hysteretic energy distribution of Inverted V and two-story X- type bracing configurations are similar, they have similar structural properties. As can be seen from Figs. 6(g) and (i), there is not a significant difference in the shape of hysteretic energy distribution for the 20-story frame except for the story levels which have continuous bracing system. For example, the 20-story frame with Inverted V-type bracing configuration has continuous bracing system at the first story, but the same frame does not have continuous bracing with X-type bracing configuration.

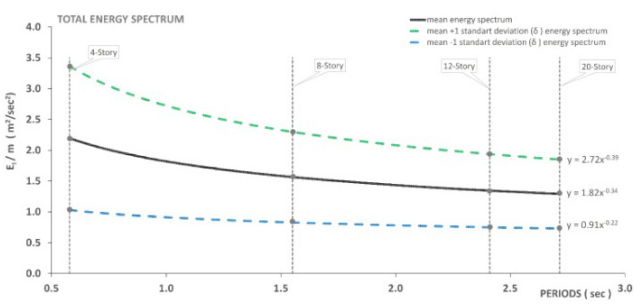
The average peak value for the hysteretic energy, E_H/m, distribution for the 4-story frame when subjected to Design Level ground motions (GM1 to GM7) is calculated as 0.50 m^2/sec^2 while it is 0.80 m^2/sec^2 for GM8 to GM14 ground motions corresponding to MCE level (Figs. 6(a) and (b)). The greatest demand for E_H/m occurs at the third story under Design Level ground motions while it occurs at the first and third stories under MCE Level ground motions. The braces at that stories dissipates most of the E_H/m. The average peak value of E_H/m for the 8-story frame is only 0.30 m^2/sec^2 under Design Level ground motions. It increases up to 0.45 m^2/sec^2 at the fifth story under MCE Level ground motions (Figs. 6(c) and (d)). The E_H/m is nearly zero at the top story. For the 12-story frame, the average peak value of E_H/m reduces to 0.17 m^2/sec^2 under Design Level ground motions. However, when subjected to MCE Level ground motions, average peak value of E_H/m increases to about 0.27 m^2/sec^2.

The average peak value of E_H/m is the smallest for the 20-story frames subjected to Design Level ground motions. It is only 0.10 m^2/sec^2. Even under MCE Level ground motions, it is only 0.20 m^2/sec^2. The significant portion of the E_H/m is dissipated by the braces at the fifteenth and sixteenth stories. It is also observed that hysteretic energy distribution does not follow a uniform pattern for the 4-, 8-, 12- and 20-story frames.

Seismic energy spectrums might also help structural engineers obtain seismic demands for structural framing systems within a certain period range for given ground motions. Using the nonlinear dynamic time history analyses results, total energy spectrum (E_I/m) and total hysteretic energy/total energy spectrum (E_H/E_I) are developed using seismic energy demands from the 4-, 8-, 12- and 20-story frames. Seismic energy spectrums can easily be applied in performance-based design corresponding to different earthquake hazard levels. The first step in such an energy-based design using performance-based design principles is to predict the energy input to the structure. It can be obtained practically from total energy spectrum (E_I/m). In the second step, to determine the inelastic dissipated energy (hysteretic energy), total hysteretic energy / total energy spectrum (E_H/E_I) can be used. Mass-normalized total energy spectrums to predict the energy input to the structure are

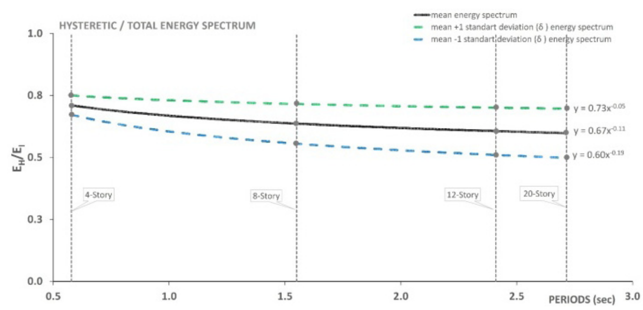


(a) Design level

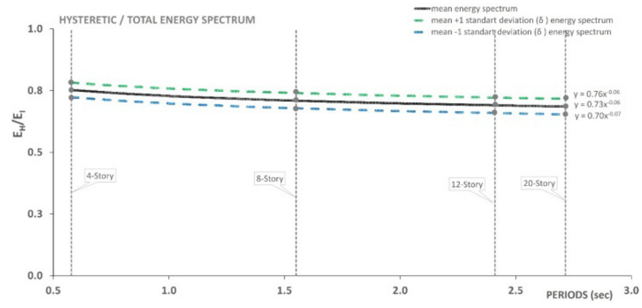


(b) MCE level

Fig. 7 Total energy spectrum



(a) Design level



(b) MCE level

Fig. 8 Hysteretic / total energy spectrum given for Design Level (Fig. 7(a)) and for MCE Level (Fig. 7(b)). Black line represents median total energy while green and blue dotted lines are for the median -/+ 1.0 standard deviation, respectively in Fig. 7. The total energy demands for 4-, 8-, 12- and 20-story are also marked at their corresponding periods. It is observed that total energy input spectrum shows downward tendency as the structural period increases in 0.6 -2.7 sec period range. As shown in Fig. 7, the 20-story frame have the smallest total energy input (E_I/m) among the all frames. It should be noted that the total energy input (E_I/m) is normalized with respect to mass. Total energy input (E_I) can then be easily calculated by multiplying the E_I/m by the mass of the structure. Considering the fact that the 20-story frame will have larger mass, the total energy input (E_I) during a strong ground motion will be higher compared to the total energy input (E_I) for the 4-story frames.

However, this does not mean that the higher the energy input, the heavier the damage to the structure. The structures with higher period and energy input are expected to experience less damage.

Total hysteretic energy/total energy spectrum (E_H/E_I) to predict the inelastic dissipated energy is given for Design Level (Fig. 8(a)) and for MCE Level (Fig. 8(b)) ground motions. The E_H/E_I ratio slightly decreases when the number of stories increases. However, the difference in E_H/E_I for Design and MCE Level ground motions is not significant.

The E_H/E_I ratio of 0.80 is quite conservative and can be assumed as an upper limit for both Design and MCE Level ground motions. The value of 0.80 can be practically used to calculate the hysteretic energy (E_H) if the total energy input (E_I) is known. It should be noted that the total energy input (E_I) and the hysteretic energy (E_H) in MCE level is larger than Design Level ground motions.

4. Conclusions

The aim of this paper is to determine seismic energy demands in steel special concentrically braced frames (SCBFs) for the evaluation of energy-based design concept in the framework of performance-based design. The study mainly focuses on demand parameters of the general design equation (demand ≤ capacity). The energy parameters are selected as total energy input, total hysteretic energy, ration of hysteretic energy to total energy input and the distribution hysteretic energy within a frame. Total energy input, hysteretic energy demands and the energy distributions are presented to understand how the energy demand parameters change for low, medium and high-rise steel special concentrically braced frames (SCBFs) with inverted V and two-story X-type bracing configurations. The inverted V and two-story X- type steel braced frames are designed in accordance with AISC 360-10 (2010), AISC 341-10 (2010) and ASCE 7-10 (2010) specifications. The 4-, 8-, 12- and 20-story SCBFs are selected and designed to represent low, medium and high rise buildings. The ratio of the inelastic hysteretic energy to total energy spectrum is derived as well as the total energy spectrum to predict the energy input to the structure. The ratio of the inelastic hysteretic energy to total energy spectrum is used to obtain hysteretic energy from total energy spectrum in a practical way. The use of the energy spectrum for a steel frame systems can help engineers specify the seismic energy input and hysteretic energy demand during the design process. Main results obtained in these study suggestions for further studies are summarized below.

- Seismic energy demands in terms of average total input energy (E_I/m), average hysteretic energy (E_H/m) and average total hysteretic energy/average total energy (E_H/E_I) appears to be relatively steady within each inverted V and two-story X- type bracing configurations.
- The distribution of total hysteretic energy, E_H/m for steel special concentrically braced frames (SCBFs) are presented to determine the inelastic dissipated energy at the each story. Braced frames designed in accordance with current design specifications distribute inelastic dissipated energy non uniformly along the height of the frames. However, shape of the hysteretic energy distribution is similar for different brace configurations for each 4-, 8-, 12- and 20-story frame.
- Hysteretic energy (E_H) at the top level is nearly zero for the 4-, 8- and 12-story braced frames and there is a small demand for the 20-story frame having continuous bracing at the top level.
- Total input energy with respect to mass, E_I/m, varies for 4-story to 8-, 12- and 20-story frames and has a decreasing tendency as the number of stories increase. E_I/m is not a constant value and depends on the properties of the structure as well as the ground motion properties.
- Total hysteretic energy (inelastic dissipated energy) with respect to mass, E_H/m also varies similarly to E_I/m. E_H/m is not constant value. The properties of the structure and ground motions have significant effect on E_H/m.
- Total energy spectrum, E_I/m, to predict the energy input for a structure under strong ground motions in Design and MCE Level ground motions is proposed. The proposed spectrum is valid for a period range of 0.6-2.7 sec. The spectrum of E_H/E_I, to estimate inelastic dissipated energy is developed. By using the proposed energy spectra, seismic energy demands, E_I/m and E_H/E_I can be obtained reliably using the fundamental period of a structure for different ground motion levels.
- E_H/E_I is assumed to be constant for all period ranges independent of the ground motion levels and can be taken as 0.80.
- Incremental dynamic analyses is a need to better understand the change of energy parameters under ever increasing ground motion intensity.
- For a complete design, energy dissipation capacities of braces should also be evaluated using experimental studies.

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Nomenclature

ASCE : American society of civil engineers
AISC : American institute of steel construction
MDOF : Multi degree of freedom system
SDOF : Single degree of freedom system
EQGM : Earthquake ground motion
GM : Ground motion
CP : Collapse prevention
LS : Life safety
SCBF : Steel special concentrically braced frame
SMRF : Steel moment resisting frame
LRFD : Load and resistance factor design
PEER : Pacific earthquake engineering research center
PGA : Peak ground acceleration
MCE : Maximum considered earthquake
R/C : Reinforced concrete
E_I : Total input energy
E_H : Hysteretic energy
E_D : Damping energy
E_K : Kinetic energy
E_E : Elastic strain energy
Eq. : Equation
m : Mass
u : Relative displacement
u_t : Total displacement
u_g : Earthquake ground motion displacement
f_s : Restoring force
k : Stiffness
c : Viscous damping coefficient
T : Period
C_u : Coefficient of upper limit
T_a : Approximate fundamental period
T_L : Long-period transition period
S_S : MCE spectral acceleration parameter at short period
S_1 : MCE spectral acceleration parameter at a period of 1s
ρ : Redundancy factor
R : Response modification coefficient
Ω_o : Overstrength factor
C_d : Deflection amplification factor